ÇUKUROVA UNIVERSITY
INSTITUTE OF NATURAL AND APPLIED SCIENCES

MSc THESIS

Meysam KHOSHBAKHT

DEVELOPING A COMPUTER PROGRAM TO PRE-DESIGN OF MINI HYDROELECTRIC POWER PLANTS

DEPARTMENT OF CIVIL ENGINEERING

ADANA, 2012

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We certify that the thesis titled above was reviewed and approved for the award of degree of the Master of Science by the board of jury on 12/10/2012.

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This MSc Thesis is written at the Department of Institute of Natural And Applied Sciences of Çukurova University.
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This work is supported by the Çukurova University Academic Research Projects Unit.
Proje No: MMF2012YL4

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ABSTRACT

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Year: 2012, page: 106
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A computer software is developed to help engineers for hydrologic and hydraulic design of mini hydropower plants in this study. Hydrologic design section can estimate the unit hydrograph by Rational, Snyder and Synthetic unit hydrograph methods. Hydraulic design section can compute the dimensions of ground intake, weir, stilling basin, headrace, settling basin, forebay and penstock. Developed software having independent sections can be used for the users to design some other hydraulic structures for hydrologic and hydraulic purposes besides hydropower plants.

Key Words: Hydroelectrical power, water power, hydroenergy hydropower plant, run-off river power plant

ÖZ

YÜKSEK LİSANS TEZİ

MİNİ HİDROELEKTRİK SANTRALLERİN ÖN TASARIMI İÇİN BİLGİSAYAR YAZILIMI GELİŞTİRİLMESİ

Meysam KHOSHBAKHT

ÇUKUROVA ÜNİVERSİTESİ
FEN BİLİMLERİ ENSTİTÜSÜ
İNŞAAT ANABİLİM DALI

Danışman : Prof. Dr. Recep YURTAL
Yıl: 2012, Sayfa:106
Jüri : Prof. Dr. Recep YURTAL
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Bu çalışmada mini hidroelektrik santrallerin hidrolojik ve hidrolik tasarımı için uygulayıcılara yardımcı olan bir bilgisayar yazılımı geliştirilmiştir. Hidrolojik tasarım bölümü Rasyonel, Snayder ve Sentetik Birim Hidrograf yöntemleri ile birim hidrograf tahmini yapabilmektedir. Hidrolik tasarım bölümü su alma yapısı, regülatör, düşü havuzu, iletim kanalı, çökelme havuzu, yükleme havuzu ve cebri borunun boyutlarını hesaplayabilmektedir. Bağımsız bölümlere sahip olan yazılım hidroelektrik santrallerin yanında digger su yapılarının hidrolojik ve hidrolik tasarımında da kullanılabilir.

Anahtar Kelimeler: Hidroelektrik enerji, su kuvveti, hidroenerji, hidroelektrik santral, nehir tipi hidroelektrik santral

ACKNOWLEDGEMENTS

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LIST OF ABBREVIATONS

Q	: Peak rate of runoff in cubic meters per second
C	: Runoff coefficient
i	: Average intensity of rainfall
Ca	: Antecedent moisture factor
A	: Drainage areas in hectares
Kc	: Unit conversion factor
V	: Velocity
K	: Interception coefficient
Sp	: Slope (percent)
L	: Length of shallow concentrated flow
R	: Hydraulic radius
Ku	: Units conversion factor equal to 1
n	: Manning's roughness coefficient
Tc	: Time of concentration
TL	: Lag time
Ct	: Empirical watershed coefficient
Lca	: Length along main channel from outlet to a point opposite the watershed centroid
$T_{L(adj)}$	: Adjusted lag time for the new duration
T_R	: Original unit duration
T'_R.	: Desired unit duration
p	: Unit peak discharge
C_p	: Empirical constant ranging
α.	: Conversion constant
T_B	: Time of the synthetic unit hydrograph
α_{50}	: Unit conversion constant
α_{75}	: Unit conversion constant
t_p	: Peak runoff in hour

h_0 : Flood head

L_{weir} : Length of weir

P_{weir} : Lag time

Ct : Empirical watershed coefficient

Q_{design} : Design discharge

μ : Flow coefficient

g : Gravity acceleration

a : Clearance between rake bars

d : Distance between rake bars

ß : Rake slope

b : Rake width

L : Rake length

H_{crit} : Critical water depth

h : Orthogonal water depth at the beginning of the rake

H_B : Water level in afterbay

d_1. : Supercritical flow depth

d_1 : Subcritical flow depth

E_2 : Downstream energy level

E_1 : Upstream energy level

Fr : Froude number

d : Penstock diameter

r_i : Internal radius of the penstock

σ_u : Ultimate tensile strength

p_i : Internal maximum assumed pressure at the regarded penstock section

$P_{turbine}$: Electrical power output

η : Over flow efficiency

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LIST OF ABBREVIATONS

Q	: Peak rate of runoff in cubic meters per second
C	: Runoff coefficient
i	: Average intensity of rainfall
Ca	: Antecedent moisture factor
A	: Drainage areas in hectares
Kc	: Unit conversion factor
V	: Velocity
K	: Interception coefficient
Sp	: Slope (percent)
L	: Length of shallow concentrated flow
R	: Hydraulic radius
Ku	: Units conversion factor equal to 1
n	: Manning's roughness coefficient
Tc	: Time of concentration
TL	: Lag time
Ct	: Empirical watershed coefficient
Lca	: Length along main channel from outlet to a point opposite the watershed centroid
$T_{L(adj)}$	: Adjusted lag time for the new duration
T_R	: Original unit duration
T'_R.	: Desired unit duration
p	: Unit peak discharge
C_p	: Empirical constant ranging
α.	: Conversion constant
T_B	: Time of the synthetic unit hydrograph
α_{50}	: Unit conversion constant
α_{75}	: Unit conversion constant
t_p	: Peak runoff in hour

h_0 : Flood head

L_{weir} : Length of weir

P_{weir} : Lag time

Ct : Empirical watershed coefficient

Q_{design} : Design discharge

μ : Flow coefficient

g : Gravity acceleration

a : Clearance between rake bars

d : Distance between rake bars

ß : Rake slope

b : Rake width

L : Rake length

H_{crit} : Critical water depth

h : Orthogonal water depth at the beginning of the rake

H_B : Water level in afterbay

d_1. : Supercritical flow depth

d_1 : Subcritical flow depth

E_2 : Downstream energy level

E_1 : Upstream energy level

Fr : Froude number

d : Penstock diameter

r_i : Internal radius of the penstock

σ_u : Ultimate tensile strength

p_i : Internal maximum assumed pressure at the regarded penstock section

$P_{turbine}$: Electrical power output

η : Over flow efficiency

1. INTRODUCTION

1.1. HYDROPOWER BASICS

By decreasing fossil fuel resources in the world, necessity of using renewable resources is felt. Nowadays, application of the these sources instead of the fossil's sources are as a solution of this problem. One of this renewable resources is hydroelectric power plant that generates electricity without damaging nature and polluting air.

Hydraulic power can be captured wherever a flow of water falls from a higher level to a lower level. This may occur where a stream runs down a hillside, or a river passes over a waterfall or man-made weir, or where a reservoir discharges water back into the main river. The vertical fall of water, known as the head, is essential for hydropower generation; fast-flowing water on its own does not contain sufficient energy for useful power production except on a very large scale, such as offshore marine currents. Hence, two quantities are required: a Flow Rate of water Q, and a Head, H. It is generally better to have more head than more flow, since this keeps the equipment smaller. The Gross Head (H) is the maximum available vertical fall in the water, from the upstream level to the downstream level. The actual head seen by a turbine will be slightly less than the gross head due to losses incurred when transferring the water into and away from the machine. This reduced head is known as the Net Head (The British Hydropower Association, 2005).

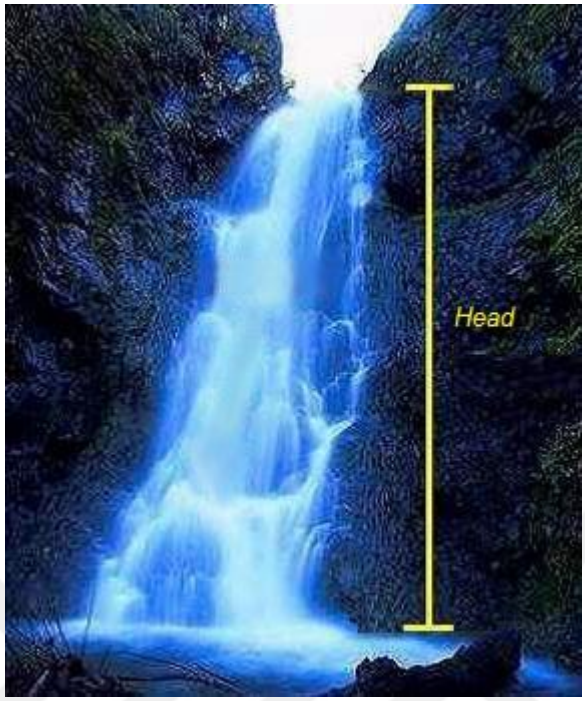


Figure 1. 1. Hydropower head (The British Hydropower Association, 2005).

1.2. Power and Energy

Energy is an amount of work done, or a capacity to do work, measured in Joules. Hydro-turbines convert water pressure into mechanical shaft power, which can be used to drive an electricity generator, or other machinery. The power available is proportional to the product of head and flow rate. The general formula for any hydro system's power output is (The British Hydropower Association, 2005):

(1.1.)

$$P = \eta \times \rho \times g \times Q \times H$$

Where

P : the mechanical power produced at the turbine shaft in Watts,

η : the hydraulic efficiency of the turbine, ρ is the density of water (1000 kg/m^3),

g : the acceleration due to gravity (9.81m/s^2),

Q : the volume flow rate passing through the turbine (m^3/s), and

H : the effective pressure head of water across the turbine (m).

1.3. Hydropower in the World

Hydropower is the most important source of renewable energy in the world for electrical power production. The world's technically feasible hydro potential is estimated as 14,370 TWh/year, which is equal to today's global electricity demand. The economically feasible proportion of this is 8,080 TWh/year. The exploited hydropower potential in the world in 1999 was 2,650 TWh which is about 19% of the world's electricity (Paish, 2002).

1.4. Classification of Hydropower Plants

The classification of hydropower plants can be based on different factors:

- Head: low (less than 50m), medium (between 50 and 250m), high (greater than 250m).
- Exploitation and storage: with daily (or seasonal) flow regulation (reservoir type), without flow regulation (runoff the river type).
- Conveyance system: pressurised (penstock), mixed circuit (canal and penstock).
- Powerhouse site: dam or diversion scheme.
- Energy conversion mode: turbining or reversible pumping-turbining.
- Type of turbines: impulse, reaction and reversible.
- Installed power: micro (Pt < 100 kW), mini (100 kW < Pt < 500 kW), small (500 kW < Pt < 10 MW).

The classification based on the power is very important because it is an institutional and legislate reference (Ramos et al, 2000).

1.5. Why Mini Hydropower?

Small-scale hydropower is one of the most cost-effective and reliable energy technologies to be considered for providing clean electricity generation. In particular, the key advantages that small hydro has over wind, wave and solar power are:

- A high efficiency (70 - 90%), by far the best of all energy technologies.
- A high capacity factor (typically >50%), compared with 10% for solar and 30% for wind.
- A high level of predictability, varying with annual rainfall patterns.
- Slow rate of change; the output power varies only gradually from day to day (not from minute to minute).
- A good correlation with demand i.e. output is maximum in winter.
- It is a long-lasting and robust technology; systems can readily be engineered to last for 50 years or more (The British Hydropower Association, 2005).

It is also environmentally benign. Small hydro is in most cases “runoff river” ;in other words, any dam or barrage is quite small, usually just a weir, and little or no water is stored. Therefore, runoff river installations do not have the same kinds of adverse effects on the local environment as large-scale hydro (The British Hydropower Association, 2005).

1.6. Components of a Hydropower Plants

1.6.1. Intake

The intake is built directly in the river. A weir built crossways to the flow direction dams the water. This slack flow allows a regulation of the water conducted to the canal. In dry phases all the water can then be diverted directly into the canal. If there is too much water, however, the intake of the canal can be throttled. The extra

water flows along the weir. In this way, one can avoid too much water entering the canal and possibly demolishing constructions. With strongly polluted water, a desilting basin must be arranged directly after the intake (Ardüser and Karcheter, 2009).

1.6.2. Headrace

To convey water from the intake to the forebay.

1.6.3. Settling Basin and Forebay Tank

The assignment of the desilting basin (sand trap) is to settle out the particulate matters floating in the water to the bottom of the construction. The water which is used for the turbine can; therefore, be separated from these solids. Otherwise, they will end up in the penstock and in the turbine which can lead to serious damage. The water volume of the forebay is made for balancing the variations of the water gauge while operating the turbine. In this example, the balancing basin is combined with the settling basin. Therefore, the water volume of the settling basin can be counted as that of the forebay tank, which leads to savings on construction material, because only one construction must be built (Ardüser and Karcheter, 2009).

1.6.4. Penstock

In the penstock, the water pressure is built up on the turbine. The water flows out of the forebay tank through the penstock directly on to the turbine. The water gauge in the forebay tank controlls the head. (Ardüser and Karcheter, 2009).

1.6.5. Powerhouse and Turbine

The turbine and the equipment required for the production of electricity are located in the powerhouse and therefore are protected from rain and other factors.

The turbined water is subsequently restored in an open canal to the stream. The produced electricity is carried to consumers using the transmission line (Ardüser and Karcheter, 2009).

1.7. Scheme of Development Layout

The tree types of run of river hydropower are shown below that an engineer can decide to use one of them.

1.7.1. Short Penstock

In this case, the penstock is short but the channel is long. The long channel is exposed to the greater risk of blockage, or of collapse or deterioration as a result of poor maintenance. Installing the channel across a steep slope may be difficult and expensive. The risk that the steep slope may erode makes the short penstock layout an unacceptable option, because the projected operation and maintenance cost of the scheme could be very expensive, and it may outweigh the benefit of initial purchase cost (Department of Energy, 2009).

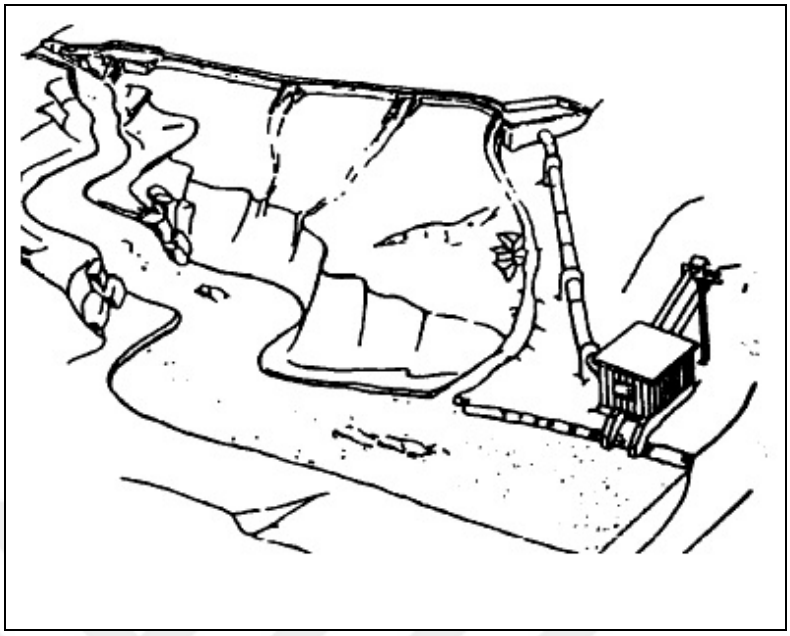


Figure 1. 2. Short penstock (Department of Energy, 2009).

1.7.2. Long Penstock

In this case, the penstock follows the river. If this layout is necessary, because the terrain would not allow the construction of a channel, certain precautions must be taken. The most important consideration is to ensure that seasonal flooding of the river will not damage or deteriorate the penstock. It is also important to calculate the most economic diameter of penstock; in the case of a long penstock, the cost will be particularly high (Department of Energy, 2009).

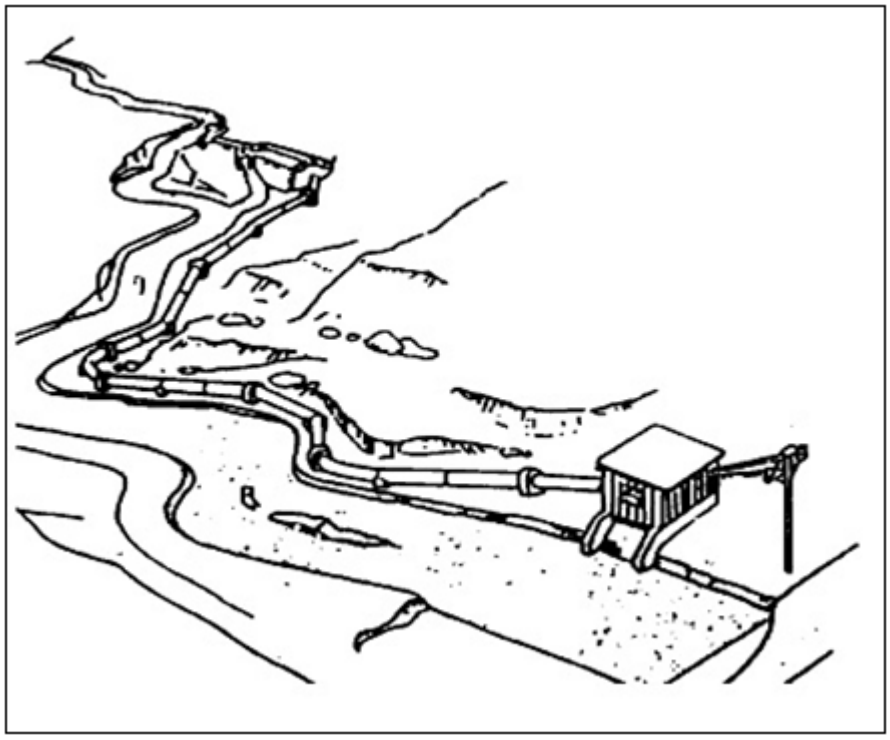


Figure 1. 3. Long penstock (Department of Energy, 2009).

1.7.3. Mid Length Penstock

Mid-length Penstock The mid-length penstock may cost more than the short penstock, but the cost of constructing a channel that can safely cross the steep slope may also be avoided. Even if the initial purchase and construction costs are greater in this case, this option may be preferable in case there are signs of instability in the steep slope (Department of Energy, 2009).

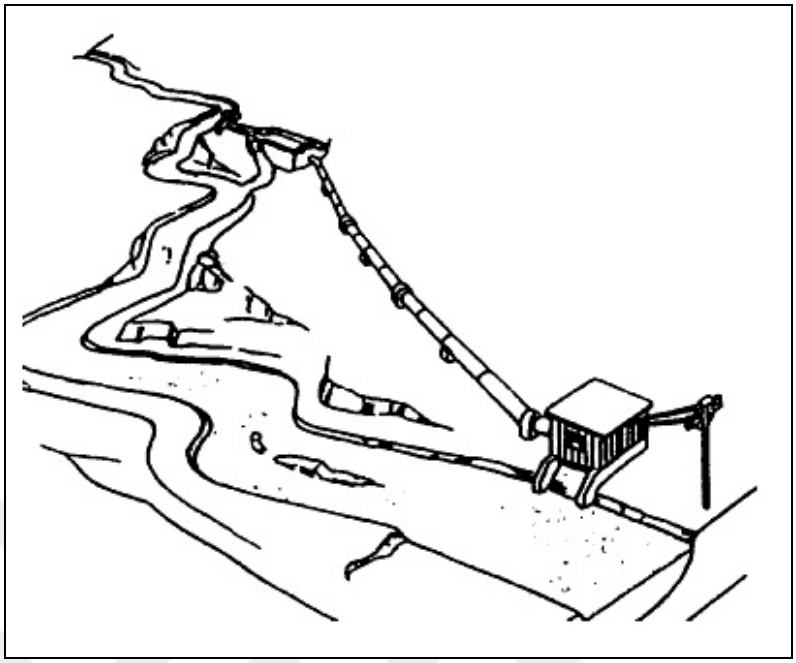


Figure 1. 4. Mid length penstock (Department of Energy, 2009).

1.8. The Outline of Thesis

In this study, the type of hydropower will be a high head runoff river and the optimum design criteria in addition to the methods currently used for mini hydroelectric power plants will be investigated. A program use for this purpose will either be obtained. The results will be discussed after application on one or several mini hydropower plants. New optimum design criteria or modification of the current criteria and methods will be proposed at the end of the study.

2. PREVIOUS STUDIES

2.1. Previous Studies About Hydroelectric Power Plants

A thesis was written by Ian David JONES (1988) about *Assessment and Design of Small Scale Hydroelectric Power Plants*. In this study the selection of the hydropower type, its components, system's cost and economic evolution were investigated.

A thesis was written by Tamene ADUGNA (2004) about *Optimization Of Small Hydropower in the Abbay Basin.* This study considers assessment of the selection of optimum sites or the optimum level of development at the sites can be undertaken with the application of non-linear optimization techniques.

A thesis was written by Ozan KORKMAZ (2007) about *Small Hydroelectric Power Plants by Using RETScreen Program.* In this study energy and cost equation dealing with energy generation and cost estimation of various items of the small hydroelectric power plants were applied by using the RETScreen software.

A thesis was written by Ebru ÖZBAY (2009) about *Modeling and Simulation of Small Hydroelectric Power Plants.* In her study, the roles of small hydroelectric power plants in hydroelectric energy production of Turkey were investigated.

An article was written by Heng LEI and Ying LI (2011) about *MATLAB Calculation of Hydraulic Transients in Hydropower Simulation Application.* In their study, they introduced the today's advanced scientific computing software MATLAB, and in their study, they used the software of the Zipingpu hydropower station hydraulic transients from the mathematical model and calculation simulation etc. were analyzed and calculated in order to seek appropriate power operation mode, unit close regularities.

A thesis was written by Emir ALİMOĞLU (2012) about *Run of River Hydroelectric Power Plants.* In his thesis, a computer program "MinuHEPP hydraulic design" was developed to hydraulic design of this type of hydropower.

2.2. Previous Studies About Potential of the Hydroelectric Power Plants

A thesis was written by Veysel ÖZKÖK (2006) studied about *Methods for Estimating Hydroelectric Potential and Their Applications.* Two main methods was used for estimation of hydroelectric potential; flow duration curve method and sequential stream flow routing method. Flow-duration curve method applied to the eight chosen flow gauging stations and sequential stream flow routing method was applied to the last 5 years monthly average data of Oymapinar dam.

A thesis was written by Mustafa AKDOGAR (2006) studied about *Energy Sources and Hydroelectric Potential Balance Study of East Black Sea Region.* In his study, the costs at the electric production with the definitions, potentials of energy sources were studied.

3. MATERIAL AND METHOD

3.1. Material

The program is developed by using MATLAB to calculate the total head, head losses, discharge rate and all of the hydropower elements dimensions.

3.2. Methods

3.2.1. Hydrologic Methods

One of the basic discussions in design of hydraulic structures is the determination of hydrologic parameters. Because by recognizing these parameters and correct evaluate of them, suitable design and analysis can be done for constructing our structures. On the other hand, parameters which are obtained by hydrologic analysis are accepted as loading hydraulic structures.

The most important of these parameters is flood discharge. There are several ways to count it, but most of them depend on the characters of the catchment such as area, slope, kind of the catchment etc. In this study, three methods of them, which are only different in limitation of the area, are selected. Because some methods give us a correct results only in particular of the limit of the area. These three methods are given below.

1. Rational Method.
2. Snyder Method.
3. Synthetic Unit Hydrograph Method (SCS or NRSC).

3.2.1.1. Rational Method

One of the simplest and most famous methods to compute the peak discharge is the Rational method. This method has a limitation and some assumption which will be mentioned them below:

The design discharge for areas proposed for building development and redevelopment should not exceed 40 hectares in catchment or contributing drainage area can be made using the Rational Method. This method is based on empirical data and hypothetical rainfall runoff events, which are assumed to model natural storm events. During an actual storm event, the peak discharge is dependent on many factors including antecedent moisture conditions; rainfall magnitude, intensity, duration, and distribution; and, the effects of infiltration, detention, retention, and flow routing throughout the watershed. The accuracy of the Rational Method is highly dependent upon the judgment and experience of the user. The method's simplicity belies the complexity in predicting a watershed's response to a rainfall event, especially when the Rational Method is used to predict post-development runoff. For that reason, the engineer must select the appropriate runoff coefficient and determine the time of concentration based on plan information (including proposed hydrologic changes) and experience in working with development and its effects on hydrology within the watershed (City of San Luis Obispo, 2003).

3.2.1.1.(1). Assumptions Inherent in the Rational Formula Are As Follows

- Peak flow occurs when the entire watershed is contributing to the flow.
- Rainfall intensity is the same over the entire drainage area.
- Rainfall intensity is uniform over a time duration equal to the time of concentration, T. The time of concentration is the time required for water to travel from the hydraulically most remote point of the basin to the point of interest.

- The frequency of the computed peak flow is the same as that of the rainfall intensity, i.e., the 10-year rainfall intensity is assumed to produce the 10-year peak flow.
- Coefficient of runoff is the same for all storms of all recurrence probabilities (Mccuen et al, 2009).

3.2.1.1.(2). The Rational Method Equation

$$Q = \frac{C \times i \times C_a \times A}{Kc} \qquad (3.1.)$$

Where

Q = Peak rate of runoff (m^3/s),

C = Runoff coefficient (Table 3.1),

i = Average intensity of rainfall for the time of concentration (Tc) for a selected design storm (mm/h),

C_a = Antecedent moisture factor (Table 3.5),

A = Drainage areas (hectares), and

Kc = Unit conversion factor equal to 360 in SI units.

3.2.1.1.(3). Determination of the Runoff Coefficients

The calculation of the runoff coefficient depended on the kind of the cover of drainage area. Obtaining the total runoff coefficient is given below:

The runoff coefficient for the drainage area was determined by using Table 3.1. If the landuse and soil cover are homogeneous for the entire drainage area, a single runoff coefficient value can be determined directly from the table. If there are multiple landuse or soil conditions, a weighted average must be calculated as follows (City of San Luis Obispo, 2003):

$$\sum\left(C1\ \frac{(\text{Area C1})}{\text{Total Area}} + C2\ \frac{(\text{Area C2})}{\text{Total Area}} + \ldots\right) = \text{Composite C value} \qquad (3.2.)$$

Table 3. 1. Runoff coefficients (City of San Luis Obispo, 2003).

Type of Developments	Hydrologic Soil Group	Run-off Coefficients for Slopes <2%	2-10%	>10%
Single-Family Residential Lots				
1,860 sq. m (20,000 sq. ft.)	D	0.40	0.45	0.55
	C	0.30	0.40	0.50
930 sq. m (10,000 sq. ft.)	D	0.40	0.50	0.60
"	C	0.35	0.40	0.50
560 sq. m (6,000 sq. ft.)	D	0.50	0.60	0.65
"	C	0.45	0.50	0.60
Apartments				
1,800 sq. ft. (167 sq. m)	C	0.60	0.70	0.80
"	D	0.50	0.60	0.70
Heavy Industrial	D	0.85	0.87	0.90
	C	0.80	0.85	0.87
Light Industrial	D	0.80	0.85	0.87
"	C	0.70	0.75	.80
Downtown Commercial	D	0.85	0.87	0.90
"	C	0.80	0.82	0.85
Neighborhood Commercial	D	0.65	0.75	0.80
	C	.50	0.60	0.70
Dense Vegetation	D	0.25	0.30	0.40
(oak woodland, brushland)	C	0.20	0.25	0.35
Moderate Vegetation	D	0.25	0.35	0.45
(grasslands w/scattered trees & brush)	C	0.25	0.30	0.35
Sparse Vegetation	D	0.40	0.45	0.50
(grasslands and pasture)	C	0.30	0.35	0.40
Agricultural	D	0.20	0.20	0.25
(cropland)	C	0.15	0.15	0.20
Impervious Surfaces		0.85	0.87	
(streets, parking lots, garages and roofs)		0.80	0.85	0.90
Unimproved Vacant Lands	D	0.15	0.20	0.30
(parks, cemeteries, golf courses, and lawns)	C	0.10	0.15	0.20

Notes:

These values are intended to be a minimum; higher values may be required by the City Engineer or County Public Works Director.

Hydrologic Soil Group

C = Sandy Loam, Gravel, Loam
D = Clay, Adobe, Shallow Soil and/or Rockland. Refer to USDA San Luis Obispo Area Soil Survey for hydrologic soil groups.

3.2.1.1.(4). Determination of the Time of Concentration

Time of concentration is the time required for runoff to flow from the most hydraulically remote part of the drainage area to the point under consideration. As runoff moves down the hydrologic flow path, flow is characterized into three types or regimes:

- Overland Flow (or sheet flow)
- Shallow Concentrated Flow
- Channel or Pipe Flow

Procedures for estimating the time associated with each of these flow types are presented in the following sections. The minimum time of concentration shall be 10 minutes (City of San Luis Obispo, 2003).

3.2.1.1.(4).(a). Overland Flow

Overland flow or sheet flow is shallow flow (usually less than on 25mm deep) over planar surfaces. For the purposes of determining the time of concentration, overland flow occurs in the upper reaches of the basin. The length of the overland flow is usually less than 90m prior to entering shallow concentrated flow path. The recommended maximum length for this type of flow is 90m. The travel time for overland flow may be determined by using the Overland Flow Chart (Seelye Chart) on Figure 3.1 (City of San Luis Obispo, 2003).

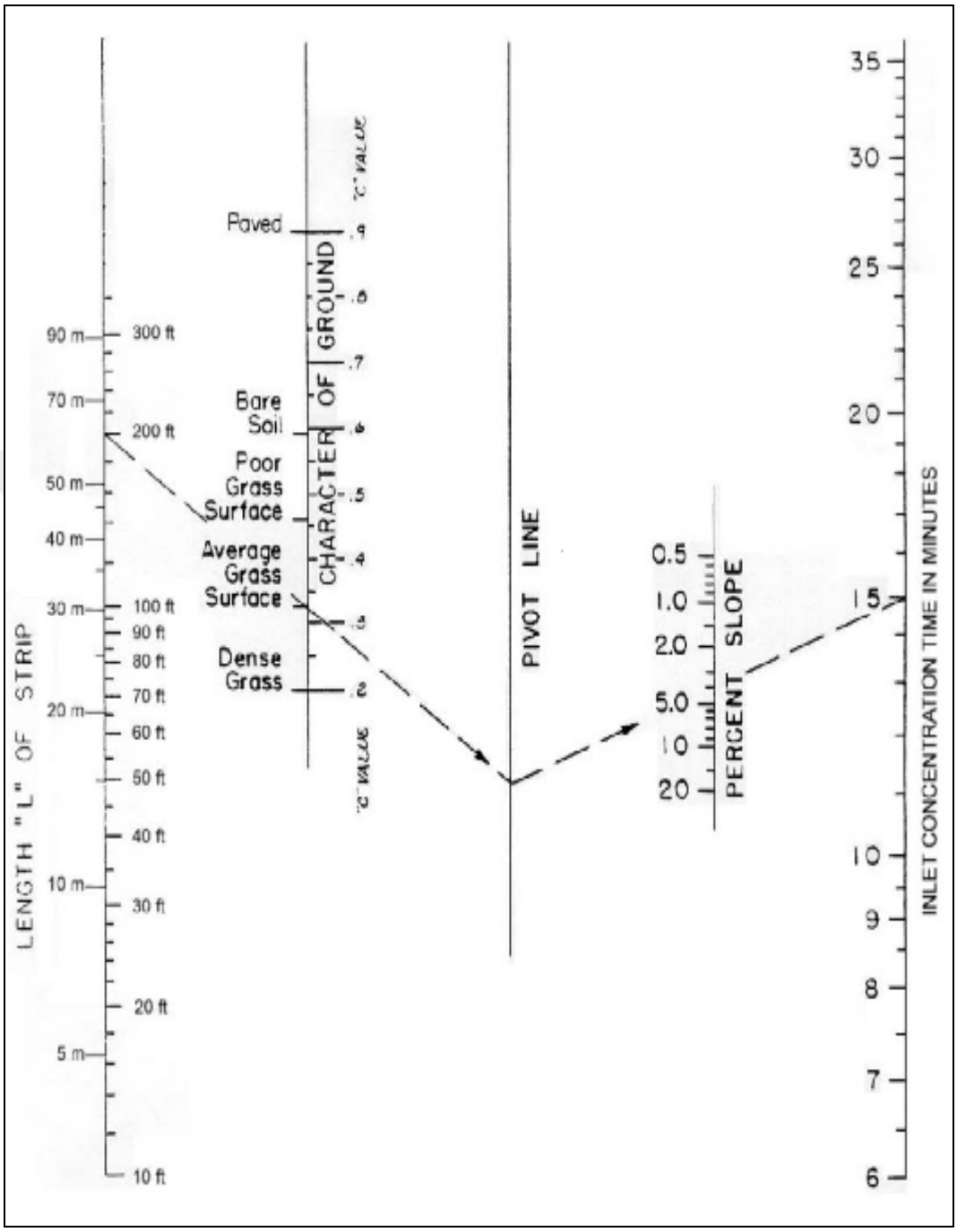


Figure 3. 1. Time of concentration (City of San Luis Obispo, 2003).

3.2.1.1.(4).(b). Shallow Concentrated Flow

Shallow concentrated flow occurs where overland flow converges to form small rills or gullies and swales. Shallow concentrated flow can exist in small, man-made drainage ditches (paved and unpaved) and in curb gutters. The recommended

maximum length for shallow concentrated flow is 300m (City of San Luis Obispo, 2003). Now determine the flow velocity by using equation 3.3 and the travel time by following equation 3.4 respectively.

$$V = k_u \times k \times Sp^{0.5} \quad (3.3.)$$

Where

V = velocity (m/s),

k_u = 1.0 in SI units,

k = interception coefficient (Table 3.2), and

Sp = Slope (percent).

Table 3. 2. Interception coefficient (Brown et al, 2009).

Land Cover/Flow Regime	k
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow); alluvial fans in western mountain regions	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow); small upland gullies	0.619

$$Tt = \frac{L}{60V} \quad (3.4.)$$

Where:

L = length of shallow concentrated flow (m), and

V = velocity (m/s) from Equation 3.3.

3.2.1.1.(4).(c). Channel or Pipe Flow

It occurs where flow converges in gullies, ditches, and natural or man-made water conveyances. Channel flow is assumed to exist in streams or where there is a well-defined channel cross-section. Use Manning's Equation for calculating channel flow. For the purposes of these calculations, it is acceptable to assume flow is bankful depth for open channels and pipeful flow for storm drain pipes. If these assumptions appear to result in over-conservative estimates, the flow rate obtained from the initial bankful Tc estimate can be used to recalculate channel velocity using the Manning equation. A separate computation should be made where channel or pipe conditions change (City of San Luis Obispo, 2003).

$$V = ku \times \frac{R^{\frac{2}{3}} \times S^{\frac{1}{2}}}{n} \quad (3.5.)$$

Where

V = average velocity (m/s),

R = hydraulic radius (m),

Ku = Unit conversion factor equal to 1,

S = slope of the grade line or channel slope (m/m), and

n = Manning's roughness coefficient (Table 3.3).

Table 3. 3. Typical Range of Manning's Coefficient for Channels and Pipes (Brown et al, 2009).

Conduit Material	Manning's n*
Closed Conduits	
Concrete pipe	0.010 - 0.015
CMP	0.011 - 0.037
Plastic pipe (smooth)	0.009 - 0.015
Plastic pipe (corrugated)	0.018 - 0.025
Pavement/gutter sections	0.012 - 0.016
Small Open Channels	
Concrete	0.011 - 0.015
Rubble or riprap	0.020 - 0.035
Vegetation	0.020 - 0.150
Bare Soil	0.016 - 0.025
Rock Cut	0.025 - 0.045
Natural channels (minor streams, top width at flood stage <30m	
Fairly regular section	0.025 - 0.050
Irregular section with pools	0.040 - 0.150

*Lower values are usually for well-constructed and maintained (smoother) pipes and channels.

3.2.1.1.(4).(d). Time of Concentration Calculation

$$T_c = T_{c\ overland} + T_{c\ shallow\ conc} + T_{c\ channel\ 1} + \ldots + T_{c\ channel\ n} \quad (3.6.)$$

3.2.1.1.(5). Determination of the Antecedent Moisture Regime

One of the most important factors in the Rational method is the antecedent moisture regime. This factor influences the amount of the peak discharge. This factor is described in the following:

The Rational Method has been revised for applying in watershed to include consideration of antecedent moisture conditions. Traditional application of the Rational Method does not account for soil saturation or base flow caused by previous storms or rainfall cells. However, large wintertime storms often occur in the series, so that peak runoff rates are greatly influenced by rainfall that may have occurred hours or days before the most intense rainfall cell. The antecedent moisture factors shown in Table 3.4 are an attempt to account for changes in soil infiltration capacity and creek base flow rates that occur during these very wet periods (City of San Luis Obispo, 2003).

Table 3. 4. Antecedent moisture factor (City of San Luis Obispo, 2003).

Recurrence Interval (years)	Antecedent Moisture Factor (C_a)
2 to 10	1.0
25	1.1
50	1.2
100	1.25

3.2.1.1.(6). Determination of the Rainfall Intensity

By computing the time of concentration, the rainfall intensity is obtained by the means of the I.D.F curves of you region.

3.2.1.2. Snyder Method

The second method, which is used, is Snyder Method. In this method, by computing some parameters, its unit hydrograph can be drawn. This method is described completely in the following:

This method which was developed in 1938 has been used extensively by the Corps of Engineers. In the Snyder method, two empirically defined terms, Ct and Cp, and the physiographic characteristics of the drainage basin are used to determine a D-

hour unit hydrograph. The entire time distribution of the unit hydrograph is not explicitly determined using this method, but seven points are given through which a smooth curve can be drawn. Certain key parameters of the unit hydrograph are evaluated and from these a characteristic unit hydrograph is constructed. The key parameters are the lag time, the unit hydrograph duration, the peak discharge, and the hydrograph time widths at 50 percent and 75 percent of the peak discharge. With these points a characteristic unit hydrograph is sketched. The volume of this hydrograph is then checked to ensure it equals 1 mm of runoff. If it does not, the ordinates are adjusted accordingly. A typical Snyder hydrograph is shown in figure 3.2 (Mccuen et al, 2002).

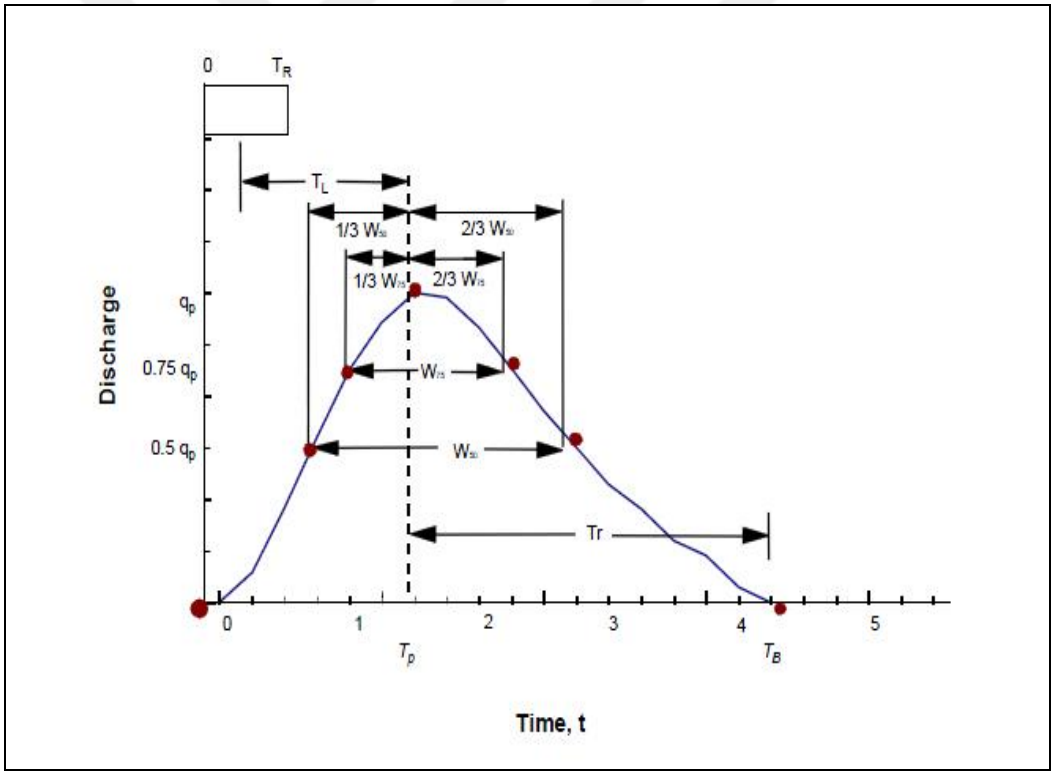


Figure 3. 2. D-hour unit hydrograph (Mccuen et al, 2002).

3.2.1.2.(1). Determination of the Data Collection and Physiographic Constants

Snyder developed his method using data for watersheds in the Appalachian Highlands and consequently the values which were derived for the constants C_t and

C_p are characteristic of this area of the country. However, the general method has been successfully applied throughout the country by appropriate modification of these empirical constants. Values for C_t and C_p need to be determined for the watershed under consideration. These can be obtained from other studies and textbooks or by analyzing unit hydrographs derived for gauged streams in the same general area. Another source of information is the Corps of Engineers, District Offices. C_t is a coefficient that represents the variation of unit hydrograph lag time with watershed slope and storage. In his Appalachian Highlands study, Snyder found C_t to vary from 1.8 to 2.2. Further studies have shown that extreme values of C_t vary from 0.4 in Southern California to 8.0 in the Eastern Gulf of Mexico. C_p is a coefficient that represents the variation of the unit hydrograph peak discharge with watershed slope, storage, lag time, and effective area. Values of C_p range between 0.4 and 0.94. In addition to these empirical coefficients, the watershed area, A, the length along the main channel from the outlet to the divide, L, and the length along the main channel to a point opposite the watershed centroid, L_{ca} need to be determined from available topographic maps (Mccuen et al, 2002).

3.2.1.2.(2). Determination of the Lag Time

The next step is to determine the lag time T_L, of the unit hydrograph. The lag time is the time from the centroid of the excess rainfall to the hydrograph peak. The following empirical equation is used to estimate the lag time (Mccuen et al, 2002).

$$T_L = \alpha \times C_t \times (L \times L_{ca})^{0.3} \qquad (3.7.)$$

Where

T_L= lag time (h),

C_t= empirical watershed coefficient which generally ranges from 1.8 to 2.2,

L = length along main channel from outlet to divide (km),

L_{ca}= length along main channel from outlet to a point opposite the watershed centroid (km), and

α = conversion constant 0.75 for SI units.

3.2.1.2.(3). Determination of the unit duration of the unit hydrograph

$$T_R = \frac{T_L}{5.5} \tag{3.8.}$$

A relationship has been developed to adjust the computed lag time for other unit durations. This is necessary because the equation above may result in inconvenient values of the unit duration (Mccuen et al, 2002).

$$T_{L(adj)} = T_L + 0.25 \times (T'_R - T_R) \tag{3.9.}$$

Where

$T_{L(adj)}$= adjusted lag time for the new duration (h),

T_L = original lag time as computed above (h),

T_R = original unit duration (h), and

T'_R = desired unit duration (h).

3.2.1.2.(4). Determination of the Peak Discharge

$$Q_p = \alpha \times \frac{C_p \times A}{T_{L(adj)}} \tag{3.10.}$$

Where

Q_p = unit peak discharge ($m^3/s/mm$),

C_p = empirical constant ranging from 0.5 to 0.7,

A = watershed area (km^2), and

α = conversion constant (0.275 for SI units).

3.2.1.2.(5). Determination of the Time Base of Unit Hydrograph

$$T_B = 3 + \frac{T_{L(adj)}}{8} \quad (3.11.)$$

Where

T_B = time of the synthetic unit hydrograph (days).

This relationship, while reasonable for larger watersheds, may not be applicable for smaller watersheds. A more realistic value for smaller watersheds is to use 3 to 5 times the time to peak as a base for the unit hydrograph. The time to peak is the time from the beginning of the rising limb of the hydrograph to the peak (Mccuen et al, 2002).

3.2.1.2.(6). Estimation of the W_{50} and W_{75}

The time widths of the unit hydrograph at discharges equal to 50 percent and 75 percent of the peak discharges, W_{50} and W_{75}, respectively, are approximated by the following equations (Mccuen et al, 2002).

$$W_{50} = \alpha_{50}\left(\frac{Q_p}{A}\right)^{-1.075} \quad (3.12.)$$

$$W_{75} = \alpha_{75}\left(\frac{Q_p}{A}\right)^{-1.075} \quad (3.13.)$$

Where

W_{50} = time interval between the rising and falling limbs at 50% of peak discharge (h),

W_{75} = time interval between the rising and falling limbs at 75% of peak discharge (h),

Q_p = unit peak discharge ($m^3/s/mm$),

A = watershed area (km^2),

α_{50} = unit conversion constant (0.18 in SI units), and

α_{75} = unit conversion constant (0.10 in SI units).

3.2.1.2.(7). Construction of the Unit Hydrograph

By using the obtain parameters the unit hydrograph can be drawn. This hydrograph is computed for the 1mm of the total runoff depth (Mccuen et al, 2002).

3.2.1.3. Synthetic unit hydrograph method (SCS or NRSC)

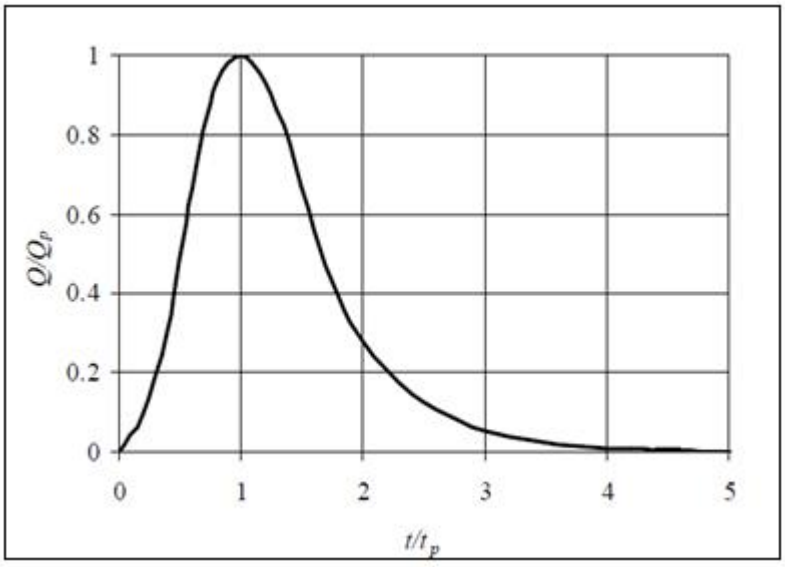
The third method, which is used, is Synthetic unit hydrograph method. In this method, by computing some parameters, its dimensionless unit hydrograph can be drawn. This method is described completely in the following:

The NRCS dimensionless unit hydrograph, tabulated in Table 3.5. and illustrated in figure 3.6, was developed based on data from a large number of watersheds (SCS, 1985). The dimensionless time and runoff ordinates can be dimensionalized by multiplying the corresponding values (i.e., t/ t_p or Q/Q_p) by time from the beginning of excess rainfall to the time of peak discharge, t_p, or the peak runoff, respectively (Nicklow et al, 2006).

Table 3. 5. NRCS dimensionless unit hydrograph (Nicklow et al, 2006).

t/t_p	Q/Q_p	t/t_p	Q/Q_p
0.0	0.000	1.4	0.780
0.1	0.030	1.5	0.680
0.2	0.100	1.6	0.560
0.3	0.190	1.8	0.390
0.4	0.310	2.0	0.280
0.5	0.470	2.2	0.207
0.6	0.660	2.4	0.147
0.7	0.820	2.6	0.107
0.8	0.930	2.8	0.077
0.9	0.990	3.0	0.055
1.0	1.000	3.5	0.025
1.1	0.990	4.0	0.011
1.2	0.930	4.5	0.005
1.3	0.860	5.0	0.000

Table 3. 6. NRCS dimensionless unit hydrograph (Nicklow et al, 2006).



Based on NRCS recommendations, time-to-peak discharge is a function of and highly sensitive to time of concentration. The relationship between these variables can be expressed as:

$$t_p = \frac{2}{3} t_c \quad (3.14.)$$

Where

t_c should be computed using one of the NRCS formulas one of this equations expressed below:

$$t_c = \frac{L}{60V} \quad (3.15.)$$

$$Q_p = \frac{K_p \cdot A}{t_p} \quad (3.16.)$$

Where

A = watershed area (km²),

K_p = constant equal to 2.08 in SI units, and

t_p = peak runoff in hour.

The time associated with the recession limb of the unit hydrograph, or time from peak discharge to the end of direct runoff, can be approximated multiplying t_p by 4.0. Note that the resulting synthetic unit hydrograph is applicable only for an effective duration of excess rainfall, t_d given as (SCS, 1985):

$$t_d = 0.133 t_c \quad (3.17.)$$

3.2.1.4. Determination of the Flood Discharge

The rivers with the stream gage stations, the flood discharge is obtained by means of the flood frequency analysis otherwise by using of the rain values and the unit hydrograph, the flood hydrograph and flood discharge is obtained.

3.2.2. Hydraulic Methods

3.2.2.1. Design of the Intake and Weir

Among the various patterns intakes, a ground intake is chosen because in high head schemes most river slopes are severe, hence for these types of intakes, it is suitable. In table 3.7, you can see its qualifications in terms of the slope and the use of design discharge.

Determining the size of the weir and intake depended on the site location and the flood discharge (Q_{flood}). So, the width of the river's base which consists of intake and the weir is necessary to obtain the length of the weir.

Table 3. 7. Ground intake qualification in terms of the slope and the use of design discharge (Ardüser and Karcheter, 2009)

	Ground intake
Q_{design}	up to 100% of Q_{river}
River slope I > 10%	very good
River slope 10% > I > 1%	good
River slope 1% > I > 0.01%	bad
River slope 0.01% > I > 0.001%	bad
River slope I < 0.001%	bad

Two dimensional and three dimensional views of ground intake are shown in follower:

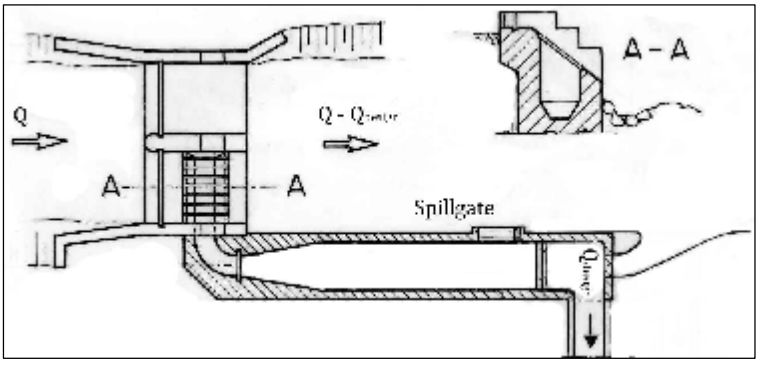


Figure 3.3.Two dimensional view of ground intake (Ardüser and Karcheter, 2009).

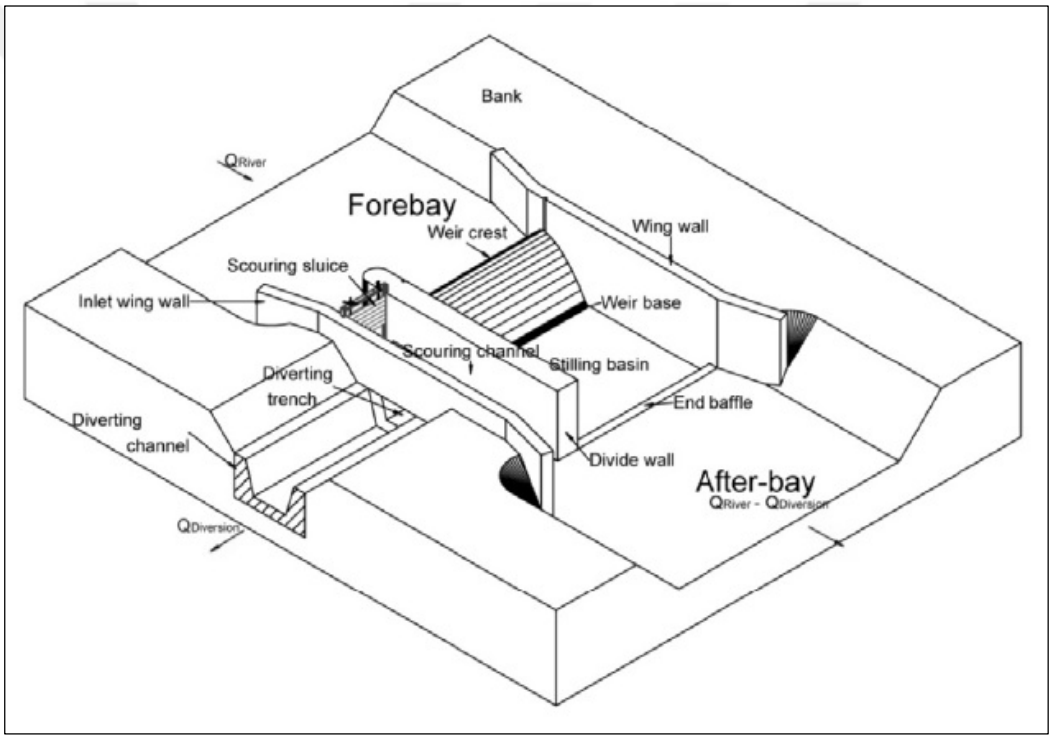


Figure 3.4.Three dimensional view of ground intake (Ardüser and Karcheter, 2009).

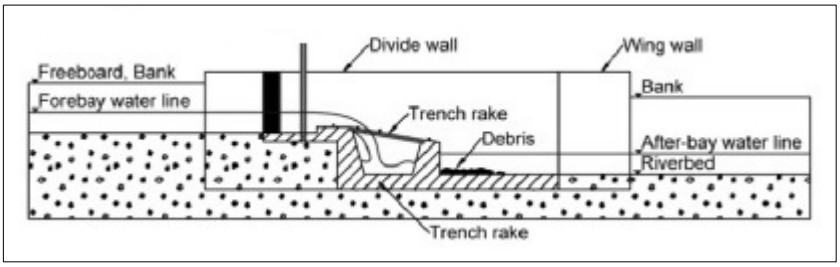


Figure 3. 5. Scourmig channel section (Ardüser and Karcheter, 2009).

3.2.2.2. Elements of Ground Intake

3.2.2.2.(1). Wing Walls

It is defined as the intakes side walls, improving flow conditions up-section. By joining abundant structures to an earth dike or the banks, the wing walls provide a longer path of percolation around the structure (Ardüser and Karcheter, 2009).

3.2.2.2.(2). Scouring Channel

It is defined as the portion of a river channel leading water to the undersluices and away from it to join the river downstream of the weir. Often built to spill debris and silt deposits away from the diverting channel inlet (Ardüser and Karcheter, 2009).

3.2.2.2.(3). Scouring Sluices Pocket

It is defined as the portion of river channel upstream of divide wall, inlet of the head regulator structure and the undersluices (Ardüser and Karcheter, 2009).

3.2.2.2.(4). Stilling Basin

It is defined as a structure below a spillway, chute or drop in which all or part of the energy dissipation occurs and into which kinetic energy is converted into turbulent energy (Ardüser and Karcheter, 2009).

3.2.2.2.(5). End Baffle

It is defined as a vertical, stepped slope or dentate wall constructed at the downstream end of a stilling basin (Ardüser and Karcheter, 2009).

3.2.2.2.(6). Freeboard

It is defined as the difference between the maximum flow line and the top of the bank or structure (Ardüser and Karcheter, 2009).

3.2.2.2.(7). Forebay

It is defined as the water immediately upstream of any structure. In some cases, this is a reservoir or pond at the head of a penstock (Ardüser and Karcheter, 2009).

3.2.2.2.(8). After-Bay or Tailrace

The term may be applied to a short stretch of stream immediately after a structure (Ardüser and Karcheter, 2009).

3.2.2.3. The Size of the Weir

Before designing is started, every parameter that is known and unknown should be specified. These parameters are listed below.

3.2.2.3.(1). Known Parameters of the Weir

1. Flood discharge (Q_{flood}) that is obtained from hydrologic assessments,
2. Design discharge (Q_{design}) that is obtained from hydrologic assessments,
3. Water level in the afterbay ($H_{afterbay}$ or H_B) in flood time that is obtained from the flow rating curve of rivers,

4. Thickness of wing walls ($W_{wing\,wall}$ = 0.25m) and divider wall ($W_{divider\,wall}$ = 0.5m), (Ardüser and Karcheter, 2009),
5. Height of the freeboard ($P_{weir\,freeboard}$) about 0.5m,
6. Widths of the scouring channel ($W_{scouring\,channel}$) about 1m, and
7. Length of river bed or gap between two banks (W_{total}) that is obtained from site location.

3.2.2.3.(2). Unknown Parameters of the Weir

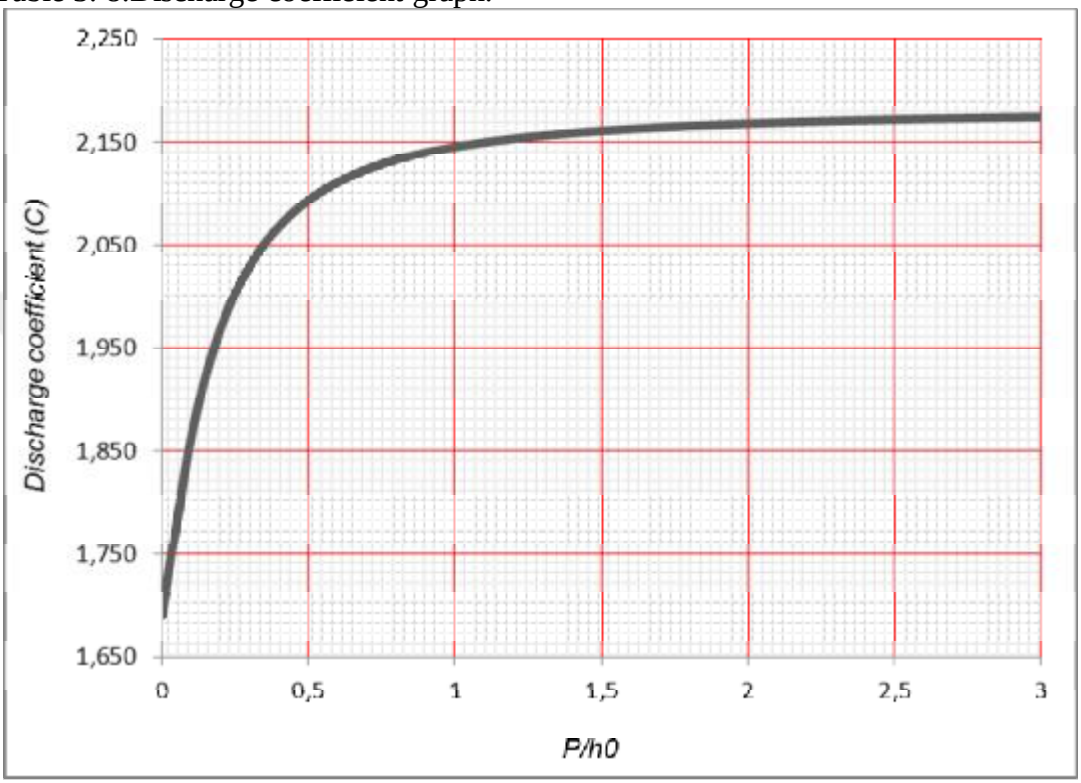
1. Flood head (h_0),
2. Length of weir (L_{weir}),
3. Height of weir (P_{weir}), and
4. Discharge coefficient (C).

After defining the known and unknown parameters, designing the weir is started. To compute the height of the weir, the discharge coefficient should be obtained. However, to choose it from its diagram, the height of the weir (P_{weir}) and the flood depth (h_0) are needed to obtain. In general, spillways are divided into two types: high-overflow spillways and low-overflow spillways.

A distinction is made between high-overflow spillways, which have a negligible velocity of approach, and low-overflow spillways, which have a significant velocity of approach that affects both the shape of the crest and the discharge coefficients. Discharge over a high-overflow spillway is also not affected by downstream submergence conditions. A spillway with a P/h_0 ratio of 1.33 or greater is considered a high overflow spillway, and the discharge coefficient no longer varies with P/h_0 (Wurbs and Stuart, 1991).

Discharge coefficient graph and table of the variation of c and P/h_0 are presented below:

Table 3. 8.Discharge coefficient graph.



To design the weir, follow these steps respectively.

Step 1: Determine the flood head (h_0).

Because of the change of the weir into a high-overflow spillway and disregard the velocity of approach, $H_e = H_B$ is resulted.

Step 2: Determine of the weir's width for initial trying and discharge coefficient (C).

In the first attempt, by assuming width of weir (L_{weir}) which is limited and obtained from 3.19 equation the ratio was accepted between 1.3 and 3 then the corresponding discharge ratio was obtained from table 3.9, also H_B was accepted instead of H_0. The passing discharge of the weir can be computed from presented formula:

$$Q = C \times L_{weir} \times (H_0)^{1.5} \quad (3.18.)$$

Table 3. 9.Variation of discharge coefficient and P/H_0

P/	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2	2.1
C	2.156	2.159	2.161	2.163	2.165	2.166	2.167	2.168	2.169
P/	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3
C	2.170	2.171	2.172	2.172	2.173	2.173	2.174	2.174	2.175

$$L_{weir} = W_{tot} - 2 \times W_{wing\,wall} - W_{scouring\,channel} - W_{divider\,wall} \quad (3.19.)$$

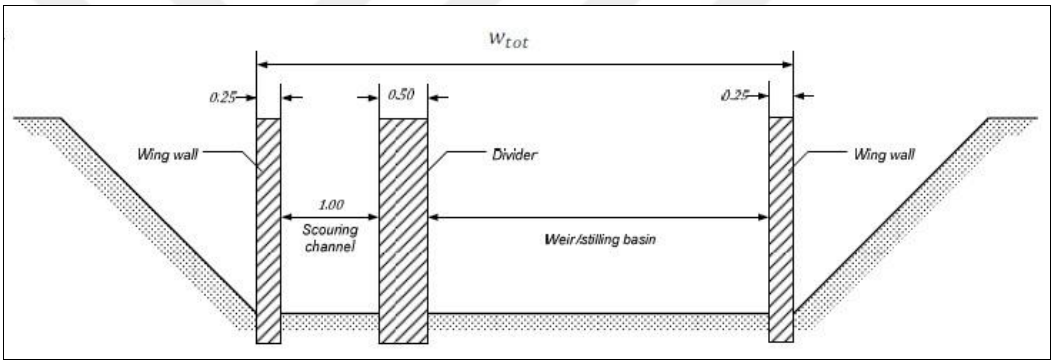


Figure 3.6. Determined weir width (Ardüser and Karcheter, 2009).

Step 3:.Compare calculated discharge ($Q_{calculate}$) with flood discharge (Q_{flood}).

If the calculated discharge flow is equal to Q_{flood}, L_{weir} will be accepted and the height of the weir will be obtained, otherwise, the C coefficient value will be increased until 2.175 then recalculate the discharge flow and repeat the previous step. After comparing the calculated discharge and flood discharge, if $Q < Q_{flood}$ again, the length of the weir (L_{weir}) will be increased until the calculated discharge flow is equal to flood discharge.

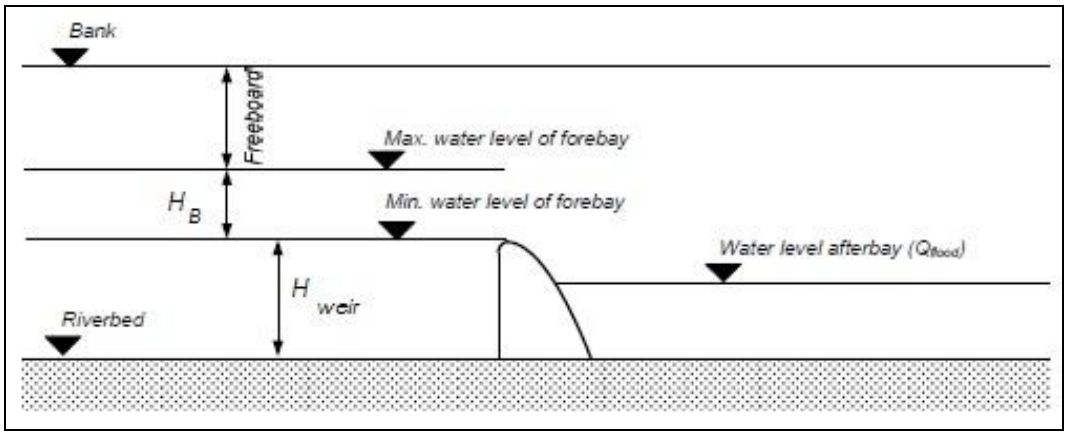


Figure 3. 7. Longitude weir section (Ardüser and Karcheter, 2009).

Step 4: Determination of the weir's height (P_{weir}) by obtaining the value of the C.

The height of the weir (P_{weir}) can be computed by multiplying corresponding C value by H_0.

3.2.2.3.(3). The Size of the Standard Weir Crest

The U.S. Army Corps of Engineers (USCE) developed a standard shape for a weir crest with a high discharge capacity μ and no prohibitive negative pressure on its lower slope. When regarding the peak point s, between a left and a right branch is differentiated. In mathematical terms, this means a curve in the up-water quadrant and a curve in the low-water quadrant. In order to shape the weir crest, one must find the appropriate values for a specific situation. The low water sloping branch of the weir is constructed by the function in relation of x according to (Ardüser and Karcheter, 2009):

$$y(x) = 0.5 \times \frac{x^{1.85}}{H_B^{0.85}} \qquad (3.20.)$$

The fillets are created by a radius, which is assumed by (Ardüser and Karcheter, 2009).

$$L_0 = 0.27 \times H_B \tag{3.21.}$$

$$L_1 = \left(2 \times P \times H_B^{0.85}\right)^{\frac{1}{0.85}} \tag{3.22.}$$

$$L_{total} = L_0 + L_1 \tag{3.23.}$$

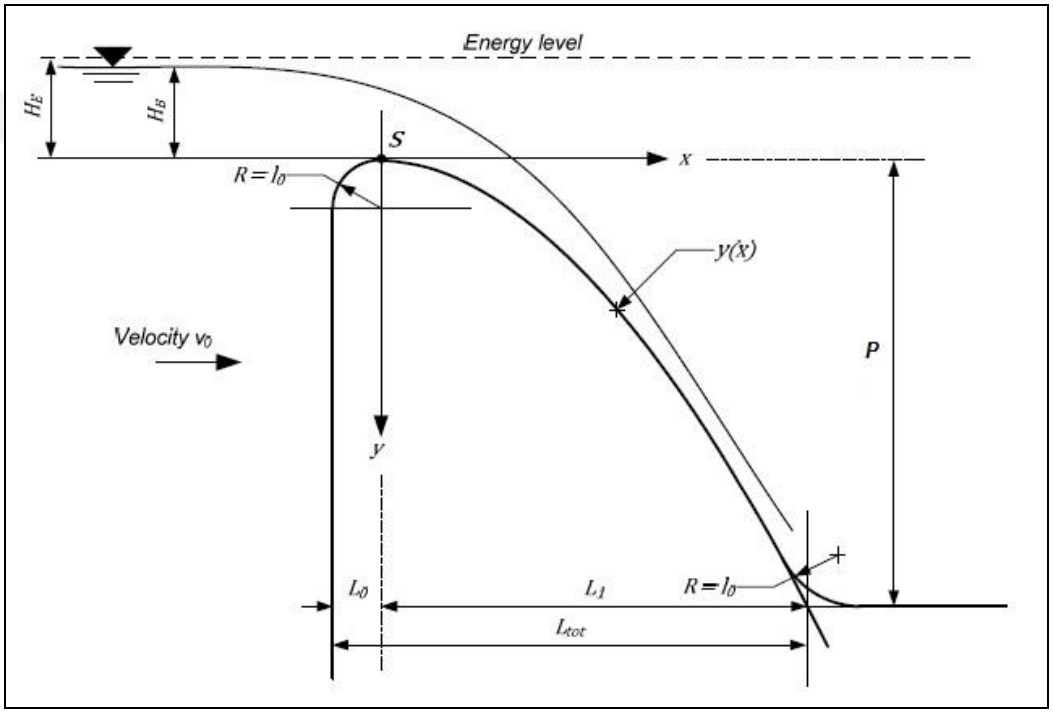


Figure 3. 8. Standard weir shape according to USCE (Ardüser and Karcheter, 2009).

3.2.2.4. The Size of the Trench Weir

To design the trench weir, its formula that is given below is used directly. In this step, design discharge is selected because in flood time extra, the water of the flood lead to scouring channel and overflow by the weir.

$$Q_{design} = \frac{2}{3}.c.\mu.b.L.\sqrt{2.g.h} \tag{3.24.}$$

3.2.2.4.(1). Known Parameters of the Trench Weir Design

1. Design discharge (Q_{design}),
2. Flow coefficient (μ) as in figure 5.10,
3. gravity acceleration (g),
4. Clearance between rake bars (a) that is obtained from available rake bars,
5. Distance between rake bars (d) that is obtained from available rake bars,
6. Rake slope (ß) that between (0° to 28°), (Ardüser and Karcheter, 2009), and
7. Design discharge water level in forebay from flow rating curve.

3.2.2.4.(2). Unknown Parameters of the Trench Weir Design

1. Rake width (b),
2. Rake length (L),
3. Critical water depth (H_{crit}), (~2/3 of water level in forebay), (Ardüser and Karcheter, 2009),
4. Orthogonal water depth at the beginning of the rake (h) as in figure 5.8, and
5. Coefficient of (c).

By defining the known and unknown parameters, designing the trench weir is started. All of the parameters would be specified except for the rake width (b) and the rake length (L). At the end of the calculation, the coefficient of b×L is obtained that an engineer or a user can choose the suitable length and width of the trench.

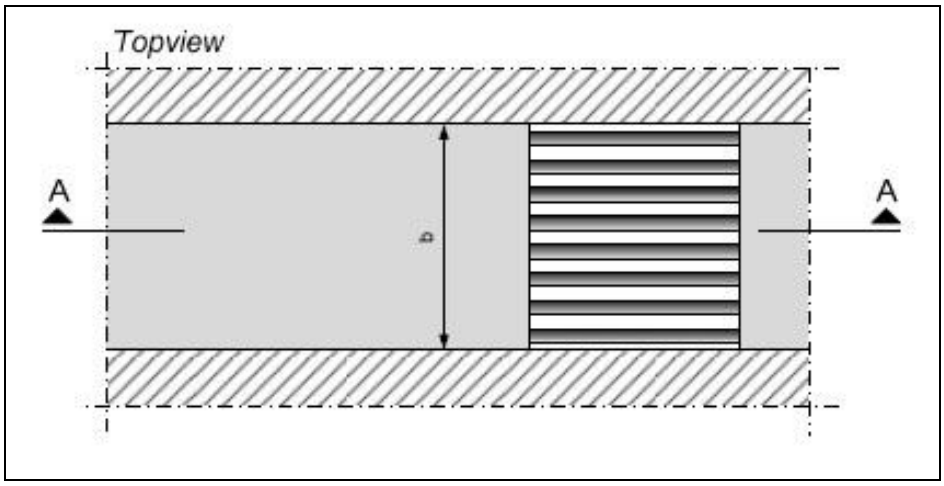


Figure 3. 9. Trench weir top view (Ardüser and Karcheter, 2009).

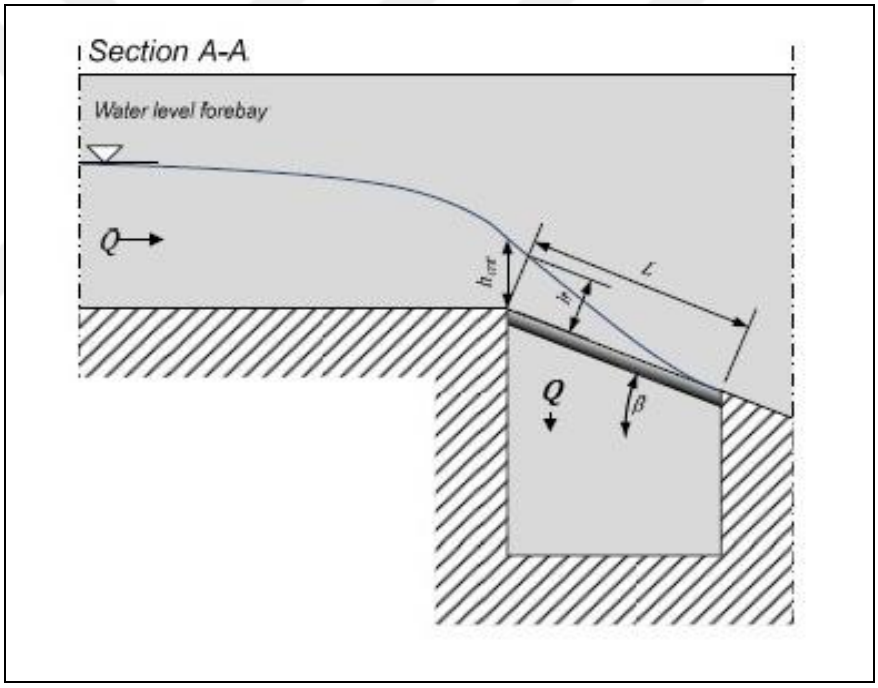


Figure 3. 10. Trench weir cross section (Ardüser and Karcheter, 2009).

By having Q_{design}, the water level forebay can be obtained via the flow rating curve then calculate H_{crit} and the orthogonal water's depth at the beginning of the rake (h) as flowing:

$$H_{crit} = \frac{2}{3} \times H_{forebay} \quad (3.25.)$$

$$h = H_{orit} \times k \tag{3.26.}$$

Using figure 3.9 and by having rake slope ß that present below, k coefficient can be found.

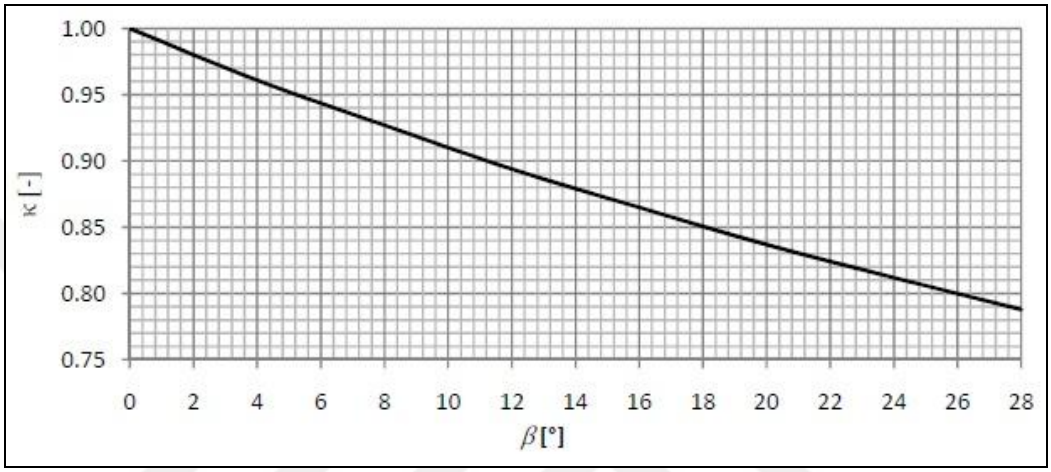


Figure 3. 11.Relative between rake slope ß and k coefficient (Ardüser and Karcheter, 2009).

C coefficient will be obtained by the flowing formula, a and b parameters in this formula and μ in equation 3.24 will be selected from figure 3.10.

$$c = 0.6 \times \frac{a}{b} \times (\cos ß)^{\frac{5}{2}} \tag{3.27.}$$

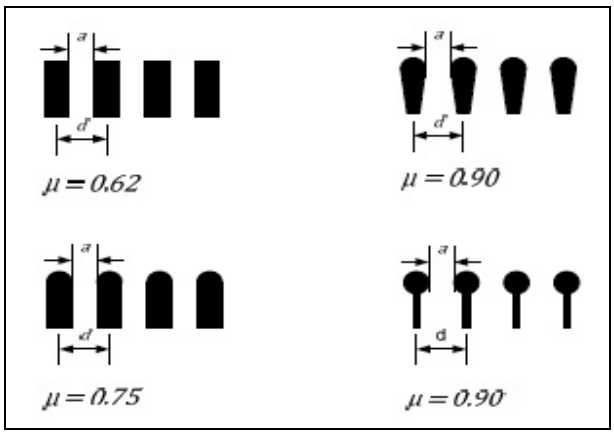


Figure 3. 12. Rake pattern (Ardüser and Karcheter, 2009).

3.2.2.5.The Design of the Stilling Basin

At first, finding the approach velocity on the top of the weir is attempted with using the flowing figure:

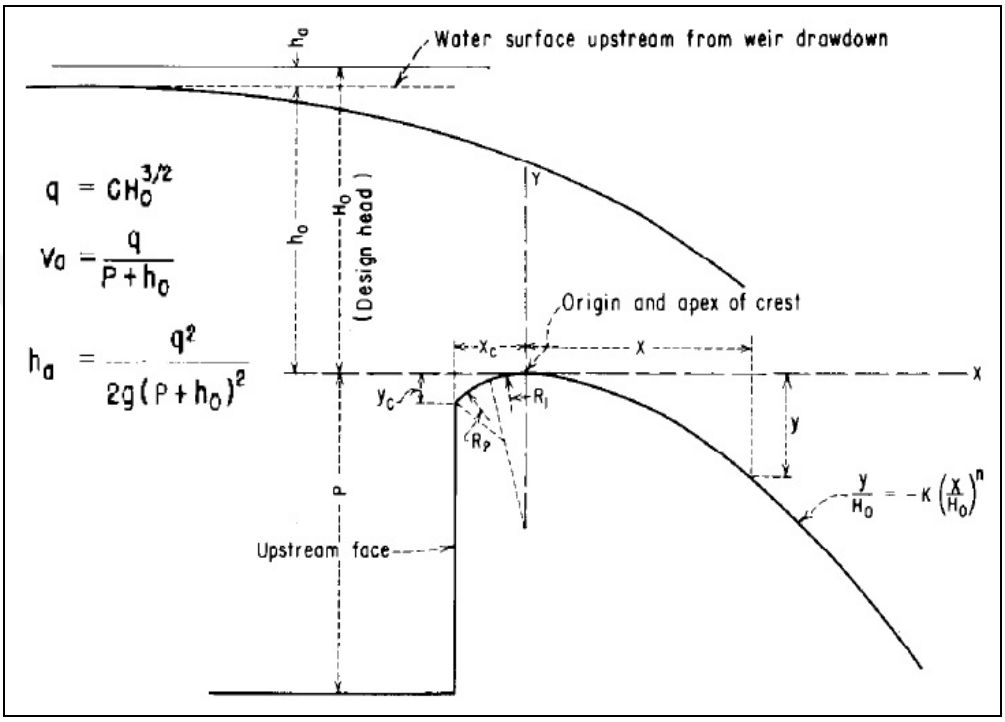


Figure 3. 13. Velocity approach on top of the weir. (United states department of the interior, 1960)

3.2.2.5.(1). Known Parameters of the Stilling Basin Design

1. Flood discharge (Q_{flood}),
2. Length of the weir (L_{weir}),
3. Flood head (h_0),
4. Height of weir (P_{weir}), and
5. Water level in afterbay ($H_{foerbay} = H_B = d_3 = h_0$).

3.2.2.5.(2). Unknown Parameters of the Stilling Basin Design

1. Supercritical flow depth (d_1),
2. Subcritical flow depth (d_2),
3. Downstream energy level (E_2),
4. Upstream energy level. (E_1),
5. Froude number (Fr) at section one, and
6. Approach velocity at section one (V_1).

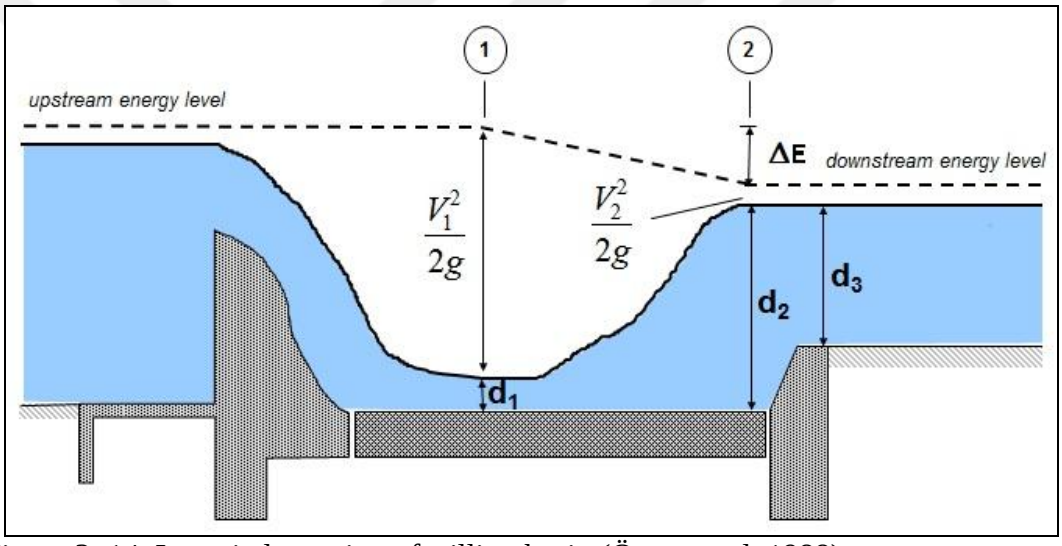


Figure 3. 14. Longtiude section of stilling basin (Özyar et al, 1988).

To design the stilling basin, the next steps are followed respectively.

Step 1: Determine the afterbay approach velocity (V_3) and downstream energy level.

$$V_3 = \frac{Q_{flood}}{L_{weir} \times d_3} \tag{3.28.}$$

$$E_2 = d_3 + \frac{{V_3}^2}{2 \times g} \tag{3.29.}$$

Step 2: Determine the Froude number at section one.

$$q = \frac{Q_{flood}}{L_{weir}} \quad (3.30.)$$

$$Fr = \frac{q^2}{g \times {d_1}^3} \quad (3.31.)$$

Step3: Determine d_2 as the flowing equation:

$$d_2 = \frac{d_1}{2} \times \sqrt{1 + 8 \times Fr^2} - 1 \quad (3.32.)$$

Step4: Determine the upstream energy level E_1 by using V_1.

$$E_1 = P_{weir} + h_0 + h_a \quad (3.33.)$$

$$H_a = \frac{q^2}{2 \times g \times (P + h_0)^2} \quad (3.34.)$$

Step 4: Determine d_1 by using the following iteration.

$$\Delta E = E_1 - E_2 = \frac{(d_2 - d_1)^3}{4 \times d_2 \times d_1} \quad (3.35.)$$

Step 5: According to d_1, the approach velocity in section one and the Froude number can be obtained via equation 3.31 and d_2 by equation 3.32.

$$V_1 = \frac{Q}{b \times d_1} \quad (3.36.)$$

Step 6: Classification stilling basin depended on Froude number.

- $Fr < 1.7$ No stilling basin required.

- $1.7 < Fr < 2.6$ Only use stilling basin and does not require energy dissipater blocks and end sill.

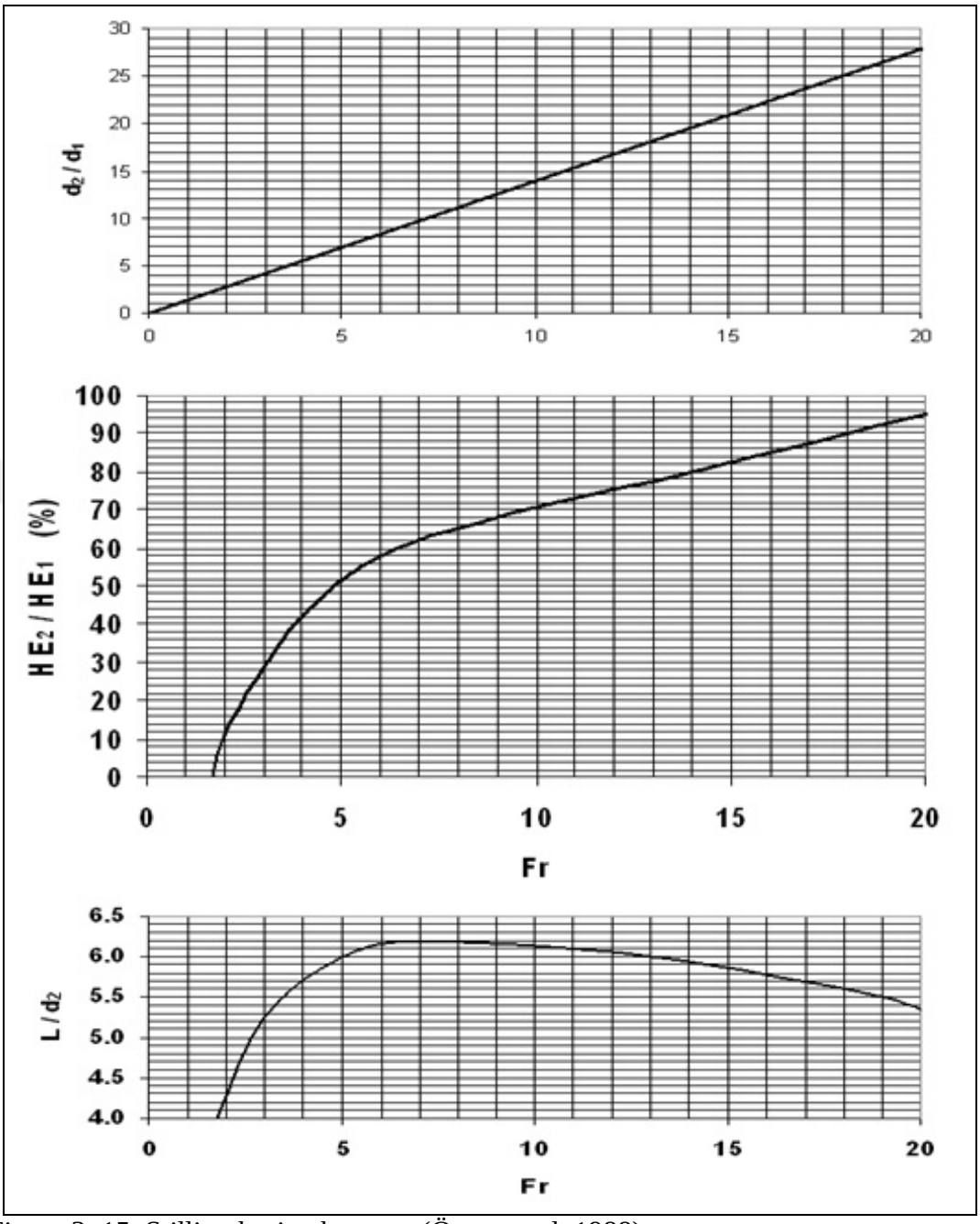


Figure 3. 15. Stilling basin chracters (Özyar et al, 1988).

- $2.6 < Fr < 4.5$ Stilling basin USBR type IV

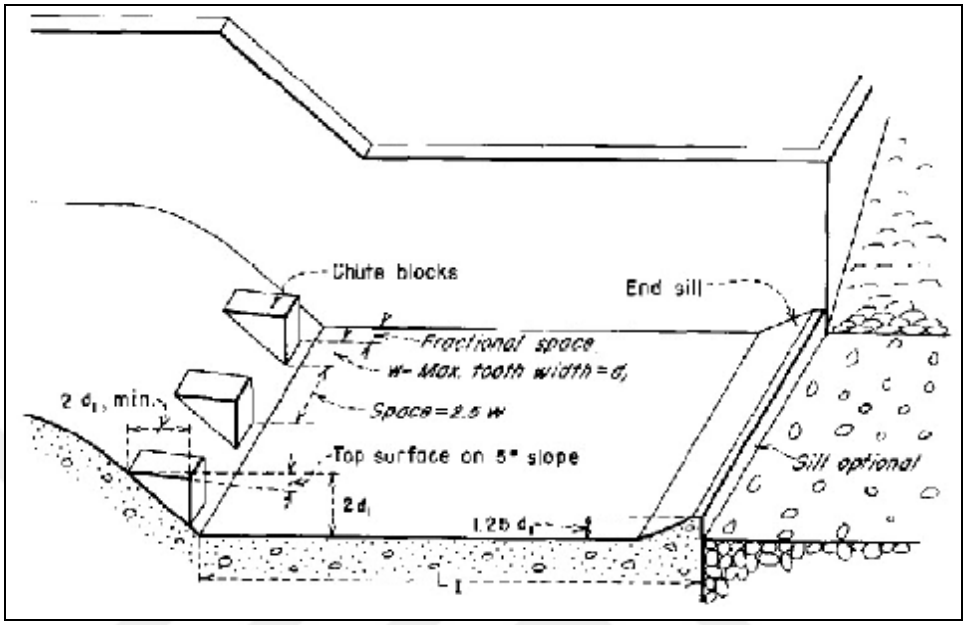


Figure 3. 16. Stilling Basin USBR type IV (United states department of the interior, 1960).

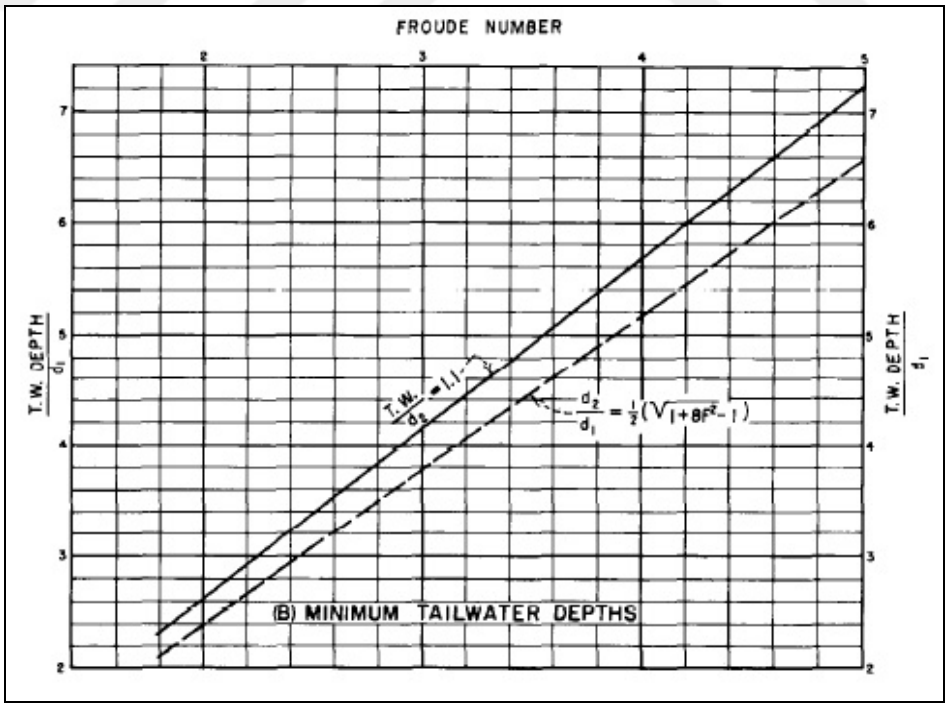


Figure 3. 17. Minimum tailwater depths in Stilling Basin USBR type IV (United states department of the interior, 1960).

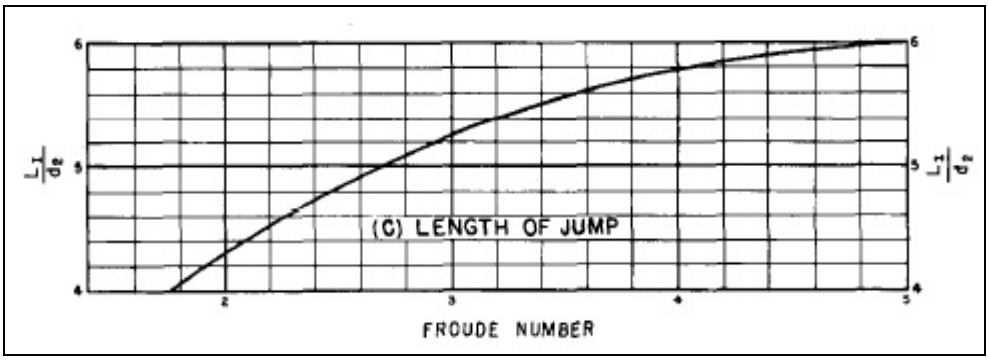


Figure 3. 18. Length of jump in stilling Basin USBR type IV (United states department of the interior, 1960).

- $Fr > 4.5$ and $V < 15$ (m/s) Stilling basin USBR type III

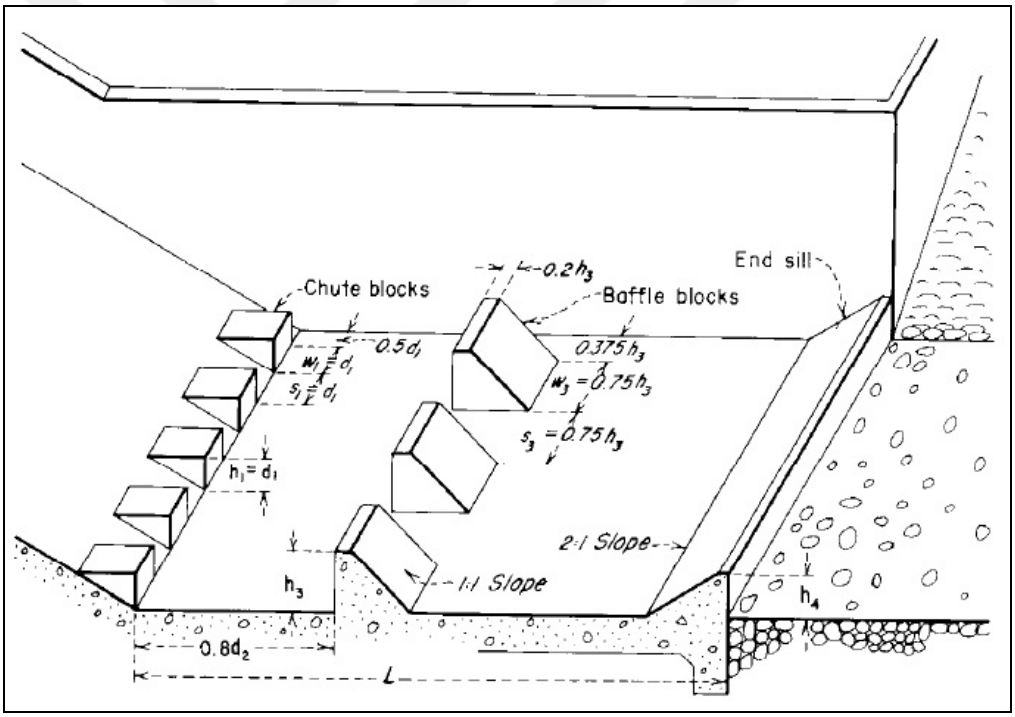


Figure 3. 19. Stilling Basin USBR type III (United states department of the interior, 1960).

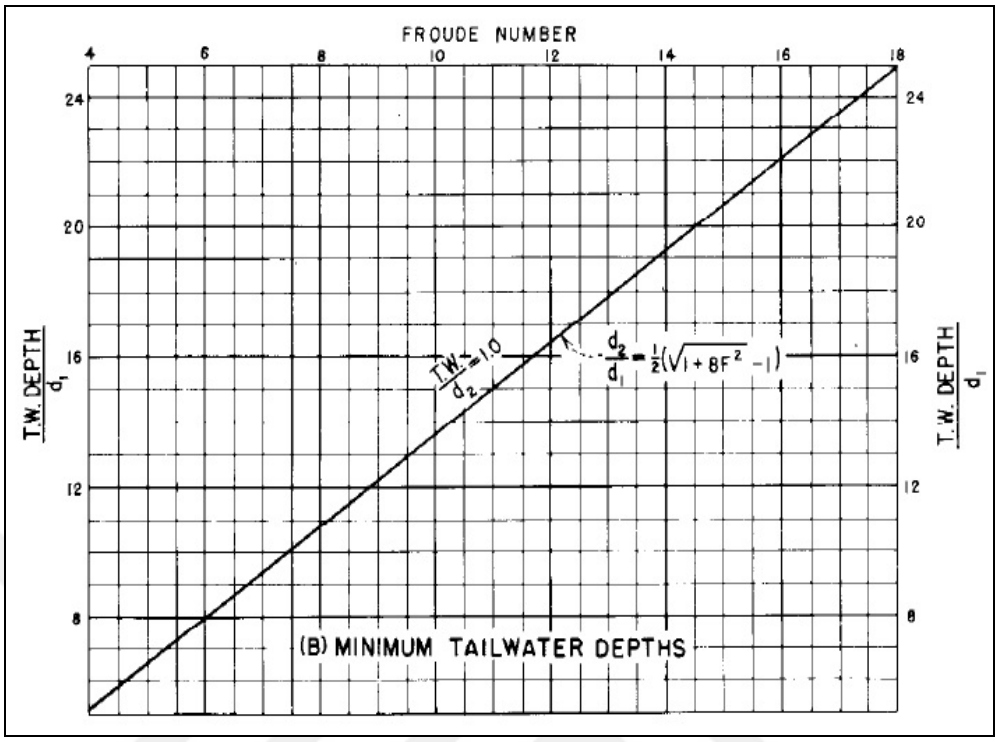


Figure 3. 20. Minimum tailwater depths in stilling Basin USBR type III (United states department of the interior, 1960).

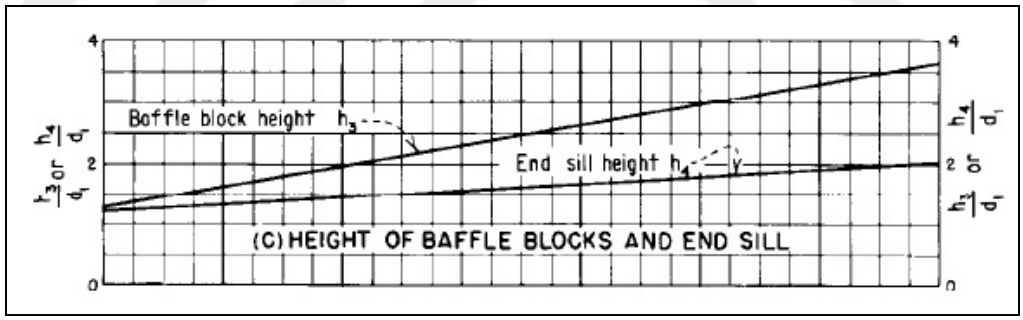


Figure 3. 21. Height of baffle blocks and end sill in stilling Basin USBR type III (United states department of the interior, 1960).

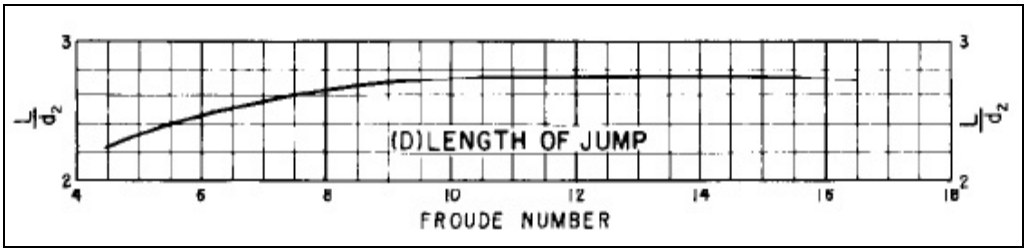


Figure 3. 22. Length of jump in stilling Basin USBR type III (United states department of the interior, 1960).

- $Fr > 4.5$ and $V > 15$ (m/s) Stilling basin USBR type II

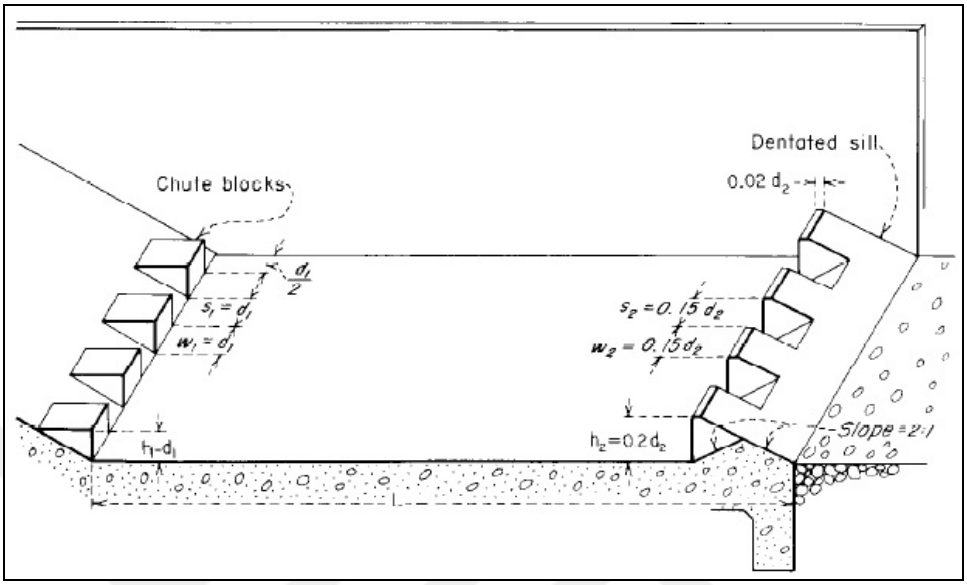


Figure 3. 23. Stilling Basin USBR type II (United states department of the interior, 1960).

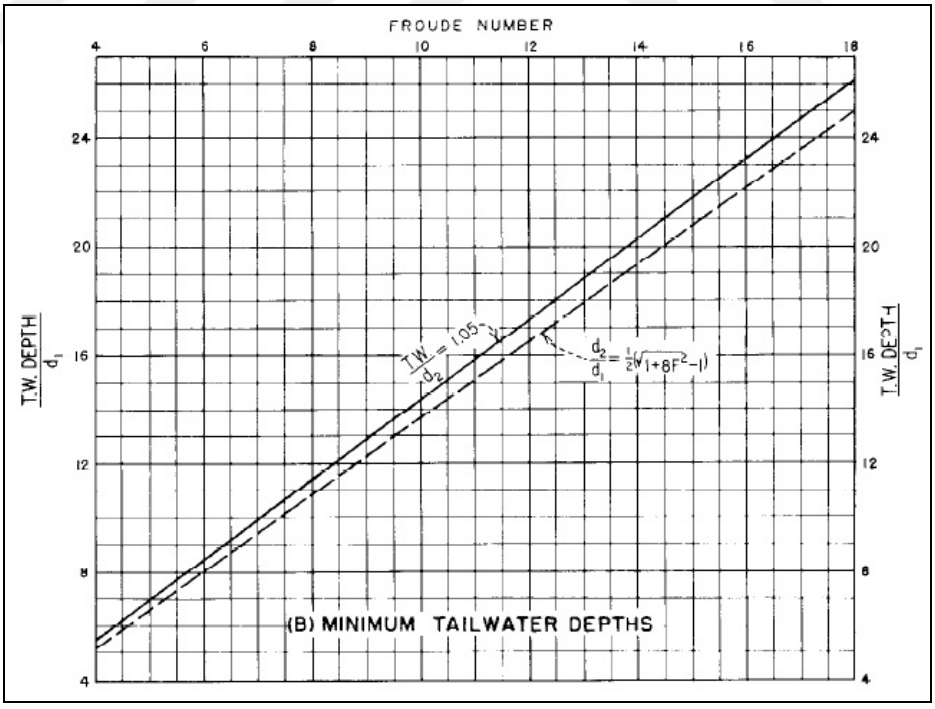


Figure 3. 24. Minimum tailwater depths in stilling Basin USBR type II (United states department of the interior, 1960).

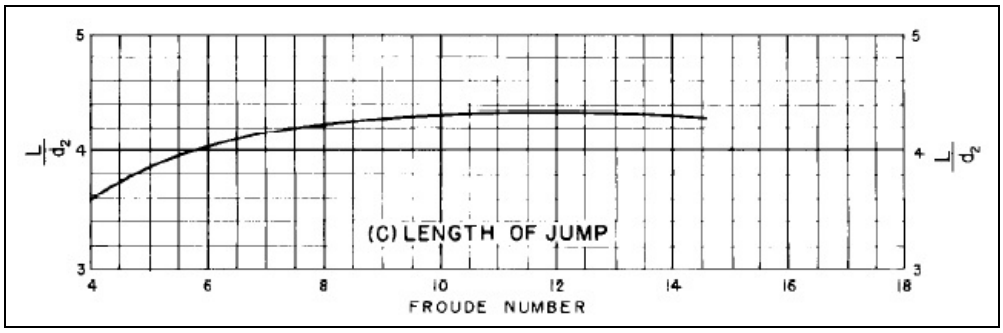


Figure 3. 25. Length of jump in stilling Basin USBR type II (United states department of the interior, 1960).

3.2.2.6. Designing the Headrace

To design the headrace channel, in order to convey the water flow from intake to forebay, the headrace or the leat can be used. Depending on the scheme layout of the project, the leat can be either used or not. For instance, in the short penstock scheme the leat was not utilize. To design the headrace and because of the uniform flow, the best hydraulic section can be employed.

3.2.2.6.(1). Known Parameters of the Headrace Design

1. Design discharge (Q_{design}),
2. Slope of the project (S).

3.2.2.6.(2). Unknown Parameters of the Headrace Design

1. Width of the channel ($b_{channel}$),
2. Height of the channel ($H_{channel}$),
3. Water level in the channel (y), and
4. Channel freeboard.

To design the headrace channel, follow these steps respectively.

Step 1: Selecting the channel section from the giving table:

Table 3. 10. Optimum profile for different channel sections (Penche, 1998).

Channel section	Area A	Wetted perimeter P	Hydraulic radius R	Top width T	Water depth d
Trapezoid: half hexagon	$1.73\ y^2$	3.46 y	0.500 y	2.31 y	0.750y
Rectangle : half square	$2\ y^2$	4 y	0.500 y	2 y	y
Triangle: half square	y^2	2.83 y	0.354 y	2 y	0.500y
Semicircle	$0.5\pi y2$	π y	0.500 y	2 y	$0.250\pi y$

Step 2: Determine the Manning's coefficient from the following table:

Table 3. 11. Manning coeffcient values (Penche, 1998).

Type of Channel	Manning's n
Excavated earth channels	
Clean	0.022
Gravelly	0.025
Weedy	0.030
Stony, cobbles (or natural streams)	0.035
Artificially lined channels	
Brass	0.011
Steel, smooth	0.012
Steel, painted	0.014
Steel, riveted	0.015
Cast iron	0.013
Concrete, well-finished	0.012
Concrete, unfinished	0.014
Planed wood	0.012
Clay tile	0.014
Brickwork	0.015
Asphalt	0.016
Corrugated metal	0.022
Rubble masonry	0.025

Step 3: Use the Manning's equation to calculate the water level (y) and channel width ($b_{channel}$) by iteration.

$$Q = \frac{A \times R^{\frac{2}{3}} \times S^{\frac{1}{2}}}{n} = \frac{A^{\frac{5}{3}} \times S^{\frac{1}{2}}}{n \times p^{\frac{2}{3}}} \quad (3.37.)$$

Step 4: Compare obtained velocity with the allowance velocity

In headrace the allowed velocity for preventing sediment deposits is 0.6 to 0.9m/s. (Maghsoudi and Kouchakzaedh, 2008).

Step 5: Determine the freeboard of the channel.

Actual dimensions have to include a certain freeboard (vertical distance between the designed water surface and the top of the channel bank) to prevent water level fluctuations overspilling the banks. Minimum freeboard for lined canals is about 10 cm, and for unlined canals, this should be about one third of the designed water depth with a minimum of fifteen centimeters (Penche, 1998).

3.2.2.7. Designing the Penstock

To design the penstock is required because of economic conditions, optimum diameter and thickness should be significant.

3.2.2.7.(1). Known Parameters of the Penstock Design

1. Design discharge (Q_{design}).

3.2.2.7.(2). Unknown Parameters of the Penstock Design

1. Penstock diameter.
2. Penstock thickness.

For designing the headrace channel, follow these steps respectively.

Step 1: Determine the optimal penstock diameter in mm and safety factor.

$$d = 720 \times Q_{design}^{0.5} \quad (3.38.)$$

This formula is suggested by Gordon and Penman (1979) for estimating the optimal diameter (mm) from the design discharge (Adugna, 2004).

Step 2: Determine the penstock thikness.

$$t_{min} = r_i \times \left(\sqrt{\frac{(\sigma_u - p_i)}{(\sigma_u + p_i)}} - 1\right) \quad (3.39.)$$

Where

r_i = The internal radius of the penstock,

σ_u = Ultimate tensile strength (Ardüser and Karcheter, 2009), and

p_i = Internal maximum assumed pressure at the regarded penstock section

$p_i = 1.1 \times$ hydrostatic pressure . for instance, if the head (Hp, refer to following figure) which from headtank to turbine is 25m, P=2.5×1.1=2.75 kgf/cm^2 (Penche, 1998).

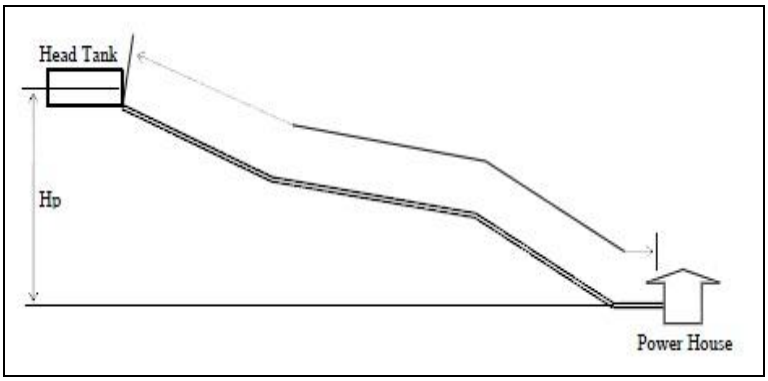


Figure 3. 26. Defined the Hp (Penche, 1998).

Table 3. 12. Ultimate tensile strength of materials (Ardüser and Karcheter, 2009).

Material	Ultimate tensile strength σ [N/m²]
Steel	$350 \cdot 10^6$
PVC	$28 \cdot 10^6$
HDPE/MDPE	$6\text{-}9 \cdot 10^6$
Ductile iron	$350 \cdot 10^6$
Cast iron	$140 \cdot 10^6$
Concrete	variable
GRP	$360 \cdot 10^6$

$$SF = \frac{t_{chosen}}{t_{min}} \quad (3.40.)$$

It is advised to consider a safety factor to also ensure that disturbances in the material or hidden erosion will not lead to failures at maximum stress. A safety factor of 2.5 up to 3.5 is adequate for most applications. (Ardüser and Karcheter, $SF =$

2009).

3.2.2.8.Design of the Settling Basin

The settling basins (or sand traps) are designed to allow sediment particles up to the size of 0.2 mm in diameter to settle. Specifically, all grains larger than 0.2 mm

must be removed before the water enters the turbine. The maker of the turbine offers detailed specifications as to the maximum diameter of particles, which may pass through the turbine safely without damaging the turbine blades. To reach the appropriate settling result, the flow velocity must to be reduced in order to minimize turbulence. Therefore, the cross-section of the basin should be widen gently until the flow is slow enough to let the particles sink. The flowing water is quite sensitive to the recesses and edges of the structure (Ardüser and Karcheter, 2009).

3.2.2.8.(1). Known Parameters of the Settling Basin Design

1. Electrical power output ($P_{turbine}$) from the maker of the turbine in Kw,
2. Usable gross head (h_{gross}),
3. Maximum particles diameter that turbine permits to pass without damaging the turbine blades (d),
4. Over flow efficiency (η).
5. Guaranty discharge for use of irrigation and pass through the forebay tank ($Q_{irregation}$), and
6. Channel width ($b_{channel}$).

3.2.2.8.(2). Unknown Parameters of the Settling Basin Design

1. Total discharge (Q_{total}) that should be available during turbine operation,
2. The maximum permitted flow velocity in the settling basin is determined with the approximation formula for critical velocity (V_{crit}), critical particle diameter (Ardüser and Karcheter, 2009),
3. The settling speed of the critical particle (w_0) in standing water (Ardüser and Karcheter, 2009), and
4. Dimension of the settling area.

To design the settling basin, follow these steps respectively.

Step 1: Determine the discharge flow in forebay and total discharge (Q_{total}).

$$Q_{turbine} = \frac{P_{turbine}}{\eta \times 9.81 \times h_{gross}} \quad (3.41.)$$

$$Q_{total} = Q_{turbine} + Q_{irregation} \quad (3.42.)$$

Step 2: Determine the V_{crit} by means of d so that it does not exceed from 0.6m/s (Ardüser and Karcheter, 2009).

$$V_{crit} = 0.44 \times \sqrt{d} \quad (3.43.)$$

Step 3: Determine w_0 by the equation 3.44. if the water temperature is 20° of Celsius otherwise use the following diagram.

$$w_0 = \frac{100}{9 \times d} \times \left(\sqrt{1 + 1.57 \times 10^2 \times d^3} - 1\right) \quad (3.44.)$$

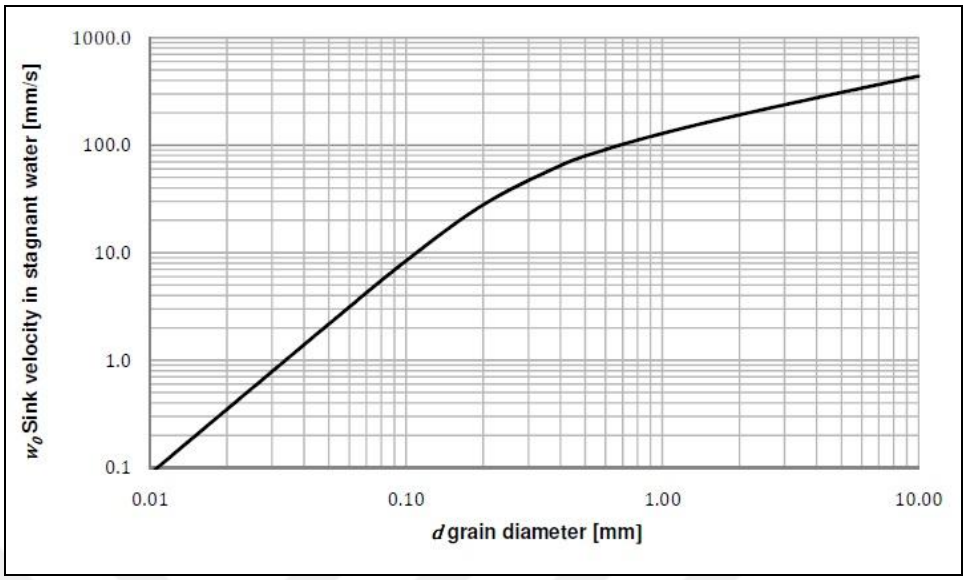


Figure 3. 27. Sink velocity according to the grain diameter (Ardüser and Karcheter, 2009).

Step 4: Determine the minimum depth and the width of the settling basin.

The width can be chosen based on the available space, but the width is usually between 2 to 15 times the widths of the channel. In case of a trapezoid channel, the average width is used. If the channel runs through soil, it is recommended to pave the last five meters of the channel before the basin with concrete, which will improve the flow into the settling basin (Ardüser and Karcheter, 2009).

$$W_{settling} = 2 \times W_{channel} \tag{3.45.}$$

$$H_{min} = \frac{Q}{W_{settling} \times V_{crit}} \tag{3.46.}$$

Step 5: Determine the height of the settling basin by forming a table which includes the range of the minimum to maximum height and its corresponding velocity of the moving water (w) and flow velocity of the water in settling basin (v) then calculate the length of the settling basin as following formula:

It is preferable to build a deeper basin if possible, because the length of the effective area of the settling area is then shortened. The maximum depth should not exceed the width of the basin by 1.25 (Ardüser and Karcheter, 2009).

$$v = \frac{Q_{total}}{2 \times H_{settling}} \tag{3.47.}$$

$$w = \frac{w_0}{1000} - \left(v \times \frac{0.132}{\sqrt{H_{settling}}}\right) \tag{3.48.}$$

$$L_{settling} = H_{settling} \times \frac{v}{w} \tag{3.49.}$$

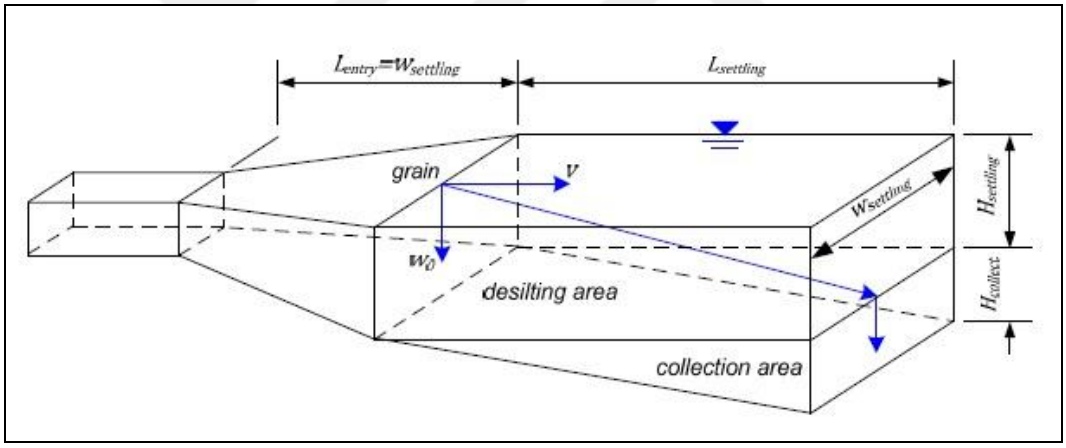


Figure 3. 28. System of a settling basin (Ardüser and Karcheter, 2009).

Step 6: Determine the volume of the collection area ($\tilde{V}_{capasity}$).

$$\tilde{V}_{cllection\ area\ capasity} = \frac{\tilde{V}_{settling\ basin}}{4} = \frac{W_{settling} \times H_{settling} \times L_{settling}}{4} \tag{3.50.}$$

This formula gives us a benchmark and should be regarded as the minimum volume. The size can be enlarged if large amounts of sand and silt are expected. This may be necessary if the river has a steep decline or if the ground in the catchment area is rich in clay. Exact data about the flushing interval can be evaluated after a few

months of service. The collecting area should never be overfilled. Otherwise, the basin cannot be completely emptied and the valve of the flush gate cannot be opened (Ardüser and Karcheter, 2009).

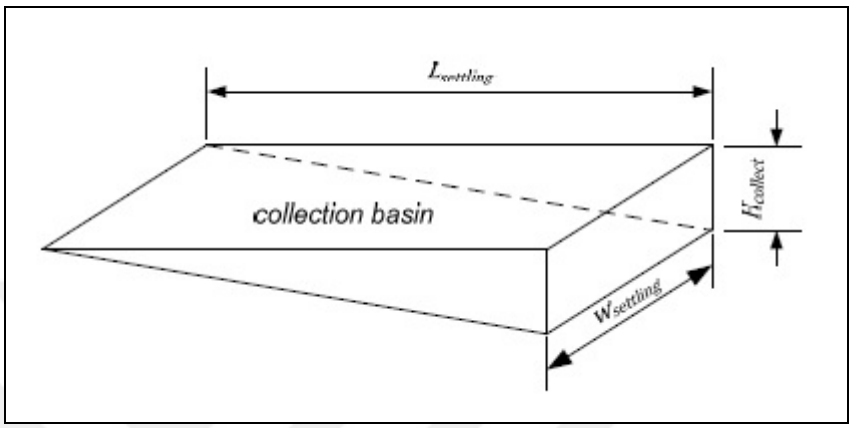


Figure 3. 29. Dimension of the collection area (Ardüser and Karcheter, 2009).

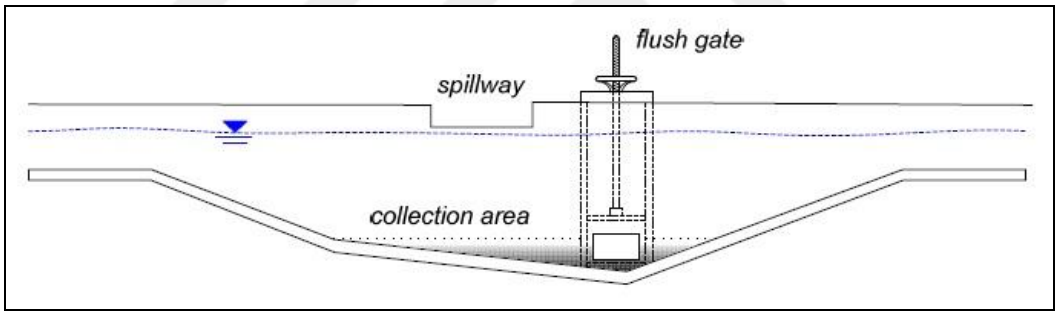


Figure 3. 30. Settling basin with flush gate and spillway (Ardüser and Karcheter, 2009).

An incline of 4:5 of the basin floor is most appropriate in order to guarantee the proper movement of the particles. This is suggested, but not always possible and compromises may be necessary. Where this is not possible, frequent manual maintenance with a broom may be necessary (Ardüser and Karcheter, 2009).

3.2.2.9. Design of the Forebay Tank

To design the forebay, follow these steps respectively.

Step 1: Determine the volume of the forebay tank.

$$\hat{V}_{\text{forebay tank}} = 75 \times Q_{\text{turbine}} \quad (3.51.)$$

$$\hat{V}_{\text{forebay tank}} = \hat{V}_{\text{balancing}} \quad (3.52.)$$

$$\hat{V}_{\text{balancing}} = \hat{V}_{\text{entry}} + \hat{V}_{\text{settling}} + \hat{V}_{\text{forebay tank}} \quad (3.53.)$$

$$L_{\text{entry}} = W_{\text{settling}} \quad (3.54.)$$

$$\hat{V}_{\text{entry}} = \frac{L_{\text{entry}}}{3} \times (A_1 + A_2 + \sqrt{A_1 \times A_2}) \quad (3.55.)$$

$$A_1 = W_{\text{settling}} \times H_{\text{settling}} \quad (3.56.)$$

$$A_2 = W_{\text{channel}} \times \text{water level in the channel} \quad (3.57.)$$

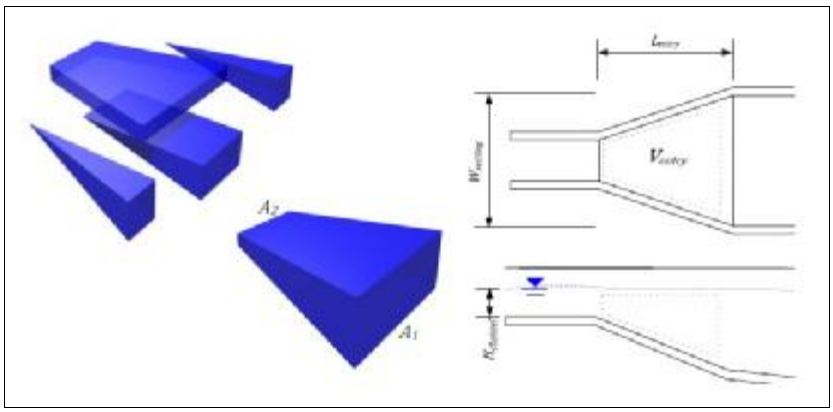


Figure 3. 31. Entery water volume (Ardüser and Karcheter, 2009).

$$\hat{V}_{\text{settling}} = L_{\text{settling}} \times W_{\text{settling}} \times H_{\text{settling}} \quad (3.58.)$$

$$\hat{V}_{\text{forebay chamber}} = \hat{V}_{\text{balancing}} - \hat{V}_{\text{entry}} - \hat{V}_{\text{settling}} \quad (3.59.)$$

Step 2: Determine the dimensions of the forebay chamber.

The penstock is mounted to the concrete body of the forebay tank four times its diameter below the water level. This measure is necessary to prevent air from being sucked into the penstock or to create a tornado vortex (Ardüser and Karcheter, 2009).

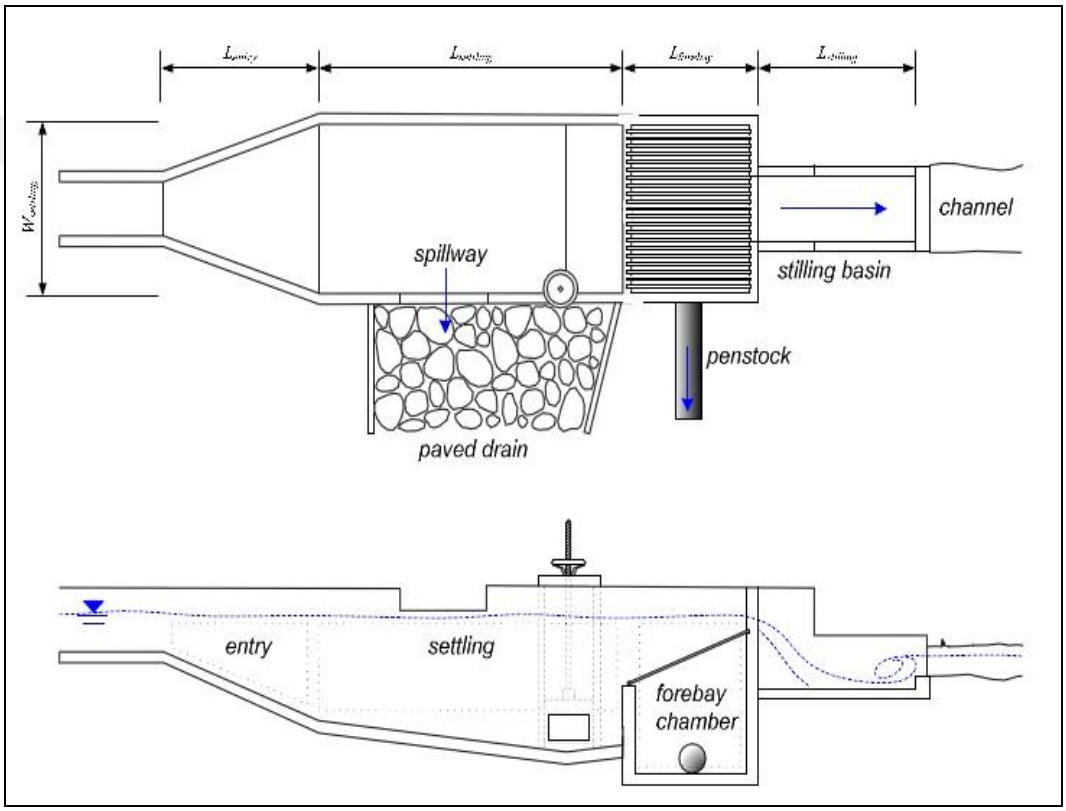


Figure 3. 32. Possible design of a forebay tank including settling area (Ardüser and Karcheter, 2009).

$$H_{\text{forebay chamber}} = 4 \times D_{\text{penstock}} + \frac{1}{2} \times D_{\text{penstock}} \quad (3.60.)$$

$$\hat{V}_{\text{dividing wall}} = [H_{\text{forebay chamber}} - (H_{\text{settling}} - H_{\text{dividing wall}})] \times W_{\text{settling}} \times W_{\text{dividing wall}} \quad (3.61.)$$

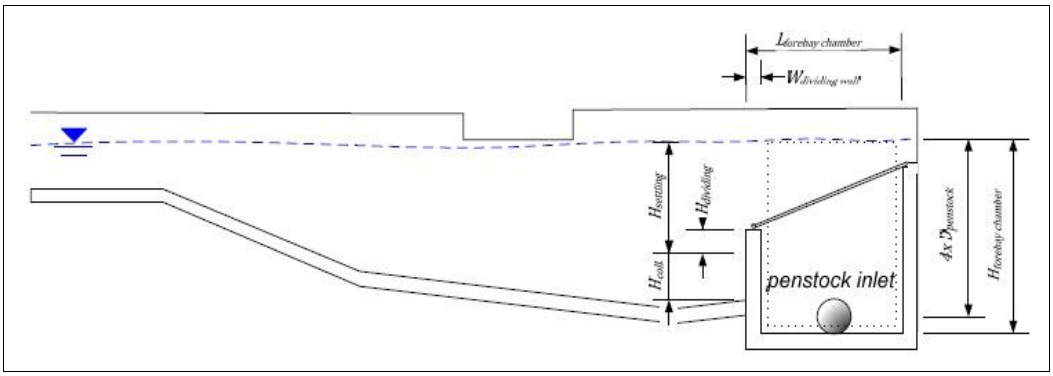


Figure 3. 33. Forebay chamber with dimensioning (Ardüser and Karcheter, 2009).

$$L_{\text{forebay chamber}} = \frac{\tilde{V}_{\text{forebay chamber}} + \tilde{V}_{\text{dividing wall}}}{W_{\text{settling}} \times H_{\text{forebay chamber}}} \quad (3.62.)$$

Step 3: Determine the weir of the forebay.

$$W_{\text{channel}} = W_{\text{weir}} \quad (3.63.)$$

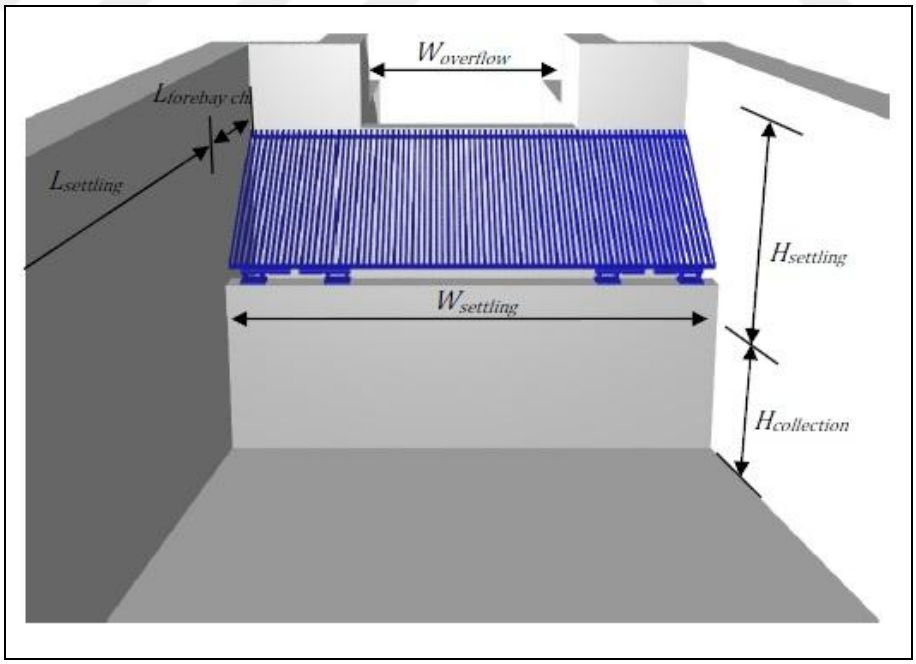


Figure 3. 34. The trash rack and weir of forebay (Ardüser and Karcheter, 2009).

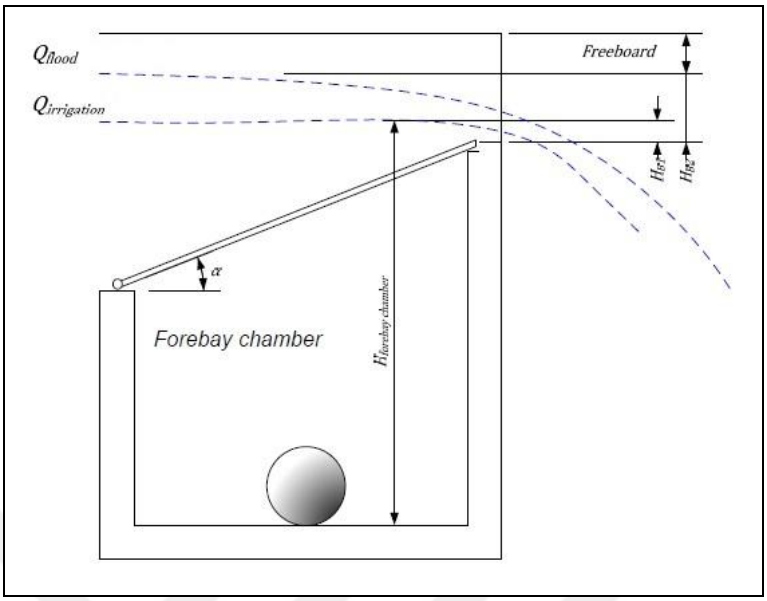


Figure 3. 35. Overflow situation in the channel (Ardüser and Karcheter, 2009).

$$H_{B1} = \frac{Q_{irrigation}^{\frac{2}{3}}}{(2.95 \times \mu \times b)^{\frac{2}{3}}} \quad (3.64.)$$

$$H_{B2} = \frac{Q_{flood}^{\frac{2}{3}}}{(2.95 \times \mu \times b)^{\frac{2}{3}}} \quad (3.65.)$$

Where

$$\mu = 0.5$$

$$H_{building} = H_{forebay\ chamber} + (H_{B2} - H_{B1}) + freeboard \quad (3.66.)$$

Where

freeboard=0.15m

Step 4: Determine the trash rack of the weir of the forebay.

The shapes and distances between the bars may vary. The design of the trash rack must be tailored to the available steel bars. Once a bar is chosen, the total

number of bars can be determined based on the below-mentioned maximum distances

- 0.5 times the nozzle diameter in case of Pelton turbine with fixed nozzle,
- 0.25 times the maximum clearance in a Pelton nozzle with needle valve
- 0.5 times the distance between runner blades for other turbine types. (Ardüser and Karcheter, 2009)

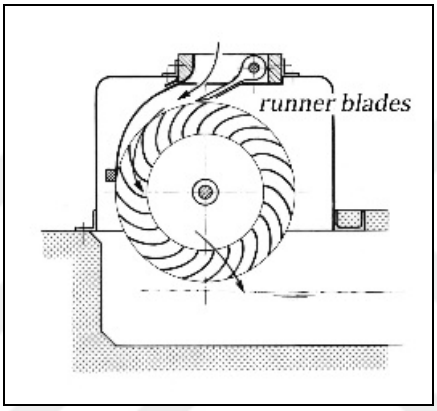


Figure 3. 36. Crossflow turbine (Ardüser and Karcheter, 2009).

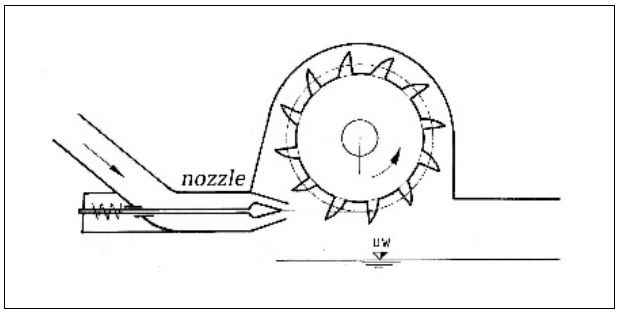


Figure 3. 37. Pelton turbine and nozzle with needle valve (Ardüser and Karcheter, 2009).

(3.67.)

$$\alpha = \tan^{-1}\left(\frac{H_{settlin} - H_{dividing} - H_{B\,1}}{L_{forebay\ chamber} - W_{dividing\ wall}}\right)$$

$$\text{distances between the trash rack bars} = \frac{b}{(a+s)} - 1 \qquad (3.68.)$$

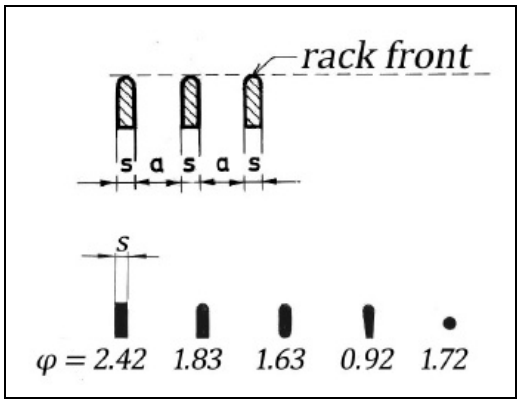


Figure 3.38. Longitudinal section of rack and its coefficient (Ardüser and Karcheter, 2009).

3.2.2.10. Computing of the Head Losses

Head losses majority occurs in conveying water from intake to forebay, in forebay tank and penstock. To obtain it,the head losses in each part of hydro plant and sum all of them to reach total head losses should be calculated.

Step 1: Determine the head losses due to headrace by means of hydraulic radius, Manning coefficient from section 3.2.2.6 and length of the headrace.

$$h_{f_{headrace}} = k_f \times \frac{V^2}{2 \times g} \tag{3.69.}$$

$$k_{f_{of\ channel}} = 2 \times g \times \frac{n^2 \times L_{of\ channell}}{R^{\frac{4}{3}}} \tag{3.70.}$$

Step 2: Determine the head losses due to the bar space of a trash rack.

The head losses can be computed by means of φ from figure 5.35 and know that average flow in forebay tank is (max 0.5 to 1 m/s), (Ardüser and Karcheter, 2009).

(3.71.)

$$h_{f_{trash\ rack}} = k_{f_{of\ trash\ rack}} \times \frac{v^2}{2 \times g}$$

$$k_{f_{of\ trash\ rack}} = \Psi \times \left(\frac{s}{a}\right)^{\frac{4}{3}} \times \sin \alpha \quad (3.72.)$$

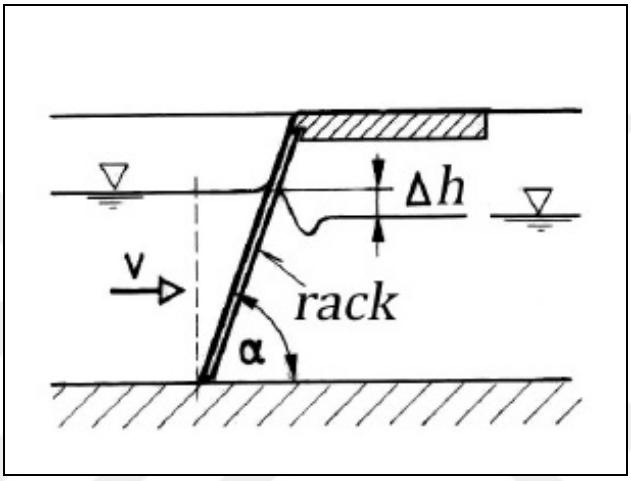


Figure 3. 39. Head losses in the trash rack (Ardüser and Karcheter, 2009).

Step 3: Determine the head losses due to penstock.

Head losses in penstock are divided to friction losses, local losses, bend or sudden contractions losses and valve losses while the majority of the head losses are because of friction losses.

At first determined the friction losses:

$$h_{f_{penstock}} = k_{f_{of\,penstock}} \times \frac{v^2 \times L}{2 \times g \times d} \quad (3.73.)$$

$$k_{f_{of\,penstock}} = 124.5 \times \frac{n^2}{d^{\frac{1}{2}}} \quad (3.74.)$$

Where

n = Coefficient of roughness for steel pipe n=0.12 and plastic pipe n=0.011 (Penche, 1998).

Determine the local losses:

$$h_{f_{local}} = k_{f_{of\,local}} \times \frac{v^2}{2 \times g} \quad (3.75.)$$

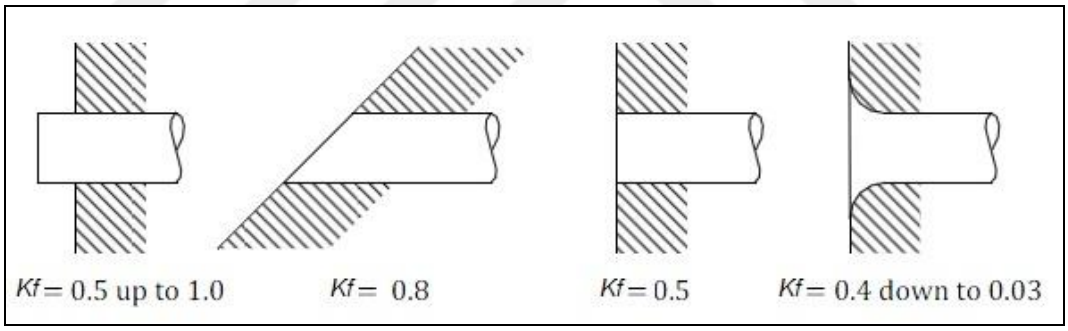
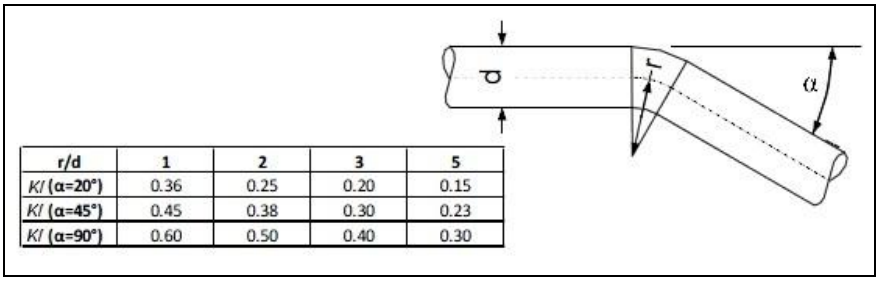


Figure 3. 40. Head loss coefficients for penstock intakes from a forebay tank (Ardüser and Karcheter, 2009).

Determine the bending losses:

$$h_{f_{of\ bend}} = k_{f_{of\ bend}} \times \frac{v^2}{2 \times g} \quad (3.76.)$$



r/d	1	2	3	5
Kf (α=20°)	0.36	0.25	0.20	0.15
Kf (α=45°)	0.45	0.38	0.30	0.23
Kf (α=90°)	0.60	0.50	0.40	0.30

Figure 3. 41. Head losses coefficient for bends and sudden contractions (Ardüser and Karcheter, 2009).

Determine the valve losses.

$$h_{f_{valve}} = k_{f_{of\ valve}} \times \frac{v^2}{2 \times g} \quad (3.77.)$$

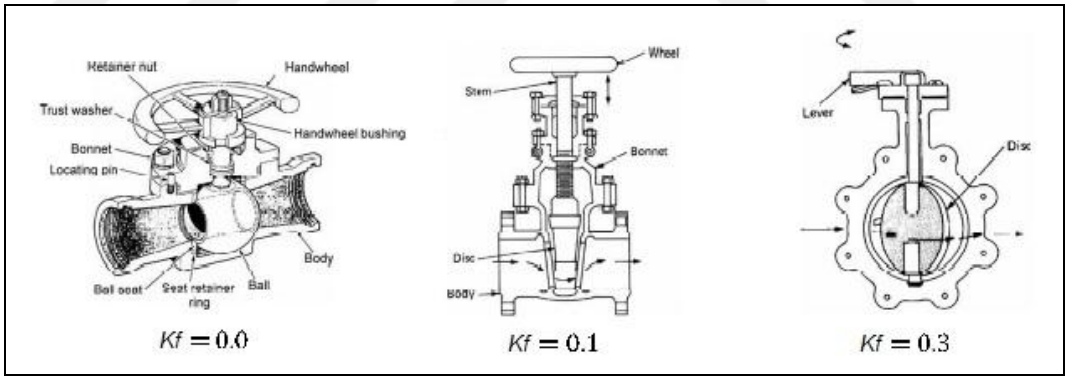


Figure 3. 42. Head losses coefficient for valves (Ardüser and Karcheter, 2009).

$$h_{f_{total}} = h_{f_{headrace}} + h_{f_{trash\ rack}} + h_{f_{penstock}} + h_{f_{local}} + h_{f_{bend}} + h_{f_{valve}} \quad (3.78.)$$

3.2.2.11. Determination of the Net Head

By calculating the total of losses head, the net head of the project can be computed as following:

$$H_{net} = h_{gross} - h_{f_{total}} \quad (3.79.)$$

4. DEVELOPED PROGRAM

The developed program using Matlab code consists of two sections. The first section computes hydrological parameters, especially flood discharge by using rational, Snyder and synthetic unit hydrograph methods. The second section designs all of the hydro power components.

4.1. The Hydrologic Section

After running the program in MATLAB software, the input values are entered respectively.

4.1.1. Computing the Flood Discharge by Rational Method

At first, the area of the project is entered in m^2. Because of the limitation of the rational method's area, if a value larger than 40 hectares or 400000 m^2 is entered the program needs to revise the area of the project or select another method to compute the unit peak discharge.

Calculation of the time of the concentration is divided into three parts that are mentioned previously. The user can select any one of them if required.

To obtain time of concentration of overland flow, the Seelye chart is used directly so that the user can choose it according to the project features and the length of the overland flow.

To calculate the time of concentration of the shallow flow, the program uses equation 3.3 and 3.4. Because of the limitation of the shallow flow's length, if a value larger than 300m is entered the program needs to revise the inputted value.

To calculate the time of concentration of the channel or pipe flow, the program uses the equation 3.5.

After entering all of the parameters, the total time of concentration will be compute according to the equation 3.6. This total time is utilizes to obtain the rainfall intensity by means of the I.D.F curves of the user's region.

To obtain the runoff coefficient, the program needs to enter the kind of cover of the project area and its corresponding c value. By entering of them, the program computes the total runoff coefficient and finally calculates the unit peak discharge of the rational method.

4.1.2. Computing the Flood Discharge by Snyder Method

To calculate unit peak discharge by Snyder method, the user enters the needed parameters by means of the presented tables and figures. Eventually, the program computes unit peak discharge and the required parameters to design of Snyder's curve.

4.1.3. Computing the Flood Discharge by SCS Method

To compute unit peak discharge by SCS method, the time of concentration should be obtained same as Rational method. At last, by using equation 3.16 the unit peak discharge is computed.

4.2. The Hydraulic Section

By running this section in MATLAB, the dimension of the hydropower and net head will be obtained.

4.2.1. Designing the Weir

To design the weir, the length of the river's base, the flood discharge and the water's depth at the afterbay are entered. Therefore, the program computes all of the required parameters that are necessary to design of the weir.

For designing of the trench of the weir by entering the angel of the rack and other parameters, the program computes a coefficient of the b×L so that the user can choose a suitable value for each of them.

4.2.2. Designing the Stilling Basin

In this section, the program obtains d_1, d_2, Froude number and type of the settling basin.

4.2.3. Designing the Headrace

To design the headrace, the number of the slopes is entered, and program according to the number of slopes designs a rectangular best hydraulic section.

4.2.4. Designing the Penstock

In this section, the program obtains the diameter and minimum thickness of the penstock.

4.2.5. Designing of the Settling Basin and the Forebay

To design the settling basin, the user enters the amount of the required energy in kWh and turbine's features. After this stage, the program shows two lists which consist of length and height of the settling basin. Therefore, the user selects the suitable values of them and enters. Finally, the program computes all of the settling basin and forebay's parameters.

4.2.6. Determining the Net Head

At last, the program computes the values of the head losses according to the type of the project's system, for instance in headrace, penstock, valve etc. therefore calculates the net head of the project.

5. RESULTS AND DISCUSSIONS

When designing a hydropower is wanted, in majority time it is expected that the design will be done in a specified turbine power. But, how? In developing this program, this question is tried to answer by inverse design from turbine to settling basin. In this condition, a gross head is used at the end of the calculations. Now, the program is explained by using an example.

5.1. Hydrologic Section of the Algorithm

After running the program, any value that is required for the program should be entered. At first, the rational method is selected by entering 1 then enter the project area in m^2. If time of concentration for overland flow is wanted to calculate, we need to insert 1 then by means of the Seelye chart, then time of concentration for overland flow in minutes can be selected and entered its value in the program.

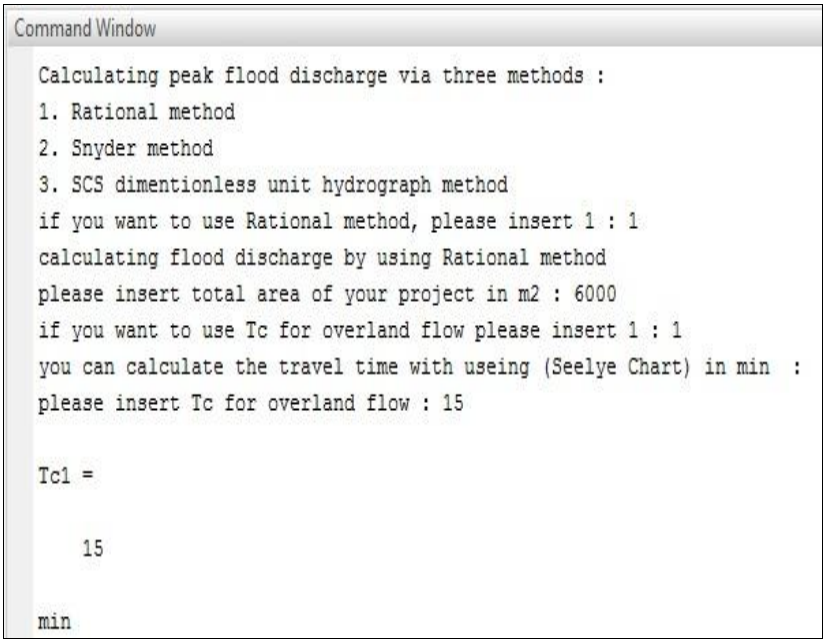


Figure 5. 1. Using time of the concentration of the overland flow.

Because of the project area in the Rational method must not exceed from 40 hectares or 400000m^2, if a value larger than 40 hectares is entered, the program needs to revise the area or select another method to compute unit peak discharge.

```
Calculating peak flood discharge via three methods :
1. Rational method
2. Snyder method
3. SCS dimentionless unit hydrograph method
if you want to use Rational method, please insert 1 : 1
calculating flood discharge by using Rational method
please insert total area of your project in m2 : 450000
the rational method is not valid for A>400000 m2
you can revise inserted area or use one of Snyder method or SCS dimentionless unit hydrograph
```

Figure 5. 2. Using the rational method by means of the program.

If time of concentration for shallow flow is wanted to calculate, insert 1. Since the length of shallow flow must not exceed from 300m, if a value larger than 300m is inserted, the program needs to revise the flow length. The time of concentration of the shallow flow calculates via equation 3.3 and 3.4.

```
please insert L (Flow length) in m : 400
Maximum shallow flow length is 300m, please revise inserted length
please insert L (Flow length) in m equal or less than 300m :
```

Figure 5. 3. Using time of the concentration of the shallow flow.

To calculate the time of concentration of the channel or pipe follows, enter 3, then enter hydraulic radius and channel slope respectively. Therefore, obtain Manning roughness coefficient (n) from the presenting table by program and enter its value.

```
if you want to use Tc for channel, please insert 3 : 3
please insert R (hydraulic radius) in m : 0.5
please insert s (slope of the grade line or channel slope percent) in m/m : 3
please insert n (Manning's roughness coefficient) from presented table : 0.011

Tc3 =

    0.0504

min
Tc that we calculated is :

Tc =

   22.7868

min
```

Figure 5. 4. Using time of the concentration of the channel or pipe flow.

After computing the total concentration time by using equation 3.6, it can be applied to find the runoff intensity from the user's region I.D.F curves. Then insert antecedent moisture factor value (Ca) from presenting table by the program.

If the cover of the area formed by several kinds, a number of them is inserted then the program will ask area and Runoff coefficient for each kind of cover respectively. The program computes the total runoff coefficient by using the 3.2 equation.

Finally, the program calculates the unit peak discharge of the rational method by means of the equation 3.1.

```
please insert i in mm/hr via obtained TC from I-D-F curve of your region : 34
please insert ca (antecedent moisture factor) from presented table : 1
please insert kind of cover: 2
please insert Area of covers in m2 ,respectively : 4000
please insert Runoff Coefficient of this cover from presented table : 0.45
please insert Area of covers in m2 ,respectively : 2000
please insert Runoff Coefficient of this cover from presented table : 0.3

Q =

    0.0013

m3/s
```

Figure 5. 5. Calculating the unit peak discharge of the rational method.

To compute the unit peak discharge by using Snyder method, should be entered 1. Then the empirical watershed coefficient and the empirical constant in a

given range that are depend on user and his watershed features are entered. Therefore, Insert (L) and (Lca) from topography maps in km. For the last step, the desired unit duration in hour is entered.

Finally, the program calculates the unit peak discharge of the Snyder method by means of the equation 3.10.

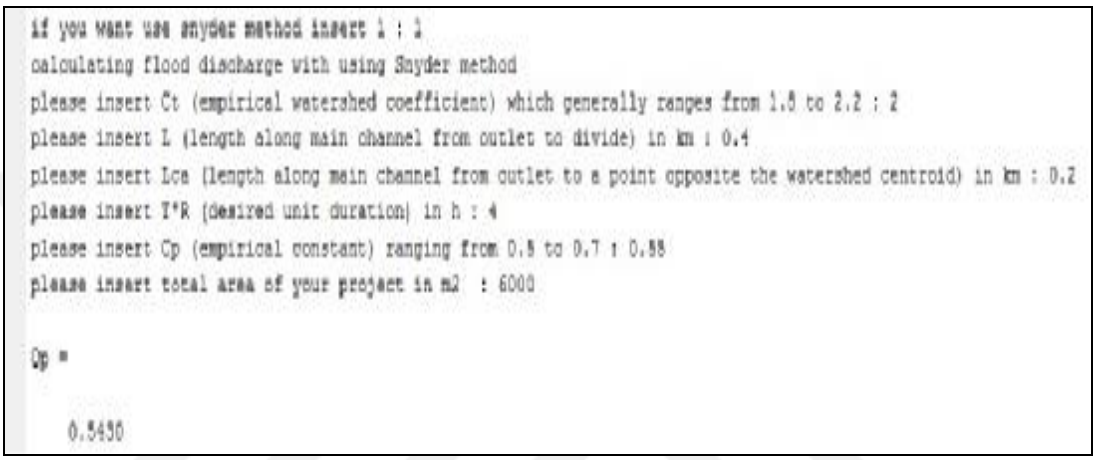


Figure 5. 6. Calculating the unit peak discharge of the Snyder method.

By obtaining components of the Snyder hydrograph, it can be drawn easily.

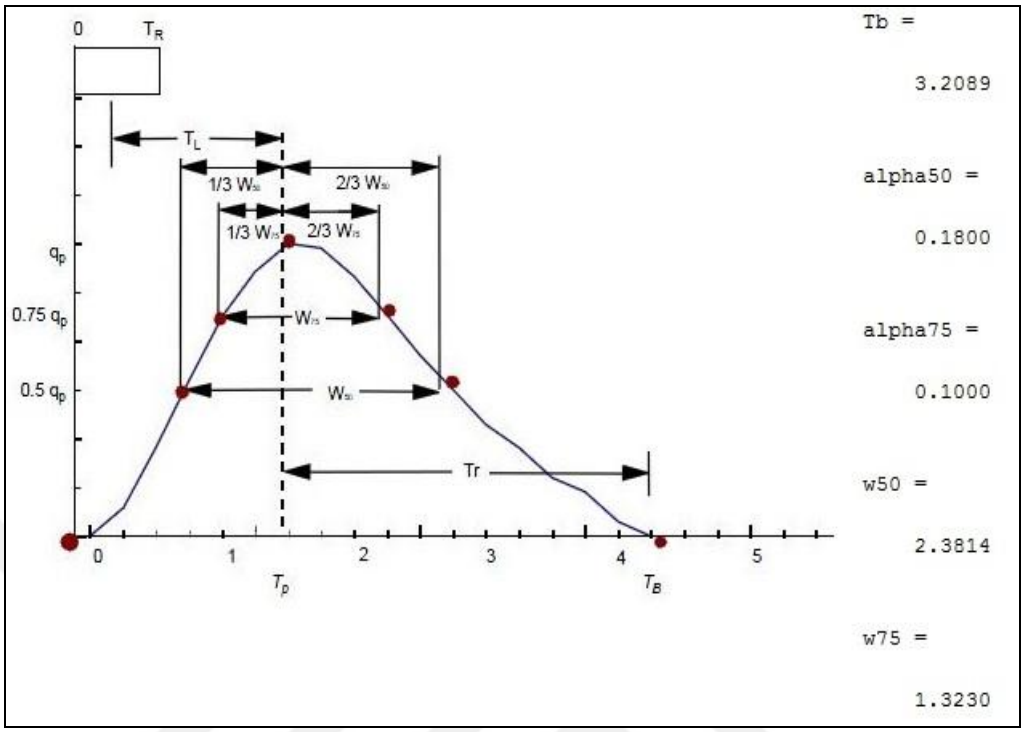


Figure 5. 7. Components of the Snyder hydrograph.

In order to calculate unit peak discharge by SCS method, enter 1. Then accept the previous stage in the rational method to compute the time of concentration. By entering the project area, the program computes peak time, unit peak discharge and maximum unit peak discharge among three methods.

```
please insert total area of your project in m2 : 6000

Tp =

   15.1912


Qp =

    0.8215


Qmax =

    0.8215

m3/s
```

Figure 5. 8. Calculating the unit peak discharge of the SCS method.

The rivers with the stream gage stations, the flood discharge is obtained by means of the flood frequency analysis otherwise by using of the rain values and the unit hydrograph, the flood hydrograph and flood discharge is obtained.

5.2. Hydraulic Section of the Algorithm

After running the program, any value that is required for the program should be entered, the same as the section of the hydrology. At first, it should be entered flood discharge form previous section or another one if is needed. Then insert river length and depth of the water in the after-bay respectively

In the first attempt, by assuming width of weir limited and obtained from 3.19 equation. If the calculated discharge flow is equal to flood discharge, the program accepts the length of the weir and the height of the weir will be obtained from table 3.9 and section 3.2.2.3; otherwise, the program increases the length of the weir until the calculated discharge flow is equal to flood discharge. Then the program calculates the size of the standard weir crest by using the 3.2.2.3.(3). section.

At last the program computes dimensions of the weir and plot weir curve.

```
insert Q_flood in m3/s : 0.82
insert w_tot in m : 4
insert Hb in m : .5
Weir lenght is acceptable

L_weir =                L0 =

     2                       0.1350

m                       m

C =                     L1 =

    2.1560                   0.8381

                        m
P_weir =
                        L_tot =
    0.6500
                             0.9731
m
                        m
```

Figure 5. 9. Computing the weir parameters by the program.

In this stage, the program calculate the trench weir and stilling basin dimensions by inserting the Rake slope (ß) and its correspond value (k) from presenting table by the program, (a) and (b) distance from available rack, design discharge from hydrologic studies and eventually Flow coefficient (μ).

Note: Trench weir dimension is the coefficient of (b) and (l) that the user can choose suitable values for them.

```
insert beta from 0 to 28 degree : 11
insert k by corresponding beta from presented table : 0.9
insert a in mm : 30
insert b in mm : 80
insert Q_design in m3/s: 0.3
insert miu : 0.62
bl =
    1.3671
m
h0 =
    0.5000
m
h_a =
    0.0065
m
d1 is acceptable
d1 =
    0.2100
m
f_r =
    1.8503
V1 =
    1.9524
m/s
d2 =
    0.5595
only use stilling basin and didnt require to
energy disipater blocks and end sill
```

Figure 5. 10. Calculating the trench weir and stilling basin dimensions.

The program computes water depth, channel dimensions, and penstock diameter by entering the number of project's slopes, corresponding length and Manning coefficient.

To design the headrace channel, the program uses the rectangular best hydraulic section because of the uniform flow. The program accepts the velocity of the flow for preventing sediment deposits equal to 0.6m/s.

To design of the penstock diameter, the program utilizes the Gordon and Penman formula.

```
insert number of your project slop : 2
insert slop to design headrace, respectively : 0.06
insert L channel in m , respectively : 400
insert slop to design headrace, respectively : 0.02
insert L channel in m , respectively : 60
insert n : 0.014
Y =

    0.2100    0.3100

m

v =

    3.4014    1.5609

m

b =

    0.4200    0.6200

m

P_channel =

    0.2800    0.4133

m

d_penstock =

    0.3944

m
```

Figure 5. 11. Calculating the channel and penstock dimensions by the program.

In this stage by entering losses coefficient (kf), penstock length, power of the turbine, gross head and some of the turbine features, visible two lists of the height and the length of the settling basin are visible.

```
insert L (penstock lenght) in m : 120
insert kf local : 0.5
insert nomber of your bend : 2
insert kf bend, respectively : 0.25
insert kf bend, respectively : 0.38
insert nomber of your valve : 1
insert kf, respectively : 0.3
insert electrical power in kw : 500
usable head (gross head) in m : 270
Maximum particles diameter  that turbine permits to pass (smaller than 1.85) in mm : 0.75
overflow efficiency : 0.6
Guaranty discharge for use of irrigation and pass through the forebay tank in m3/s : 0.06
distance between the runner blades from turbine feature in m : 0.05
```

Figure 5. 12. Requiring values to compute the settling basin, and head losses.

Designing the settling basin continued. By choosing suitable height and length of settling basin from the list below:

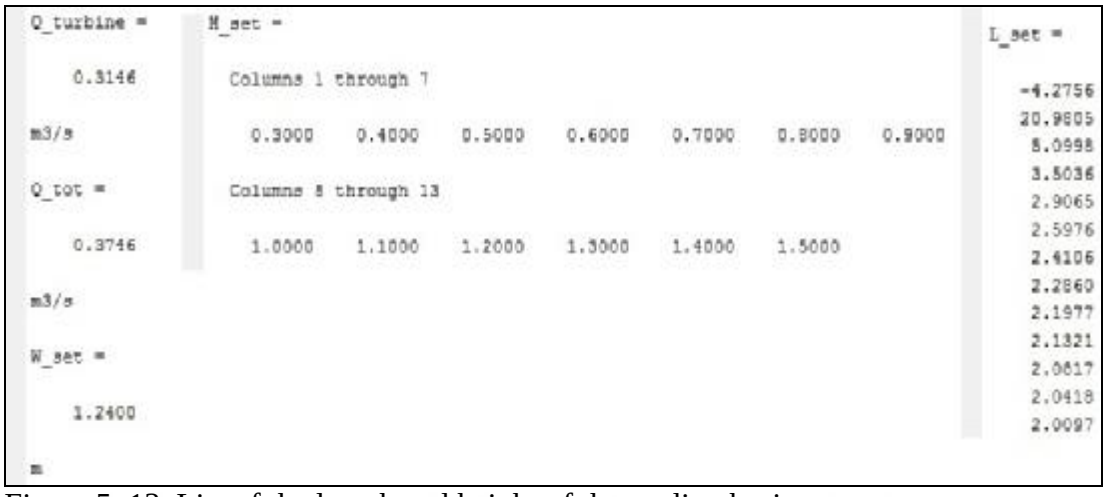


Figure 5. 13. List of the length and height of the settling basin.

Now, by entering chosen values, the program computes the dimensions of the forebay.

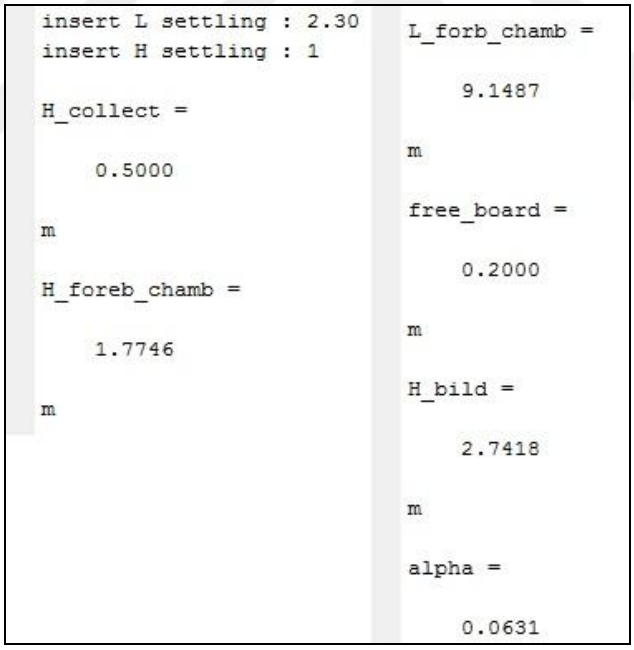


Figure 5. 14. Computing the forebay dimensions by the program.

Eventually, total loses head and net head is calculated.

```
insert phi from presented figure : 2.42
insert s from presented figure : 0.005

Hf_total =

   22.2047

m

H_net =

  247.7953

m
```

Figure 5. 15. Computing the net head and loses head by the program.

6. CONCLUSIONS

6. 1. The Aim of Writing the Program

When the engineer wants to design a hydropower system, most of the time he/she is asked to design a hydropower based on the specified turbine power. But, how? It is attempted to develop a computer program to pre-design the mini hydropower system so that the engineer has mental images about the system dimensions. By using this pre-design in calculations, engineers will design a better hydropower system. These mental images consist of site selection for intake and weir structures, channel, or penstock and gross or net head selection in his project.

6.2. Applications of the Program

The first section of the program is used to obtain unit peak discharge via Rational method, Snyder method and SCS method. The second section of the program is used as a pre-design into mini hydropower design.

6.3. Other Applications of the Program

In the hydrologic studies, to compute the unit peak discharge by means of Rational method, Snyder method and SCS method or one of them and using these results in designing any hydro structures.

Hydraulic section is used to design micro hydropower; in addition, it is to design the mini hydropower system. Also it is used to design a spillway or to compute the dimensions of the channel and penstock alone.

6.4. Recommendations for Further Development

- Because of the high number of the program's inputs, maybe some mistakes can occur when entering values. It can be reduced these

inputs by rearranging the program to decrease the volumes of the mistakes.

- If there is a stream gage station in the site of a project , It can be directly used its values; therefore, by adding this condition the sufficiency of the program (Matlab code) will be increased.
- This program is designed for runoff river condition that other conditions of the hydropower can be added.
- In the hydraulic section, a menu to select one or several elements of hydropower for designing can be added.
- The financial analysis to optimize the design of the hydropower can be used.
- It can be increased the number of selections for designing the intake, weir, channel, and penstock.

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CURRICULUM VITAE

Meysam KHOSHBAKHT was born in Maragheh, Iran in 1984. He received his B.S degree in Civil Engineering Department from Azad University Of Maragheh in 2006. After completing his B.S education he started MSc education in Civil Engineering Department in Çukurova university in Turkey. His research area is Hydro power plants and hydraulic structures.

APPENDIX

% hydrological section %

```
clear
clc
% calculating rational method
% Tc1 for overland flow
disp('Calculating peak flood discharge via three methods : ')
disp('1. Rational method')
disp('2. Snyder method')
disp('3. SCS dimentionless unit hydrograph method')
meth1=input('if you want to use Rational method, please insert 1 :
');
if meth1==1
disp('calculating flood discharge by using Rational method');
A=input('please insert total area of your project in m2 : ');
if A<=400000
no1=input('if you want to use Tc for overland flow please insert 1 :
');
if no1==1
disp('you can calculate the travel time with useing (Seelye Chart)
in min  : ');
imshow('fig02.jpg');
Tc1=input('please insert Tc for overland flow : '),disp('min')
else
    Tc1=0;
end
no2=input('if you want to use Tc for shallow, please insert 2 : ');
if no2==2
    Ku=1;
   imshow('fig03.jpg')
    K=input('please insert K (interception coefficient) from
presented table : ');
    Sp=input('please insert Sp (slope of the grade line or channel
slope percent) in m/m : ');
    L=input('please insert L (Flow length) in m : ');
    if L<=300
    V=Ku*K*(Sp^.5);
```

```
    Tc2=L/(60*V),disp('min')
    else
        disp('Maximum shallow flow length is 300m, please revise
inserted length ')
        L=input('please insert L (Flow length) in m equal or less
than 300m : ');
         V=Ku*K*(Sp^.5);
    Tc2=L/(60*V),disp('min')
end
else
    Tc2=0;
end

no3=input('if you want to use Tc for channel, please insert 3 : ');
if no3==3
   Ku=1;
   R=input('please insert R (hydraulic radius) in m : ');
   s=input('please insert s (slope of the grade line or channel
slope percent) in m/m : ');
   imshow('fig04.jpg');
   n=input('please insert n (Manning's roughness coefficient) from
presented table : ');
      V=Ku*((R^(2/3))*s^(1/2))/n;
   Tc3=L/(60*V),disp('min')
else
    Tc3=0;
end
Tc=Tc1+Tc2+Tc3;
if Tc<10, disp('your Tc less than 10 min, but the minimum of TC must
be 10 minutes')
    Tc=10;
end
disp('Tc that we calculated is : ')
Tc,disp('min')
i=input('please insert i in mm/hr via obtained TC from I-D-F curve
of your region : ');
imshow('fig05.jpg');
```

```
Ca=input('please insert ca (antecedent moisture factor) from
presented table : ');
n=input('please insert kind of cover: ');
AreaC=zeros(1,n);
c=zeros(1,n);
for i=1:n
    AreaC(i)=input('please insert Area of covers in m2 ,respectively
: ');
    imshow('fig01.jpg');
        c(i)=input('please insert Runoff Coefficient of this cover
from presented table : ');
end
C=0;
for j=1:n
    C=C+(c(1,j)*(AreaC(1,j)/A));
end
Q=(1/360)*C*i*Ca*(A/10000),disp('m3/s')
else
    disp('the rational method is not valid for A>400000 m2')
    disp('you can revise inserted area or use one of Snyder method
or SCS dimentionless unit hydrograph ')
end
end
meth2=input('if you want use snyder method insert 1 : ');
% calculating the snyder method
if meth2==1
disp('calculating flood discharge with using Snyder method');
alpha1=.75;
Ct=input('please insert Ct (empirical watershed coefficient) which
generally ranges from 1.8 to 2.2 : ');
L=input('please insert L (length along main channel from outlet to
divide) in km : ');
Lca=input('please insert Lca (length along main channel from outlet
to a point opposite the watershed centroid) in km : ');
Tl=alpha1*Ct*(L*Lca)^.3;
Tr=Tl/5.5;
%tr is T'R
tr=input('please insert T"R (desired unit duration) in h : ');
```

```
Tladj=Tl+.25*(tr-Tr);
Cp=input('please insert Cp (empirical constant) ranging from 0.5 to
0.7 : ');
alpha2=.275;
A=input('please insert total area of your project in m2  : ');
Qp=alpha2*((Cp*(A/1000))/Tladj)
Tb=3+(Tladj/8)
alpha50=.18
alpha75=.1
w50=alpha50*(1000*Qp/A)^(-1.075)
w75=alpha75*(1000*Qp/A)^(-1.075)
end
% w=[0 w50/3 w75/3 Qp 2*w75/3 2*w50/3 0];
% t=[0 .7 .9 Tb-Tr 2.4 2.9 Tb]

% calculating SCS
meth3=input('if you want use SCS method insert 1 : ');
if meth3==1
disp('calculating flood discharge with using SCS method')
no1=input('if you want to use Tc for overland flow insert 1 : ');
if no1==1
disp('you can calculate the travel time with useing (Seelye Chart)
: ');
imshow('fig02.jpg');
Tc1=input('please insert Tc for overland flow : '),disp('min')
else
    Tc1=0;
end
no2=input('if you want to use Tc for shallow insert 2 : ');
if no2==2
    Ku=1;
   imshow('fig03.jpg')
    K=input('please insert K (interception coefficient) from
presented table : ');
    Sp=input('please insert Sp (slope of the grade line or channel
slope percent) in m/m : ');
    L=input('please insert L (Flow length) in m : ');
```

```
    if L<=300
    V=Ku*K*(Sp^.5);
    Tc2=L/(60*V),disp('min')
    else
        disp('Maximum shallow flow length is 300m, please revise
inserted length : ')
        L=input('please insert L (Flow length) in m equal or less
than 300m : ');
         V=Ku*K*(Sp^.5);
    Tc2=L/(60*V),disp('min')
end
    V=Ku*K*(Sp^.5);
    Tc2=L/(60*V),disp('min')
else
    Tc2=0;
end
no3=input('if you want to use Tc for channel insert 3 : ');
if no3==3
   Ku=1;
   R=input('please insert R (hydraulic radius) in m : ');
   s=input('please insert s (slope of the grade line or channel
slope percent) in m/m : ');
imshow('fig04.jpg');
   n=input('please insert n (Manning's roughness coefficient) from
presented table : ');
      V=Ku*((R^(2/3))*s^(1/2))/n;
   Tc3=L/(60*V),disp('min')
else
    Tc3=0;
end
Tc=Tc1+Tc2+Tc3;
if Tc<10, disp('your Tc less than 10 min, but the minimum of TC must
be 10 minutes')
    Tc=10;
end
disp('Tc that we calculated is : ')
Tc,disp('min')
A=input('please insert total area of your project in m2 : ');
```

```
Tp=2*Tc/3
Kp=2.08;
Td=.133*Tc;
Qp=(Kp*(A/1000))/Tp

end
% Q/Qp=qiu
qiup=[0 0.0300 0.1000 0.1900 0.3100 0.4700 0.6600 0.8200 0.9300
0.9900 1.0000 0.9900...
    0.9300 0.8600 0.7800 0.6800 0.5600 0.3900 0.2800 0.2070 0.1470
0.1070 0.0770...
    0.0550 0.0250 0.0110 0.0050 0];
tipi=[0 0.1000 0.2000 0.3000 0.4000 0.5000 0.6000 0.7000 0.8000
0.9000 1.0000...
     1.1000 1.2000 1.3000 1.4000 1.5000 1.6000 1.8000 2.0000 2.2000
2.4000...
     2.6000 2.8000 3.0000 3.5000 4.0000 4.5000 5.0000];

qiu=Qp*qiup;
T=Tp*tipi;
plot(T,qiu)
Qmax=max(qiu), disp('m3/s')
```

```
% design of minihydro power plants
% sizing of the weir

clear
clc
Q_flood=input('insert Q_flood in m3/s : ');
W_tot=input('insert w_tot in m : ');
W_wingwall=.25;
W_scarringchannel=1;
W_dividing=.5;
L_weir=W_tot-2*W_wingwall-W_scarringchannel-W_dividing;
H_b=input('insert Hb in m : ');
H_0=H_b;
P_H0=1.3:.1:3;
c=[2.156 2.159 2.161 2.163 2.165 2.166 2.167 2.168 2.169 2.17 2.172
2.173 2.173 2.174 2.174 2.175];
for i=1:size(c,2)
    Q_call=L_weir*c(i)*H_0^1.5;
    if Q_call>=Q_flood
        disp('Weir lenght is acceptable')
        L_weir=W_tot-2*W_wingwall-W_scarringchannel-W_dividing,
disp('m')
        C=c(i)
        P_weir=H_0*P_H0(i), disp('m')
        break
    else
        continue
    end
end
if Q_call<=Q_flood
    l_weir=.01:.1:1000;
    c=2.175
    P_H0=3;
    for i=1:size(l_weir,2)
        Q_call=l_weir(i)*c*H_0^1.5;
        if Q_call>=Q_flood
           disp('Weir length is computed')
```

```
            L_weir=l_weir(i-1), disp('m')
            P_weir=H_0*P_H0, disp('m')
            break
         else
            continue
        end
    end
end
% sizing a standard weir crest
imshow('fig101.jpg');
x=0:.1:P_weir;
y=.5*x.^1.85./H_b^.85;
L0=.27*H_b, disp('m')
L1=(2*P_weir*H_b^.85)^(1/1.85), disp('m')
L_tot=L0+L1, disp('m')
y=-y;
plot(x,y)



% design the trench weir
beta=input('insert beta from 0 to 28 degree : ');
imshow('fig102.jpg');
k=input('insert k by corresponding beta from presented table : ');
h_crit=(2/3)*H_b;
h=h_crit*k;
imshow('fig103.jpg');
a=input('insert a in mm : ');
b=input('insert b in mm : ');
beta=pi*beta/180;
c=0.6*(a/b)*(cos(beta)^(1.5));
Q_design=input('insert Q_design in m3/s: ');
miu=input('insert miu : ');
g=9.81;
bl=Q_design*3/(2*c*miu*sqrt(2*g*h)), disp('m')
```

```
% design of stilling basing
d3=H_b;
V3=Q_flood/(L_weir*d3);
g=9.81;
E2=d3+V3^2/(2*g);
q=Q_flood/L_weir;
h0=H_b, disp('m')
h_a=q^2/(2*g*(P_weir+h0)^2), disp('m')
E1=P_weir+h0+h_a;
E=E1-E2;
d11=.01:.1:100;
fr=zeros(1,size(d11,2));
d22=zeros(1,size(d11,2));
d=zeros(1,size(d11,2));
for i=1:size(d11,2)
    fr(i)=q^2/(g*d11(i)^3);
    d22(i)=(d11(i)/2)*sqrt(1+8*fr(i)^2);
    d(i)=(d22(i)-d11(i))^3/(4*d22(i)*d11(i));
    if E>=d(i)
        disp('d1 is acceptable')
       d1= d11(i), disp('m')
       f_r= fr(i)
        V1=Q_flood/(L_weir*d1), disp('m/s')
        d2=d22(i), disp('m')
        break
    end

end
if f_r<1.7
    disp('No stilling basin required')
elseif 1.7<f_r<2.6
    disp('only use stilling basin and didnt require to')
    disp('energy disipater blocks and end sill')
elseif 2.6<f_r<4.5
    imshow('fig104.jpg');
    disp('stilling basin USBR type IV')
elseif f_r>4.5 && v<15
```

```
    imshow('fig105.jpg');
    disp('stilling basin USBR type III')
else
    imshow('fig106.jpg');
    disp('stilling basin USBR type II')
end

% design of the headrace
no=input('insert number of your project slop : ');
s=zeros(1,no);
L_chan=zeros(1,no);
for z=1:no
    s(z)=input('insert slop to design headrace, respectively : ');
    L_chan(z)=input('insert L channel in m , respectively : ');
end
imshow('fig112.jpg');
n=input('insert n : ');
y=.01:.1:1000;
A=zeros(1,size(y,2));
p=zeros(1,size(y,2));
L=zeros(1,size(y,2));
a=zeros(1,no);
v=zeros(1,no);
Y=zeros(1,no);
r=zeros(1,no);
b=zeros(1,no);
P_channel=zeros(1,no);
kf_headrace=zeros(1,no);
hf_headrace=zeros(1,no);
g=9.81;
Hf_headrace=0;
for j=1:no
for i=1:size(y,2)
    A(i)=2*y(i)^2;
    p(i)=4*y(i);
    L(i)=(A(i)^(5/3)*s(j)^(1/2))/(n*p(i)^(2/3));
    if L(i)>=Q_design
```

```
        Y(j)=y(i), disp('m')
        a(j)=A(i);
        v(j)=Q_design/a(j), disp('m')
        if v(j)< .6
           v(j)=.6;
           a(j)=Q_design/v(j);
           Y(j)=sqrt(a(j)/2), disp('m')
        end
        r(j)=.5*y(i);
        kf_headrace(j)=2*g*(n^2*L_chan(j)/r(j)^(4/3));
        hf_headrace(j)=kf_headrace(j)*v(j)^2/(2*g);
        b(j)=2*Y(j), disp('m')
        P_channel(j)=Y(j)+(Y(j)/3), disp('m')
        break

    else
        continue
    end
end
     Hf_headrace=Hf_headrace+hf_headrace(j);
end
H_channel=Y(no);

% design of penstock
d_penstock=(720*Q_design^.5)/1000, disp('m')
ri=d_penstock/2;
imshow('fig113.jpg');
sigma_u=input('insert sigma_u : ');
imshow('fig107.jpg');
diff_h=input('insert diff_h between turbine and forebay tank in m :
');
p_i=1.1*diff_h, disp('m')
t_min=-ri*(sqrt((sigma_u-p_i)/(sigma_u+p_i))-1), disp('m')
t_chosen=3.5*t_min;
L=input('insert L (penstock lenght) in m : ');
g=9.81;
A=(pi*d_penstock^2)/4;
```

```
v=Q_design/A;
kf_penstock=(124.5*n^2)/d_penstock^(1/3);
hf_penstock=(kf_penstock*L*v^2)/(2*g*d_penstock);
imshow('fig108.jpg');
kf_local=input('insert kf local : ');
hf_local=kf_local*v^2/(2*g);

no1=input('insert nomber of your bend : ');
kf_bend=zeros(1,no1);
for z=1:no1
    imshow('fig109.jpg');
    kf_bend(z)=input('insert kf bend, respectively : ');
end
hf_bend=zeros(1,no1);
for i=1:size(kf_bend,2)
    hf_bend(i)=kf_bend(i)*v^2/(2*g);
end
Hf_bend=0;
for j=1:size(hf_bend,2)
    Hf_bend=Hf_bend+hf_bend(j);
end

no2=input('insert nomber of your valve : ');
kf_valve=zeros(1,no2);
for z=1:no2
    imshow('fig110.jpg');
    kf_valve(z)=input('insert kf, respectively : ');
end
hf_valve=zeros(1,no2);
for i=1:size(kf_valve,2)
    hf_valve(i)=kf_valve(i)*v^2/(2*g);
end
Hf_valve=0;
for j=1:size(hf_valve,2)
    Hf_valve=Hf_valve+hf_valve(j);
end
```

```
% design of settling basin
P_elec=input('insert electrical power in kw : ');
H_gross=input('usable head (gross head) in m : ');
d=input('Maximum particles diameter  that turbine permits to pass
(smaller than 1.85) in mm : ');
eta=input('overflow efficiency : ');
Q_irregation=input('Guaranty discharge for use of irrigation and
pass through the forebay tank in m3/s : ');
dist=input('distance between the runner blades from turbine feature
in m : ');
%step1
Q_turbine=P_elec/(eta*9.81*H_gross), disp('m3/s')
Q_tot=Q_turbine+Q_irregation, disp('m3/s')
%step2
V_crit=.44*sqrt(d);
W0=(100/(9*d))*(sqrt(1+157*d^3)-1);
W_channel=b(no);
W_set=2*W_channel, disp('m')
H_set=.3:.1:1.25*W_set
V=zeros(1,size(H_set,2));
W=zeros(1,size(H_set,2));
L_set=zeros(1,size(H_set,2));
for i=1:size(H_set,2)
    V(i)=Q_tot/(2*H_set(i));
    W(i)=(W0/1000)-((V(i)*.132)/sqrt(H_set(i)));
    L_set(i)=H_set(i)*(V(i)/W(i));
end
V=V';
W=W';
L_set=L_set'
L_set=input('insert L settling : ');
H_sett=input('insert H settling : ');
V_collec_cap=(W_set*H_sett*L_set/4);
H_collect=2*V_collec_cap/(W_set*L_set), disp('m')
%step3
V_fore_tank=75*Q_turbine;
A1=W_set*H_sett;
A2=W_channel*H_channel;
```

```
V_entry=(W_set/3)*(A1+A2+sqrt(A1*A2));
V_set=L_set*W_set*H_sett;
V_foreb_cham=V_fore_tank-V_entry-V_set;
H_foreb_chamb=4.5*d_penstock, disp('m')
W_div_wall=.15;
H_div=.2;
V_div_wall=(H_foreb_chamb-(H_sett-H_div))*W_set*W_div_wall;
L_forb_chamb=(V_foreb_cham+V_div_wall)/(W_set*H_foreb_chamb),
disp('m')
%step4
H_b1=Q_irregation^(2/3)/(2.95*.5*W_channel)^(2/3);
H_b2=Q_flood^(2/3)/(2.95*.5*W_channel)^(2/3);
free_board=0.2, disp('m')
H_bild=H_foreb_chamb+(H_b2-H_b1)+free_board, disp('m')
alpha=atan(H_sett-H_div-H_b1)/(L_forb_chamb-W_div_wall)
alpha=alpha*180/pi;
%step5
a=dist/2;
imshow('fig111.jpg');
phi=input('insert phi from presented figure : ');
s=input('insert s from presented figure : ');
kf_trash_rack=phi*(s/a)^(4/3)*sin(alpha);
velocity=Q_flood/(W_set*H_sett);
Hf_trash_rack=kf_trash_rack*(velocity^2/(2*9.81));

% computing of head losses
Hf_total=Hf_trash_rack+Hf_valve+Hf_bend+hf_local+hf_penstock+Hf_head
race, disp('m')

H_net=H_gross-Hf_total, disp('m')
```