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ORIGINAL ARTICLE

Simulation of Flow on Bottom Turn out Structures with Flow 3D

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ABSTRACT
Bottom intake is an intake structure in which can be considered an alternative for regular intakes in mountain streams. Especially it is suitable for small hydro-power. Low construction cost ,simple structure less maintenance cost and the only intake in which can divert 100 percent of river water are advantage of this structure, bottom intake consist of low high spillway in which is built along the river bed. A canal is constructed downstream in the main body of the spillway to collect the water and transport to one side of the river. On the top of the canal, a mesh consist of parallel steel bars is considered so the river bed sediment cannot be entered inside the channel. The most important part of the structure ,is the design of the area of the opening and the slope of the downstream face of the spillway .In this study to develop relationships for the discharge coefficient ,many different physical models were constructed in a flume of 60cm width and 8meter length .These models were tested under different flow conditions. The results have shown that for small slope the sediment can trap between the screen bars and reduces the area opening. Increasing the slope is reducing the sediment to trap but the flow depth over the screen reduces and thus the flow discharge reduces. However it was found that at slope of 30 degree, diverted flow discharge is high and diverted sediment is low. In this study new equation also way developed for discharge coefficient.
Key words: *bottom intake, track, screen, small hydropower, discharge coefficient.*

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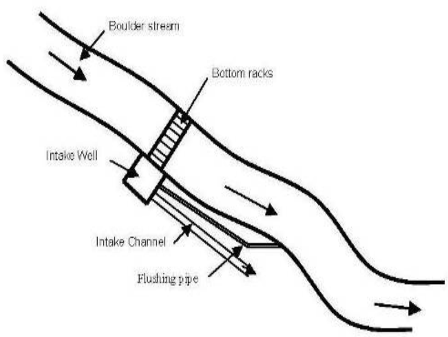
INTRODUCTION

Now days the needs for constructing small hydropower in the mountain area of Iran is increase. The water for generating electricity is supply from the high elevating the area. Because of rack of sufficient roads in this area, the intake must be simple from the construction point of view and requires less maintenance work after the construction. The structure also must be able to divert the desired flow discharge at all time even during the dry season. Bottom rack or bottom intake is the most suitable intake structure which can satisfy all the above mentioned criteria. Bottom intake is a simple structure which is consist of a channel on the river bottom vertical to the river flow and a screen on top of the channel of it is shown in fig. (1).



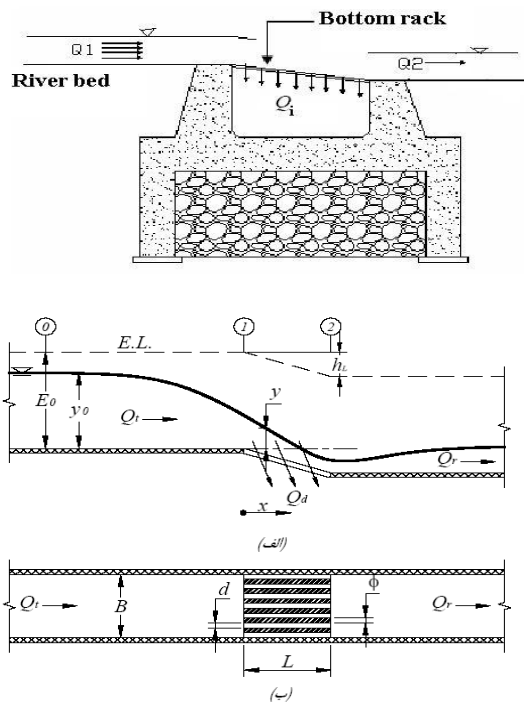


Fig (1) sketch of bottom rack structure

Move to one side of rive where there is a tunnel or open channel to transport the water to the penstock of hydropower. The screen should be sized so the riverbed sediment not to enter the channel while enough flow can be diverted. Although this structure have been on use for the past five decades, some of its design criteria for optimum design of this structure needs to be studied. The review of present knowledge is restricted to rectangular channels with an opening in the bottom made up of racks to divert sediment and to produce an intake structure for which sediment sizes larger than the bar spacing are excluded. Storm water inlets for road systems are thus not considered, not single bottom slots such as those used for combined wastewater systems. A first hydraulic description of bottom intakes was provided by orth et al [7] investigating flows on a 20% sloping channel with five different transverse rack geometries, including the simple. T, the t with a top triangle profile, the semicircular shape with a vertical bar, the full circular shape, and the avoid profile. The ovoid bar profile required the minimum structural length, where as the t- shaped bar had the poorest discharge performance. The bottom slope of the rack had only a small effect on rack clogging. Kuntzmann and bouvard [5] presented a first computational a0pproach for the free- surface profile over bottom racks by assuming constant energy head and a conventional orifice equation. The spatial distribution of discharge as a function of the stream wise coordinate resulted in a ordinary differential equation of the sixth degree, which was solved for the horizontal bottom rack. Racet- Madoux et al[8] presented general experiences on bottom intakes obtained by various projects in the savoy region of the French alps their general conclusions may be summarized as follows: (1) a knowledge of the water and sediment discharges is important for design; (2) the rack should consist of rounded profiles in the stream wise direction; (3) to obtain a minimum risk of sediment clogging, the bottom slope of the rack should be more than 20% and (4) a rack spacing of less than 0.10 m should be acceptable for mountainous regions. Bianco and Noseda [1,6] verified Noseda [6] observations with a larger model, and found no essential scale effects. Their bar profile was semicircular at the top with a rectangular bottom reinforcement. They considered ratios of intake to rack cross-sectional areas of 1/3 and 1/4. Their main data refer to intake discharge curves and the hydraulic features are not really accounted for. This review clearly demonstrates that most information available today refers to the flow above the bottom rack. Because structures with an almost 100% intake ratio are the hydraulic optimum, they deserve particular attention. Further, the energy line is not horizontal, especially close to the end of the rack where flow depths become small. The effect of rack inclination deserves further attention and the discharge coefficient for bottom racks was not thoroughly studied so far. Shafai-bejestan and shakourerad [10] conducted experimental tests. They developed the following equation for discharge coefficient. $C_d = 0.223\varepsilon^{-0.79} Fr^{-0.295} (\frac{\varphi}{L})^{0.054} (\frac{Y_1}{L})^{-0.0043}$.

In which eq . C_d , ε , Fr , φ , y_1 , L Are: coefficient discharge, area opening, froude number, bar diameter, flow depth, length of rack. Sandro Brrunella, Will H.Hager ,F.ASCE, and Hands-Erwin Minor[13] conducted experimental tests in a rectangular channel 0.5-m wide and 7-m long .Base on extended laboratory observation ,the effect of various parameters were explored ,such as the bottom slope ,the rack geometry ,and the rack porosity . In addition .a novel approach to determine the discharge coefficient of a rack structure was developed .Finally, the intake channel below the bottom rack was investigated and several interesting features were found, including a significant flow instability that may have a strongly adverse effect on the rack performance. Although it seems that bottom intake been studied in the past there of many question regarding the optimum design of the structure. There for it is the purpose of this study to investigate the effects of bottom rack area opening and slope on the flow discharge coefficient and to have contributed new idea for better design this structure.

Governing equations:

The governing equations for the flow in a river with a lateral outflow through a bottom screen are as follow:

Continuity equation:

$$-\frac{dQ}{dx} = Qi = \varepsilon c_d b\sqrt{2gE} \quad (1)$$

Energy equation:

$$\frac{dy}{dx} = \frac{so - sf - (Q/gA^2)(dQ/dx)}{1 - fr^2} \quad (2)$$

In this equation:

Q is the discharge in the main channel

Qi is the diverted flow discharge (passing through bottom rack)

ε is the ratio of the screen

Cd is the discharge coefficient

E is the specific energy of the flow over the screen and is the sum of the flow depth (y) and velocity head $(v^2/2g)$

Y is the flow depth on the screen

so is the screen slope

S_f is the slope of energy grade line

A is the flow area cross section in the main channel

f_r is the froude number which is defined as $v/\sqrt{gy}$ in which v is the flow velocity.

To determine the water surface profile over the screen, both equations must be solved. The analytical solution for them equations is possible through use of a few assumptions. The first assumption is that since the length a bottom rack generally is short, consequently the effect of channel and friction slopes on the flow profile can be assumed to be negligible. This assumption reveals that the value of specific energy (E) is constant. The second assumption is that the channel shape is wide and the discharge coefficient is constant. Applying these assumptions, and substituting Eq (1) into eh and calculating Q prom specific energy definition which is $Q = by\sqrt{zg(E-y)}$ and rearranging the terms in the resulting equation, one obtains:

$$\frac{dy}{dx} = \frac{z\varepsilon C_d\sqrt{E(E-y)}}{3y - 2E} \quad (3)$$

Integration of Eq. (3) yields:

$$x = -\frac{y}{\varepsilon C_d}\sqrt{1-\frac{y}{E}} + const. \qquad (4)$$

The constant of integration can be determined from the flow conditions at the Upstream of the bottom Rack which yields:

$$x = \frac{E}{\varepsilon c_d}\left(\frac{y}{\varepsilon}\sqrt{1-\frac{y_1}{E}} - \frac{y}{E}\sqrt{1-\frac{y}{E}}\right): \qquad (5)$$

Pts it can he seen the discharge coefficient is an important factor in determining the intake flow discharge (Eq.5) and water surface computations (Eq.2).

Dimensional analysis:

Before conducting experimental tests, a general relationship has to be developed. This can be done by using the dimensional analysis. The case of discharge coefficient (c_d) it can be shown that:

$$C_d = f(d, g, y, S, V, \varphi, \varepsilon, L) \qquad (6)$$

In which C_d = discharge coefficient; d= bar diameter; Fr = Froude number; ε= opening area; L = rack length; v = velocity; S= slope

By applying the π- theory, the non dimensional equation can be developed: During the experimental tests, the flow conditions were in fully turbulent flow therefore the effect of Reynolds number can be neglected therefore. Also in which Fr is approach Froude number. After simplification of above equation and eliminating the parameters with constant values in this study, one can obtain:

$$C_d = f(Fr_1, \frac{\varphi}{d}, \frac{y}{L}, \varepsilon, S)$$

$$C_d = f(Fr_1,, \varepsilon, S) \qquad (7)$$

Experimental setup:

The experimental set- up consists of a 60 cm wide flume the bottom screen installed at the center of flume. A pipe was connected to the bottom of flume to transport the diverted water into the sump A v-notch weir at the downstream end of the channel, measures the discharge and the discharge passing through the screen. Fig (2) is the sketch of the experimental setup.

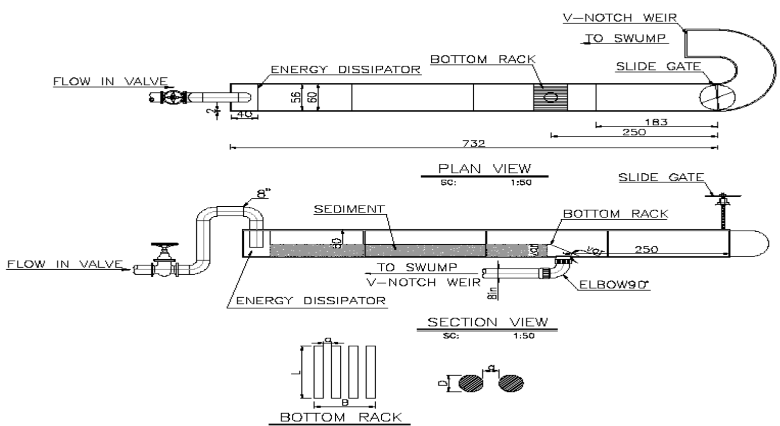


Fig (2) is the sketch of the experimental setup.

Experimental procedure:

After installing one of the bottom racks at the desired slope the flow was allowed to enter the flume by gradual opening of the entrance valve until the flow discharge reaches to the desired discharge. (See fig 3). This situation was kept constant for one hour during this time water surface elevation was measured the flume especially above the bottom rack. The diverted flow discharge also measured. Then flow discharge in the flume was in creased and the same variables ware measured. The same procedure was followed for three more discharge. Then the bottom rack was installed at a new slope and the above mentioned tests were repealed. There different slopes the above procedures was followed by installing a new model of bottom rack.

Fig (3) View of an experimental test

Therefore Six model of bottom rack with three different percent of area opening equal to 30,35,and 40 percent using two different sizes of bars equal to 6 and mm were tested. Each model was tested under three different slope and five different flow discharge. Table 1,2 are summery of the result and range of variable.

Table (1) shows summery of the results

test series	test No.	Bottom Rack			Q(L/s)	Qd(L/s)	Y1(cm)	C_d
		Dmm	E%	S(deg.)				
no sediment	A_1	8	0.3	10	12.5	12.5	2	0.46
	A_2	8			20.3	12	4.5	0.4
	A_3	8			29	12.4	7.6	0.4
test series	test No.	Bottom Rack			Q(L/s)	Qd(L/s)	Y1(cm)	C_d
		Dmm	E%	S(deg.)				
no sediment	A_1	8	0.3	20	12	12	2	0.46
	A_2	8			19.6	11.5	4.3	0.42
	A_3	8			26	12.7	6.5	0.41
test series	test No.	Bottom Rack			Q(L/s)	Qd(L/s)	Y1(cm)	C_d
		Dmm	E%	S(deg.)				
no sediment	A_1	8	0.3	30	10.1	10.1	1.8	0.52
	A_2	8			21.0	12	4.0	0.43
	A_3	8			25	12.4	6.1	0.42
test series	test No.	Bottom Rack			Q(L/s)	Qd(L/s)	Y1(cm)	C_d
		Dmm	E%	S(deg.)				
no sediment	A_1	8	0.3	40	12.5	12.5	2	0.38
	A_2	8			20.5	12	4.1	0.4
	A_3	8			25	12.4	5.6	0.4

Simulation

The main purpose of this study was to design an optimum bottom rack. This can be achieved by designing smaller size rack in such that the ratio of discharge which is passing through the rack is maximum. From dimensional analysis it was found that intake discharge depends on area opening and slope of rack as well as the froude number. Therefore to see how these parameters can change the intake. Discharge the discharge ratio (Qr= Qd/Q) was plotted versus ε ,S. Table (2) range of variables conducted in this study and fig4,5,6 is showing the simulation and modeling test.

Table (2) range of variables conducted in this study

Name of variable	**Notation**	**Range conducted**
Discharge	Q(L/S)	5-30
Size of rack's bats	D(mm)	6,8

Area opening of rack	$\varepsilon(\%)$	30,35,40
Slope of rack	S(degree)	10,20,30,40
Sediment size	Ds(mm)	2,5,7

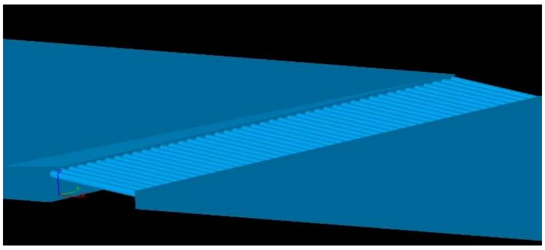


Fig.(4) view of an simulation

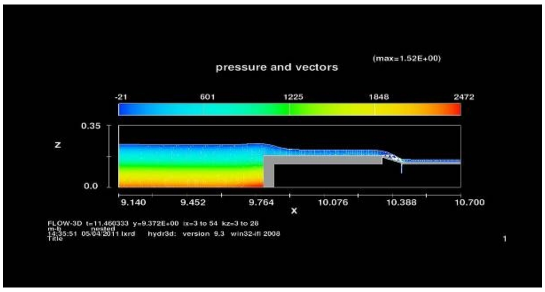


Fig.(5) view of an modeling test

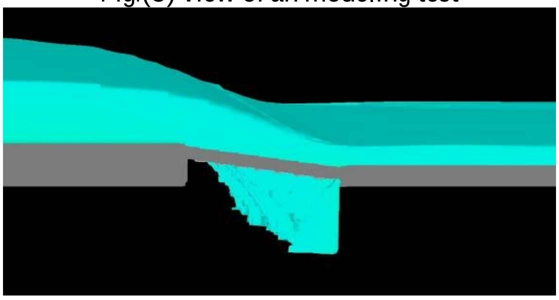


Fig.(6) view of an 3D modeling

RESULTS AND DISCUSSION

The value of discharge coefficient was calculated from the following equation using the measured data:

$$c_d = \frac{Q_i}{\varepsilon\sqrt{2gE}} \qquad (8)$$

In which E was computed from flow conditions just upstream of the bottom rack. Table (2) shows summery of the results. Values of C_d against S in experimental model for three different ε is showing fig (7) and comparing it with model show in fig 8. Also is showing the values of C_d against Fr1 for constant slopes in Fig9, 10. The summery of comparison of experimental and numerical model (ε =35%,Q=25L/S)) shows in Fig 11,12.

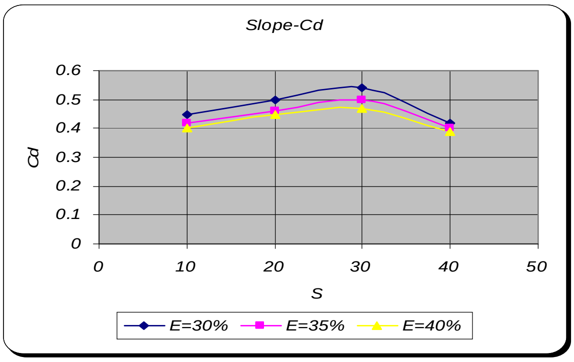


Fig.(7) Cd As function of slope

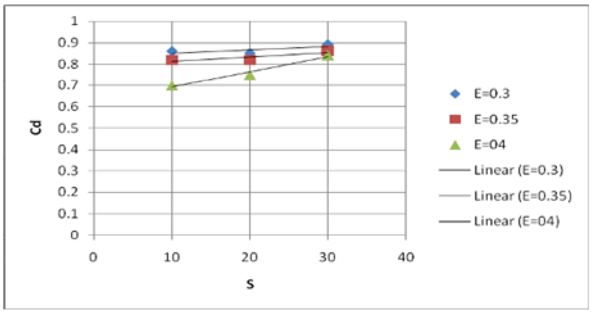


Fig.(8) Cd As function of slope

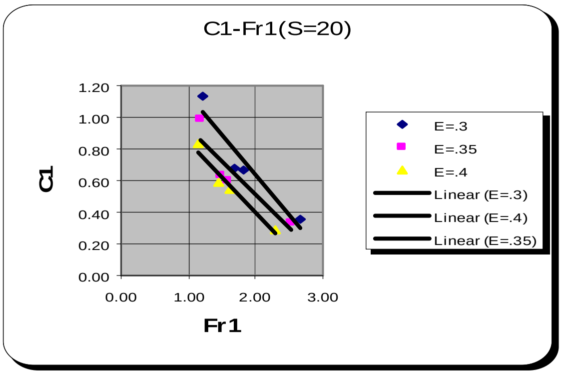


Fig.(9) Cd As function of Fr in slope 30^0, ε =30%

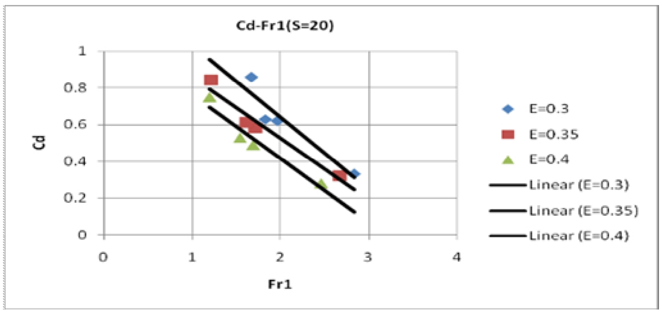


Fig.(10) C_d As function of Fr in slope

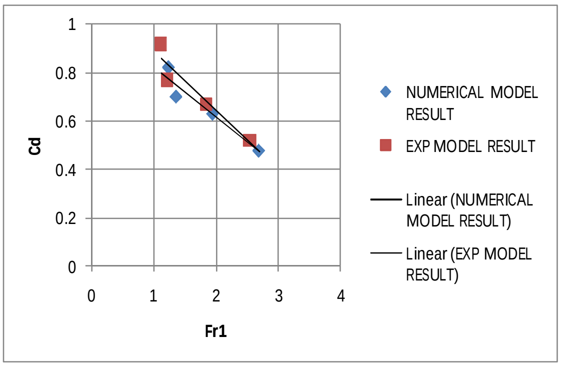


Fig.(11)comparison of result of experimental and model test

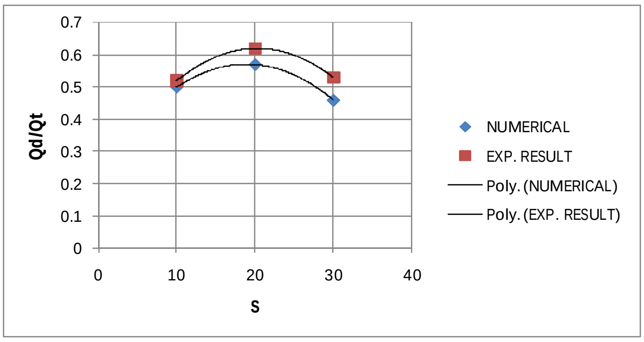


Fig.(12) comparison of experimental and numerical model(ε =35%,Q=25L/S))

CONCLUSION

By analysis of the data (bottom rack slope and area opening) it was found the best hydraulic performance can be achieved when the rack slope 30%and the rack opening is 40%.

Notation:

The following symbols are used in this paper: A= rack porosity; a= total efflux width; b= upstream channel width; C_d= discharge coefficient; D= rack drop number; d= bar diameter; F= Froude number; H= energy head; h= pressure head; L = rack length; Q= discharge; S= slope , α = rack angle

REFERENCES

1. Bianco, G., and Ripellino, p.g. (1994)" attualita delle opera di presa a traversa derivante e studio con modello idraulico di untipo di griglia suborizzontal " idrotecnica, 21(1), 3-12 (in Italian).
2. Escande, l. (1955). "prise d' eau a travers une grille nouveau complements d'hydraulique 2: 59-63 " publications scientifiques et tech niques du ministere de l air 302: paris (in French).
3. Kamanbedast, A.A. and Shafai-Bejestan. (2008). "Effects of Slope and area opening on the discharge ratio in bottom intake structures". Journal of Applied sciences, 8 (14), 2631-2635.
4. Kamanbedast.A, at all (2009). "Investigation of the hydraulic and sediment at bottom intake for hydro-power with physical modeling ". Int. conference CESSE 2009-VENICE ,ITALY
5. Kuntzmann, j. and bouvard, m. (1954) " etude theorique des grilles de prises d' eau du type en- dessous "la houile blanche 9(9/10), 569-574(in French).
6. Noseda, g. (1956). " Correnti permanenticonportata gressivamentse decrescente defluenti su griglie di fondo l energia elettrica 33(1), 41-51;33(6), 565-588 (in Italian)
7. Orth, J., E. Chardonnet, G. Meynardi (1954) "Etude de Grilles pour Prises d'eau du Type", en- dessous, La Houille Blanche, 9(6) , 343-351 (in French).
8. Ract- madoux, x., bouvard, m., molbert, j., and zumstein, j. (1955)."ouelques realizations recentes de prises en- dessous a haute altitudeen savoie la houille blanche 10(6), 852- 878(in French).
9. Sandro Brrunella,Will H.Hager ,F.ASCE, and Hands-Erwin Minor(2003)." Hydraulic of bottom rack intake", J. of Hydraulic Engineering, ASCE.
10. Shafai Bejestan, M. and A. Kammanbedast (2008). "Experimental investigation of the effects of sediment on discharge coefficient at bottom intake". 2nd Int. Junior Research on Hydraulic structures, university of Pisa, Pisa, Italy.
11. Venkataraman. P. (1977) discharges characteristics of an idealized bottom intake. "inst. Eng. India j. ci. 58(9/11), 99-104.
12. Venkatamaran. P., Nasser, m.s. and Ramamurthy, A.S. (1978). " flow behaviour in power channels with bottom diversion" 18th int. association of hydraulic research congress cagliary 2, 115-122
13. Vischer, D. L. and Hager, W.H. (1998) / dam hydraulics, wiley, chich ester.

14. Wahl, T. l (2001)/ hydraulics performance of coanda-effect screen .j.Hydraul.eng.ASCE.,127:480-488.

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