Digest 2014, December 2014, 1847-1864

Computer Assisted Preliminary Design of Run-of- River Plants†

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ABSTRACT

Run-of-river type hydroelectric power plants generate electrical energy by using a certain portion of the available flow in the river. A computer program called MINI-HPD is developed to perform the hydraulic design of run-of-river plants. This program, which runs under the Windows operating system, is developed in C# programming language. MINI-HPD is capable of performing hydraulic design of structural components of diversion weir with lateral intake and overflow spillway, canal, forebay, and penstock. In addition, it can determine the optimum design discharge, optimum installed capacity, and optimum penstock diameter for this type of plants. It is desired to have quick successive runs under various scenarios and combinations of relevant parameters. An application is presented to illustrate the use of the program.

Keywords: *Hydroelectrical energy, run-of-river plant, computer aided design.*

1. INTRODUCTION

Hydroelectric power maintains its importance in an increasing manner due to renewable and sustainable nature of water. When our country is considered in general, it is observed that the hydropower potential is in large magnitudes as the mountainous land constitutes a large portion of the country with high heads, in addition, with the use of rivers having regular regimes. There are some studies performed by State Hydraulic Works of Turkey (DSI) and the private sector in order to determine the hydropower potential of the country. In the recent years, the investments for run-of-river type plants have been encouraged by the state. Within the Law No. 4628, over 1200 permits to build run-of-river plants have been issued, of which about 200 have been put into operation, and about 200 of them are currently under construction [1].

The design of hydroelectric power systems is complicated due to the limitations introduced by local geotechnical conditions, structural and hydraulic requirements. In order to insure the desired structural safety, which may create new conditions, hydraulic design may take different forms. In addition, in each project, some components may have their own unique design features imposed by the local conditions, economy, environmental and social issues.

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† Published in Teknik Dergi Vol. 25, No. 4 October 2014, pp: 6925-6942

The designer normally seeks the optimum design of all structural components conforming to the aforementioned aspects. Therefore, it is a very much time consuming effort, to carry out numerical procedures in order to compare different design alternatives and satisfy design requirements. The aim of the present study is to develop computer software so that when utilized it could lead to time saving in such a way that it would enable quick comparisons of several alternatives in an iterative manner, comfortably. Although some software packages are available in literature for the design of various structural components of hydropower plants, a computer program incorporating the design of all components in an orderly manner in a single program does not exist. Consequently, waste of time will be avoided by using one single program to design all the components required. MINI-HPD, the software developed in this study, thanks to its easy-to-use feature and visual contribution can handle the input data entrance comfortably. It can also provide graphical outputs as well as the results table consisting of all the design parameters computed in accordance with the hydraulic design criteria upon which there is a consensus in our country. Since the program designs each hydraulic structure component in a sequential manner, the hydraulic interaction between different components can be observed, too. Prior to introducing the program developed in the present study, firstly, the widely accepted fundamentals of hydraulic design criteria in our country will be briefly visited. Subsequently, information about the program will be presented along with a case study and the discussion of it.

2. FUNDAMENTALS OF HYDRAULIC DESIGN OF RUN-OF-RIVER PLANTS

This section covers the determination of design parameters and design basics within the framework of applications in Turkey. In this respect, optimum design discharge, selection of penstock diameter and installed power along with design of forebay, conveyance channel and design of diversion weirs with side intake are reviewed.

2.1. Determination of Optimum Design Discharge

Since water storage does not exist in run-of-river plants flow-duration curve is needed for streams. In order for this curve to be reliable enough to include various flow regimes, a long record of data at least for 30 years is recommended [2]. Moreover, peak discharges with high recurrence intervals are to be determined for the design of diversion weir. In the practice of our country, a peak discharge with a 100 year recurrence interval for the design of diversion weir (Q_{100}), and moreover peak discharges with various recurrence intervals from 5 to 50 years (Q_5, Q_{10}, Q_{25}, Q_{50}) are to be used to check the performance of the energy dissipating basins. Consequently, prior to running the program, a frequency analysis of annual flow series should be performed and suitable distribution function representing the discharges in the project area including aforementioned discharge values should be determined.

To determine the optimum design discharge net benefits of different alternatives should be computed and the one that gives the maximum net benefit should be selected. In this respect, for each design discharge alternative on the flow-duration curve, the costs of penstock and installed capacity, the energy generation, and corresponding benefits are calculated and the net benefits of these alternatives are compared. The optimum design discharge is decided based on the maximum net benefit. This process is explained below:

a) In order to calculate the power and corresponding energy generation, there is a need for an approximate penstock diameter. The following velocity expression can be used for this step, [3].

$$V = 0.125\sqrt{2gH_g} \tag{1}$$

where V is the average velocity in the penstock in m/s, g is the gravitational acceleration (9.81 m/s^2) and H_g is the gross head of the power plant (m). For penstocks, the maximum allowable flow velocity is suggested to be 5 m/s [3]. This is reasonable since velocities higher than that would create both high shear stresses inside the pipe surface and high pressure surges during water hammer. The velocity obtained from Equation (1) provides a penstock diameter for each discharge considered by means of the continuity equation:

$$D = \sqrt{\frac{4Q}{\pi V}} \tag{2}$$

b) Safe wall thickness of each penstock diameter is obtained considering the water hammer.

c) The flow-duration curve is converted into power-duration curve using the power-discharge relation.

d) For each alternative, firm energy, secondary energy, and total energy are calculated. In this respect, the flow duration curve is divided into equal time intervals, Δt. In order to calculate the marginal energy generation between these equal time intervals, an equivalent discharge, Q_{eq} is computed to represent the average discharge as Q_{eq} = duration (%)*ΔQ, where ΔQ is the discharge difference between two consecutive time intervals. Then, marginal energy produced between time intervals, is expressed as $8760\gamma H_n Q_{eq} e$, where 8760 are the number of hours in a year, e is the overall plant efficiency, γ is the specific weight of water, H_n is the net head.

e) In this step, the costs of penstock and installed capacity are calculated. The weight of penstock for circular cross-section can be computed from:

$$G = \pi\gamma_p DLt \tag{3}$$

where γ_p is the specific weight of pipe material, D is the penstock diameter, L is the penstock length and t is the wall thickness of penstock. The annual cost of penstock and installed capacity can be computed respectively, from:

$$C_p = C_{pu} * CRF * G \tag{4}$$

$$C_{ic} = C_{iuc} * P_p * CRF \tag{5}$$

in which C_{pu} is the unit cost of penstock, C_{icu} is the unit cost of installed capacity, CRF is the capital recovery factor, P_p is the installed capacity. The maximum installed capacity is given by Equation (6).

$$P_p = \gamma Q H_n e \tag{6}$$

f) The total cost is obtained as the summation of the costs of penstock and installed capacity. The firm and secondary energy benefits are obtained by multiplying these energies by their unit prices. Therefore, for each discharge being studied the net benefit is calculated. The discharge that yields the maximum net benefit is selected as the design discharge, (Q_d).

2.2. Determination of Optimum Penstock Diameter

The program at this stage determines the optimum penstock diameter that has a wall thickness providing safety against water hammer while passing the optimum design discharge. The penstock wall thickness, t, mainly depends on the maximum tensile strength of the pipe material, σ_t, pipe diameter, D, and the operating pressure. In case of water hammer events the pressure on the penstock is increased drastically, hence the wall thickness must be selected by taking into account the head increase ΔH on the penstock due to water hammer. The wall thickness, t, of penstock is calculated from [4]:

$$t = \frac{\gamma(\Delta H + H_s)D}{2\sigma_t} \tag{7}$$

where ΔH is head increase due to water hammer and H_s is the steady state head. The pipe must be appropriately rigid in order to manage deformations in the field sufficiently. Therefore, ASME suggests a minimum wall thickness in mm that is equal to the 2.5 times the pipe diameter in m plus 1.2 mm [5]. Detailed information and equations on the theory of water hammer and the head increase due to water hammer may be found in [4]. Generally, penstocks are made of steel, and hence subject to corrosion. Therefore, after the wall thickness of penstock is determined with the above procedure, it is increased by additional 2 mm to provide safety.

The loss in energy generation of the plant is computed for various penstock diameters. The cost of energy loss is obtained by multiplying the losses by the unit prices of energy generation. Total cost for each penstock diameter is calculated with the summation of cost of the penstocks and cost of the energy losses. The diameter that gives the minimum total cost value is selected as the optimum penstock diameter.

2.3. Design of Forebay and Canal

The plan and cross-section of a typical forebay is given in Figure 1. The best hydraulic section approach is utilized by the program to determine the dimensions of the canal which has a trapezoidal cross-section. After obtaining the dimensions of canal, the flow depth in the canal that corresponds to the design discharge is computed. By neglecting the local losses, the elevation of the water level in the canal is assumed to coincide with the water surface elevation of the forebay. Sufficient submergence depth should be provided, even with the minimum operation level in the forebay, to prevent the formation of severe vortices near the intake, and consequently, air entrainment into the penstock. This prevents potential cavitation in turbines. The minimum submergence height, shown with the symbol s in Figure 1, measured between the centerline of the penstock and the minimum water surface elevation, is given by Knauss [6] as follows:

$$s = (D \text{ or } 1.5D) \qquad \text{for } F_r \leq 0.25 \tag{8}$$

$$s = D(0.5 + 2F_r) \qquad \text{for } F_r > 0.25 \tag{9}$$

where F_r is the Froude number obtained by using the penstock diameter.

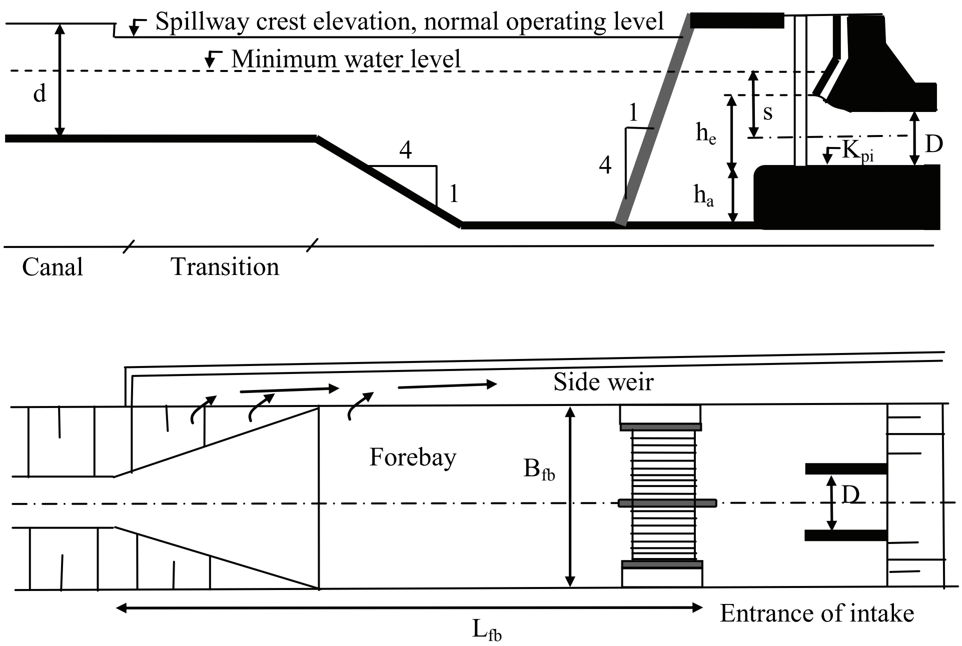


Figure 1. The plan and cross-section of forebay

In Turkish practice, the minimum water surface elevation in the forebay is obtained through the flow depth computed by the Manning's Equation for a discharge of $0.75Q_d$ [3]. In other words, any discharge less than $0.75Q_d$ would create the sediment deposition and vegetation problems on the bed which lead to increase in the coefficient of roughness, and decrease in the discharge. Assuming that the invert elevation of the penstock is known in relation to local conditions and power plant location, by adding the value of s to the centerline elevation of the penstock, the minimum operation level is obtained. The bottom elevation of the conveyance channel is determined by subtracting the flow depth found with $0.75Q_d$ from the minimum operation level. A transition at the entrance of penstock is provided in order to create a gentle approach flow. The total height just upstream of the transition, h_e, (see Figure 1) is computed from:

$$h_e = \frac{\pi D^2}{4bC_c} \tag{10}$$

where b is the width of intake section and C_c is the contraction coefficient, which can be taken as 0.6 in the preliminary design [3]. Since the width of intake section is equal to the penstock diameter, the only unknown, h_e, can be obtained from Equation (10). An aeration pipe should be installed over the penstock near the region where sub-atmospheric pressure may occur, to avoid the collapse of the pipe. The depression pressure that may collapse the pipe, P_c, in GPa is given in [5] as :

$$P_c = 882500\left(\frac{t}{D}\right)^3 \tag{11}$$

in which t and D, respectively, are the wall thickness of the penstock and the penstock diameter in mm. The diameter of aeration pipe, D_A, in cm can be calculated by Equations (12) and (13) from [5]:

$$D_A = 7.47\sqrt{\frac{Q_d}{\sqrt{P_c}}} \qquad \text{for } P_c \le 0.49 \tag{12}$$

$$D_A = 8.94\sqrt{Q_d} \qquad \text{for } P_c > 0.49 \tag{13}$$

In the above equations, P_c is in GPa, and Q_d is in m^3/s. As can be seen in Figure 1, an upward step is placed for the sediment prone to accumulate on the bed of the forebay, to prevent it from entering the penstock. The height of upward step, Δ_a, is suggested to be at least $0.3h_e$ by USBR [7]. A trashrack is provided in order to capture floating objects encountered in the conveyance canal and the forebay. The width of the trashrack, B_{tr}, is computed from:

$$B_{tr} = \frac{Q_d}{V_a\left(\Delta_a + s + 0.5h_e\right)} \tag{14}$$

where Q_d is the total design discharge, V_a is the approach velocity in front of the trashrack, desired to be around 0.6 and 0.9 m/s. To determine the total width of forebay, B_{fb}, a minimum increment of Δ_a is suggested by USBR [7] to add to the both sides of the intake.

In case of a very wide forebay width, the intake section can be divided into two sections with a pier in order to provide a gentle flow to penstock. Therefore, B_{fb} is calculated from $B_{tr}+t_{pf}+2\Delta_a$, where, t_{pf} is the thickness of the pier. In the program, the volume of forebay, V_{fb}, is an input parameter. In practice this value can be taken in terms of m^3 as $90Q_d$, [3], in which Q_d should be in m^3/s. The length of the forebay, L_{fb} is obtained from Equation (15):

$$L_{fb} = \frac{V_{fb}}{B_{fb} h_{fb}} \tag{15}$$

where h_{fb} is the difference between maximum and minimum water levels in forebay. In a preliminary approach, the ratio L_{fb}/B_{fb} is desired to be about 2.5 to 3.0 [3]. The final design can be achieved according to topographic conditions and the availability of land space. To determine the crest elevation of side spillway, normal water level in the forebay is incremented by 10 cm in order to handle fluctuations of water surface in the forebay. In the present study, the head on the side spillway is computed by the standard spillway discharge equation [8] to be on the safe side, instead of using iterative hydraulic computations based on the conservation of momentum and energy. By taking the length of side spillway equal to the length of forebay and following the iterative process that solves Equation (15) and spillway discharge equation simultaneously the unknown head and the length of forebay are calculated.

2.4. Design of Diversion Weir

The boundary condition for the design of diversion weir is accepted to be the water level at the entrance of the canal, which is explained before. Assuming uniform flow conditions throughout the canal, the entrance bottom elevation and the water level at that section are known since the related elevations are obtained at the stage of forebay design. Starting from the canal entrance, the related head losses and water surface elevations in the intake section are calculated using energy equation. Since the flow between the intake section and the canal entrance is in subcritical regime, the calculations are performed from downstream to upstream. The computation process for determining the crest elevation of spillway is explained by dividing the relevant reach into 9 sections as the section-1 being the canal entrance at the end of the intake. The plan and cross-sectional view of the intake is given in Figure 2. All the computations are based on the application of the energy equation in consecutive order by considering all possible losses. Details of the computations can be found in [9].

Settling basin is an important component of the intake since sediment free flow needs to be transmitted to the power house. In order to prevent the entrainment of sediment to the canal, an upward sill is provided at the end of the settling basin. A relatively low flow velocity is required for the suspended particles to settle in the settling basin. Size of the settling particles in the basin depends on the available head of the power house. Only relatively small particles can be permitted to be transmitted to the penstock under high heads. The relationship between the maximum permissible flow velocity, V_{max}, in the settling basin in m/s and the maximum diameter of the particle, D_{max}, in mm to be settled is defined by Camp [10] as

$$V_{max} = a\sqrt{D_{max}} \tag{16}$$

where a is a parameter determined according to the particle diameter. The values of a are 36, 44, and 51 for $D_{max} \geq 1$ mm, 0.1 mm $< D_{max} < 1$ mm, and $D_{max} \leq 0.1$ mm, respectively [10]. By selecting the height of the upward sill at the end of the settling basin, Δ_{su}, between 0.5 m and 1.0 m and applying the energy equation between sections 4 and 5, the value of the flow depth at section 5 (y_5) can be computed.

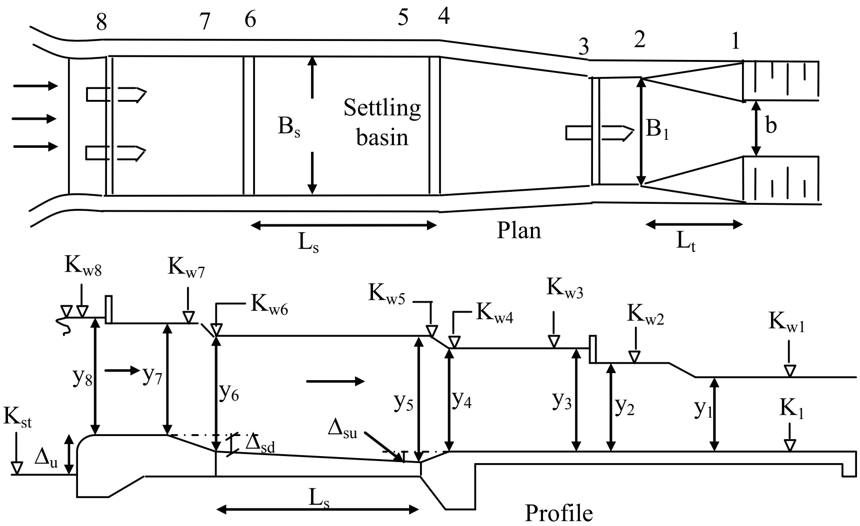


Figure 2. Plan and profile of water intake system

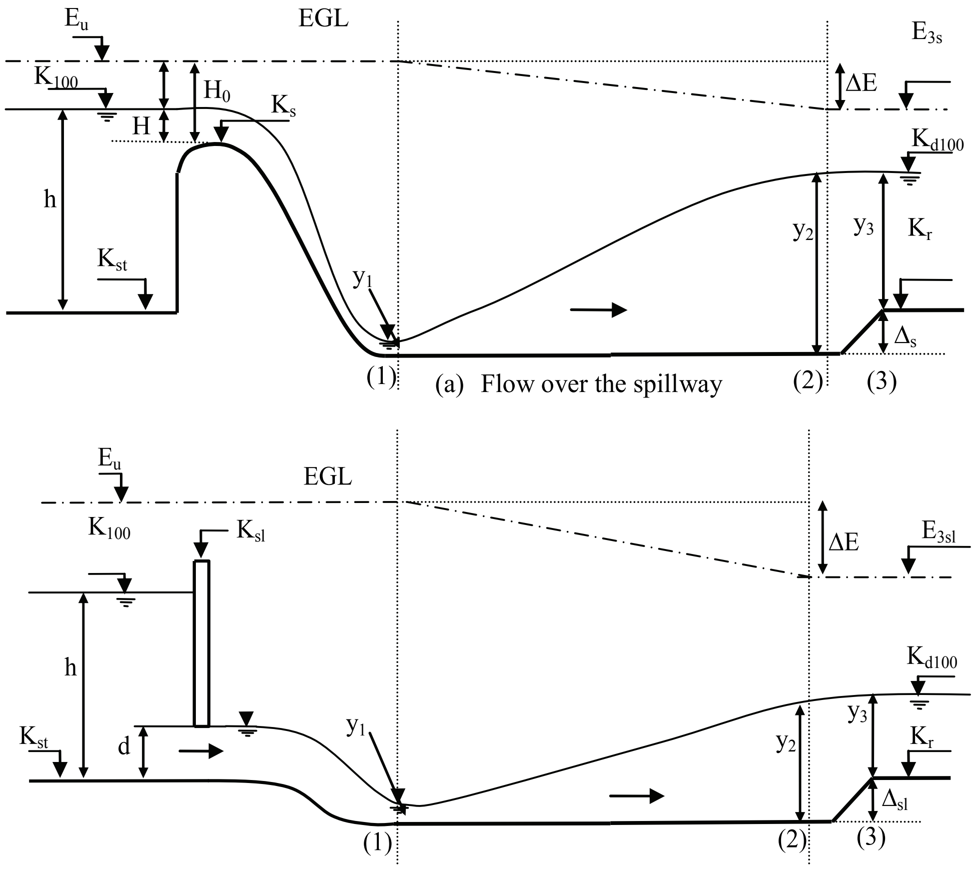
The average flow velocity at the end of the settling basin should be smaller than V_{max}. If this criterion is not satisfied, the height of the upward step is increased gradually or the width of the settling basin is increased. The length of settling basin, L_s, is determined from:

$$L_s = \frac{Q_d}{W_f B_s} \tag{17}$$

where Q_d is the design discharge, B_s is the width of the settling basin, and W_f is the fall velocity of the particle. Particles of considerable size can damage the penstock and the turbines. Therefore, the maximum size of the particles to be settled in the settling basin, D_{max}, should be determined according to the head of the plant along with the turbine type and operational conditions. After the fall velocity is obtained for the particle size, D_{max}, the length of the settling basin, L_s, is calculated by using Equation (17). To be on the safe side, the obtained value of L_s may be slightly increased. After the application of the energy equation in sequential order, the water surface elevation in front of the intake is eventually determined. This value is incremented by about 10 cm considering possible fluctuations of water level to determine the spillway crest elevation. The value of the upward sill at the entrance of the intake, Δ_u, which is in the range of 0.5 m to 1.0 m is checked [11].

2.5. Design of Spillway, Sluiceway, and Energy Dissipation Basin

In Turkish practice, the river discharge Q_{100} is transmitted to the downstream as shown in Figure 3 through the sluiceway, Q_{sl}, and over the spillway, Q_s, such that $Q_{100}=Q_{sl} + Q_s$. The determination of Q_s and Q_{sl} is an iterative process [11].



(b) Flow through the sluiceway

Figure 3. Profiles of spillway and sluiceway [9]

Preliminary dimensions are assigned to the sluiceway(s) according to the local conditions and sediment regime of the river concerned. The spillway net length is then obtained. A water surface elevation is assumed at the upstream and the corresponding fractions of spillway and sluiceway discharges are computed, sum of which should be equal to Q_{100}. This iteration is repeated by changing the upstream water level until Q_{100} discharge is obtained. The upstream energy grade line elevation is then determined. The river flow that is transmitted over the spillway and through the sluiceway creates different flow conditions at the toe of the structure. Therefore, the analysis of these sections needs to be carried out separately. With the given flow depth at the end of the riprap section, the local energy grade line elevations corresponding to the spillway (E_{3s}) and sluiceway (E_{3sl}) as shown in Figure 3 are computed.

Hence, the required depths of lowering of the stilling basin for the computed head difference between the upstream and downstream (ΔE) are computed for the spillway toe (Δ_s) and sluiceway toe (Δ_{sl}). The above process is applied for both spillway and sluiceway. If the difference between the values of these depths of excavation is small, a common stilling basin according to classical USBR types [8] is chosen for the case of greater depth of excavation. This approach is more economical compared to the case of two different stilling basins which are separated by a dividing wall. Since the necessary length of the stilling basin is dictated by the length of hydraulic jump, which may vary according to the flow conditions, the aforementioned analyses are also repeated for the other discharges to obtain the deepest and longest basin.

3. DEVELOPMENT OF SOFTWARE MINI-HPD

A computer software named MINI-HPD is developed to determine the optimum design discharge and installed power of the system and to design the diversion weir, canal, forebay, and penstock. MINI-HPD is developed in C# programming language and works under Microsoft Windows operating system. It is a user-friendly computer program that provides a simple working environment to the user. Mainly, the program consists of a main input window backed up with several output windows. The general flowchart of the program is given in Figure 4. The input data and the outputs of the subroutines are presented in Table 1.

4. APPLICATION

The hydraulic design of a run-of-river type hydropower plant is to be performed using MINI-HPD. The characteristics of the structural components and the input data necessary to execute the program are given below. The data used in this program are extracted from a design which is performed by a private company in Turkey. The outputs of the program are also presented below. Prior to the execution of the program, it is assumed that the hydrological analysis of the river has been completed. Hence, the flow duration curve as shown in Figure 5 and flood peak discharges and corresponding water surface elevations at riprap section as presented in Table 2 are input data.

A single penstock connected with two branches to the turbines is to be constructed between the forebay and power house. The related input variables regarding penstock are listed as follows: length of penstock is 254.00 m, friction factor of penstock is 0.012, elasticity modulus of pipe material is 206 GPa, allowable tensile stress is 400 MPa, unit price of penstock is 7.52 \$/kg, turbine closure time is 5 s, and penstock invert elevation is 581.25 m. A lined trapezoidal canal, which is 17.214 km long, is constructed between the diversion weir and forebay. It has side slopes 1V:1.5H, where V and H stand for the vertical and horizontal values of the side inclinations. Manning's roughness coefficient for the canal is 0.016 with a bed slope of 0.0003. The lateral intake structure will divert the flow at the left bank. There are single piers, 0.5 m wide, at sections 3 and 8 of the intake. The maximum settlement diameter for the sediment is selected as 0.2 mm. The thickness and spacing of rackbars at section 8 are 0.015 m and 0.10 m, respectively. A single sluiceway 4.5 m wide and 2 m high is considered. The spillway crest length is 60 m. The bed elevation at riprap section and the thalweg elevation at the entrance of intake are 587.00 m and 589.40 m, respectively. The gross head is 78.00 m, turbine efficiency is 0.94, generator efficiency is 0.97, transformer efficiency is 0.98, firm energy unit price is 8 \$/kWh, secondary energy unit price is 8 \$/kWh, unit cost of installed capacity, is 1,200,000 \$/MW, and the capital recovery factor is 0.11, which is frequently used in Turkey.

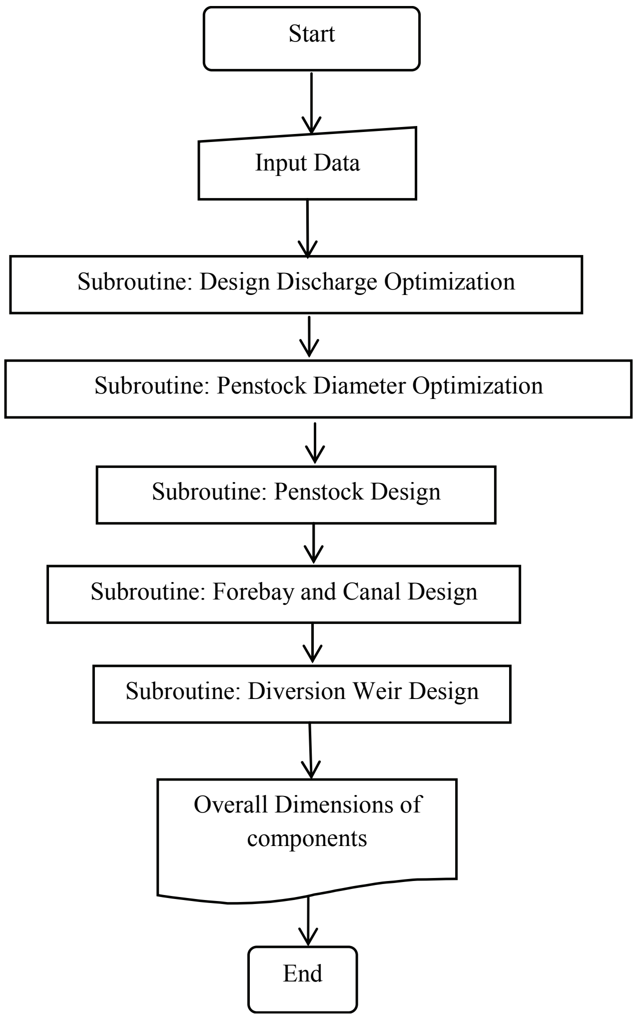


Figure 4. Flowchart of MINI HPD

Table 1. Input and output data of the program

Subroutine	Data type	Data
Optimum discharge and penstock diameter	Input	Flow-Duration curve, Gross head, Required unit cost data, Efficiencies of turbine, generator, transformer, Penstock length and its friction factor, Properties of penstock material and layout, Turbine closure time
	Output	Design discharge, net head, penstock diameter Installed power of the plant, Firm, secondary, total energy generation,
Penstock design	Input	Wave speed, increased pressure for water hammer, Gross head, Aeration pipe diameter
	Output	Penstock wall thickness
Forebay and canal design	Input	Penstock invert elevation, Required volume of forebay, Canal bed and side slopes, roughness coefficient
	Output	Forebay dimensions and water levels in forebay Trashrack width at the forebay intake, Side spillway and forebay crest elevations, Forebay bottom elevation, step height Canal dimensions and water surface elevations
Diversion weir design	Input	Length of canal, Number of piers and their details along the intake, Maximum diameter of particle to be settled, Geometric details of rackbars Thalweg elevation,
	Output	Water levels for each of the intake sections, Overall dimensions for each of the intake sections, Spillway crest elevation, K_s
Energy dissipater design	Input	Flood discharges, Riprap section water levels and bed elevation Spillway crest elevation and length Number of sluiceways and dimensions Geometric details of bridge on spillway crest
	Output	Discharge over spillway and through sluiceways Upstream water level for each flood discharge Type and dimensions of the energy dissipater

MINI-HPD is executed and the graphical representations of some of the calculations are presented in Figure 6. The optimum design discharge, Q_d, is obtained as 10.79 m^3/s. This value corresponds to 20% occurrence of time. Since the design discharge for run-of-river plants is recommended to be taken corresponding to 20 to 30% time of occurrence [2], the value of the design discharge is considered satisfactory. The optimum installed power and the corresponding optimum penstock diameter are calculated as 7.18 MW and 1.7 m (Figure 6), respectively. With this penstock size the average penstock velocity is 4.75 m/s, which is smaller than the maximum allowable value. Therefore, the design is acceptable. The outputs of the program are given in Table 3.

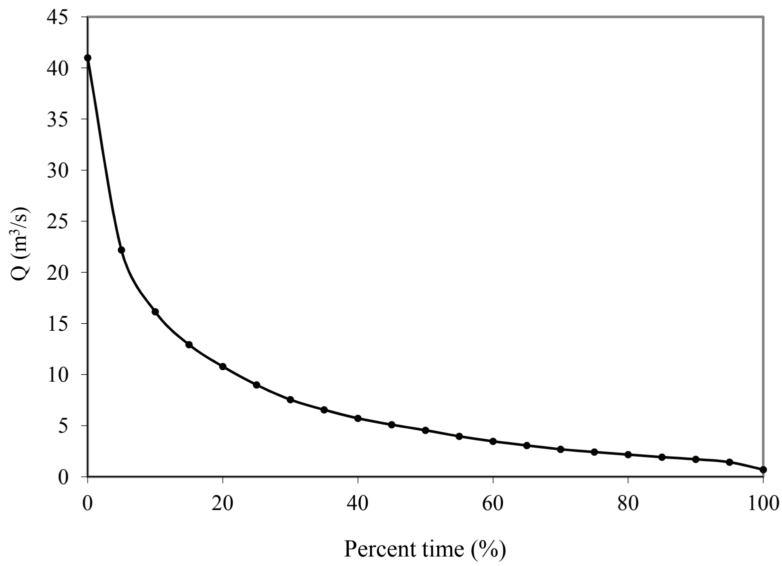


Figure 5. Flow-duration curve

Table 2. Flood discharges and water surface elevations at riprap section

Flood discharges	Discharge (m^3/s)	K_d (m)
Q_5	113.6	589.49
Q_{10}	139.7	589.85
Q_{25}	175.7	590.32
Q_{50}	205.2	590.68
Q_{100}	237.4	591.06

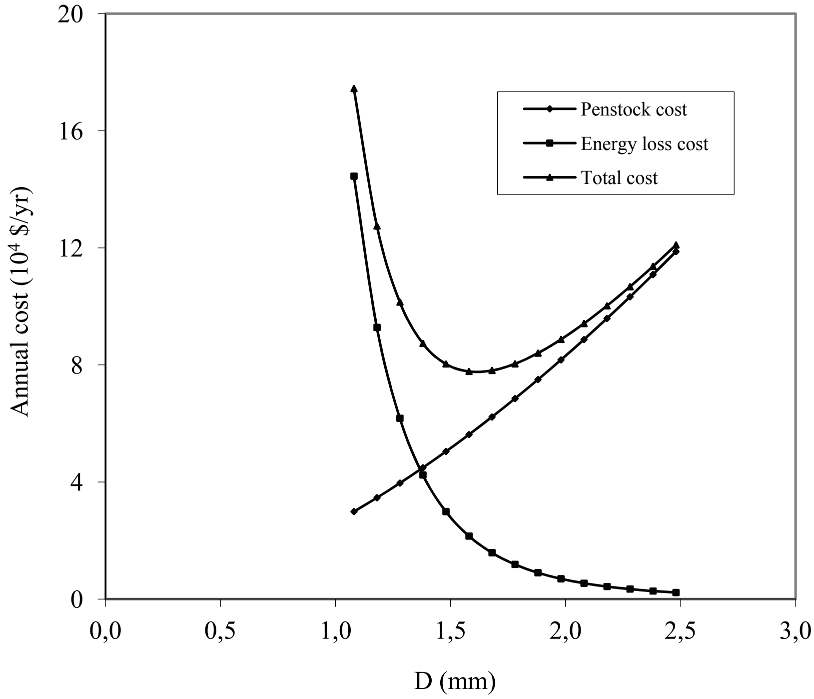


Figure 6. Determination of optimum penstock diameter

Table 3. The results of design

Parameter	Unit	Output
Design discharge	m^3/s	10.79
Penstock diameter, D	m	1.70
Crest elevation of spillway	m	593.24
Maximum water level for Q_{100}	m	594.44
Intake entrance bottom elevation	m	590.55
Settling basin length	m	33.76
Settling basin width	m	16.18
Velocity in settling basin	m/s	0.197
Forebay width	m	18.10
Forebay length	m	52.80
Minimum water level in the forebay	m	586.90
Normal water level in the forebay	m	587.66
Maximum water level in forebay	m	587.98

Table 3. Contiune

Parameter	Unit	Output
Forebay wall crest elevation	m	588.08
Penstock wall thickness	mm	8.00
Installed capacity	MW	7.18
Firm Energy	GWh/year	8.34
Secondary Energy	GWh/year	22.97
Total Energy	GWh/year	31.31
Energy dissipater length	m	31.00
Energy dissipater type	USBR	Type IV

5. CONCLUSIONS

In this study, a computer software called MINI-HPD is developed for the hydraulic design of run-of-river hydropower plants. The program starts execution by determining the optimum design discharge, optimum installed power, and penstock diameter. Afterwards the hydraulic design of structural components, such as forebay, transmission canal, spillway, sluiceways and energy dissipation facility are performed. The program runs with the assumptions that locations of the structures are pre-determined according to local and geologic requirements. Therefore, necessary local data are entered to the program as input. The program is user-friendly and is aimed to give practical guidelines to the user according to the commonly accepted criteria in Turkey. It performs interactive computations to result in time saving. The usage of the program is illustrated with an application.

Symbols

a = parameter as a function of particle size;

B = width of conveyance canal;

B_{fb} = width of forebay;

B_{tr} = width of the thrashrack in front of the intake of forebay;

B_s = width of settling basin;

C_c = contraction coefficient;

C_{ic} = cost of installed capacity;

C_{icu} = unit cost of installed capacity;

C_{pu} = unit cost of penstock;

CRF = capital recovery factor;

D = penstock diameter;

D_A = diameter of aeration pipe;

D_{max} = maximum diameter of particles to be settled in the settling basin;

E_{3s} = energy level at the downstream of the spillway;

E_{3sl} = energy level at the downstream of the sluiceway;

E_u = upstream energy level;

e = efficiency;

F_r = Froude number;

g = gravitational acceleration;

G = weight of penstock;

H_g = gross head;

H_n = net head;

H_s = steady state head;

h = water depth at the upstream of the spillway;

h_a = velocity head at the upstream of the spillway;

h_{fb} = difference between maximum and minimum water levels in forebay;

h_e = total height of in front of intake;

K_{cd} = bottom elevation at the canal exit;

K_d = water surface elevation at riprap section;

K_{100} = upstream water surface elevation for flood discharge Q_{100};

K_{d100} = water surface elevation at riprap section for flood discharge Q_{100};

K_{pi} = penstock invert elevation;

K_r = bottom elevation at riprap section;

K_s = crest elevation of spillway;

K_{sf} = crest elevation of side spillway in the forebay;

K_{st} = thalweg elevation at the entrance of intake;

L_{fb} = length of forebay;

ME = marginal energy;

P_p = power generated by the plant;

Q = discharge;

Q_d = design discharge;

Q_5 = flood peak discharge with a return period of 5 years;

Q_{10} = flood peak discharge with a return period of 10 years;

Q_{25} = flood peak discharge with a return period of 25 years;

Q_{50} = flood peak discharge with a return period of 50 years;

Q_{100} = flood peak discharge with a return period of 100 years;

Q_{eq} = equivalent discharge;

s = minimum depth of submergence;

t = wall thickness of penstock;

V = velocity;

V_{max} = maximum allowed velocity in settling basin;

V_{fb} = volume of forebay;

V_a = velocity in front of the thrashrack at the forebay intake;

W_f = fall velocity of a particle;

Δ_a = upward step height at the entrance of forebay intake;

Δ_s = end sill height at the stilling basin of spillway section;

Δ_{sl} = end sill height at the stilling basin of sluiceway section;

Δ_{sd} = downward step at the settling basin entrance;

Δ_{su} = height of upward sill at the end of stilling basin;

Δ_u = height of upward sill at the end entrance of intake;

ΔE = head loss due to the hydraulic jump;

Δt = equal time interval on flow-duration curve;

ΔQ = discharge difference between two consecutive time intervals;

ΔH = head increase due to water hammer;

σ_t = allowable tensile strength of pipe material.

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