

RECENT ADVANCES IN THE DESIGN OF TRENCH WEIR

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INTRODUCTION

On a global scale, India stands fifth in terms of hydroelectric potential and a majority of the country's potential is yet to be tapped (about 88%). Northeastern region of the country accounts for more than a third of the total hydro potential in the country but only 2% has been developed so far. Maximum hydro potential has been tapped in west flowing rivers of south India, i.e., 58.01%. Technically feasible hydropower potential of top five countries is China-2,138,000 GWh/year; Brazil-1,300,000 GWh/year; Russia-1,670,000 GWh/year; Canada-981,000 GWh/year; India- 660,000 GWh/year. Small hydro potential in India is equal to 15000 MW with installed capacity 1423 MW in total 420 schemes. A further 187 schemes with a total installed capacity of 512 MW are under construction; and, 4096 additional schemes are planned (Hydropower & Dams 2004).

Most of the developments of small hydro power projects (SHP) are in hills due to the fact that there are large potential of SHP available in hilly regions in India. Hill streams carry large amount of heavy boulders rolling and moving along the bed along with smaller gravel and sand particles moving as bed load. Water is diverted from the streams to the intake channel of hydro power scheme using suitable diversion structures. Conventional types of raised-crest weirs are not well suited for hill streams. If such a weir is constructed across the stream, the rise in water level upstream of the weir brings a remarkable change in the flow conditions upstream. The sediments are deposited upstream of the crest as a result the intake gets easily choked up. Since diversions are interference to natural flow regime of the river, the extraction of water changes the flow conditions downstream of the intake. Undesirable sediment deposition occurs downstream of weir when too much water is extracted. Conversely, severe erosion downstream of the diversion occurs when remaining river flow passing downstream is carrying less sediment than its transport capacity. Moreover, any structural component that protrudes out of the river-bed (i.e., the top of a weir) gets damaged easily by the force of large sediments rolling down during floods (Paudyal and Tiwathai 1987). Figure 1a shows raised crest weir with upstream and downstream condition at the time of construction and Figure 1b the condition of weir after some year of its construction. Figure 1 clearly indicates deposition of boulders in the upstream and degradation in the downstream of the weir.

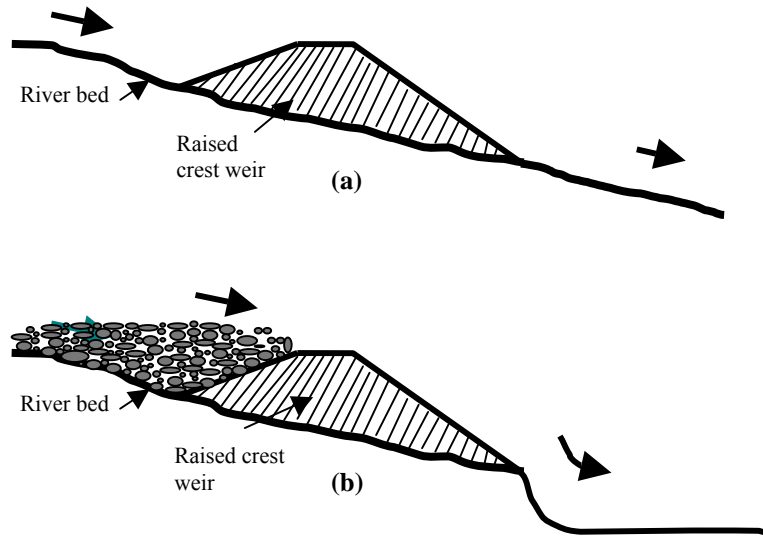


Figure 1: Condition of upstream and downstream of a raised crest weir (a) at the time of construction of weir; and (b) some year after the construction.

Kota raised crest weir on the river Dabka near Kaladungi (Nainital) was constructed in the year 1987 to divert water into an irrigation canal. The Dabka river is a boulder stream with a bed slope of about 3.6 % and carrying maximum size of boulder of the order of 1.2 m. Due to raised crest above bed level, the boulder starts accumulating upstream of the crest, which results in sediment deficient flow on the downstream side. Due to passage of such sediment deficient water on downstream, the bed levels go down as more and more sediment is picked-up by the flow. As a result, the undermining started in the first year itself and a breach developed in the third year (see Figure 2).



Figure 2: Breach in Kota Weir (Photograph taken in year 2002).

TRENCH WEIR

To overcome the above problems associated with raised-crest weir, the most common type of weir adopted in boulder streams for SHP and also for other uses like irrigation and water supply schemes etc is trench weir. A definition sketch of trench weir and its component is shown in Figure 3.

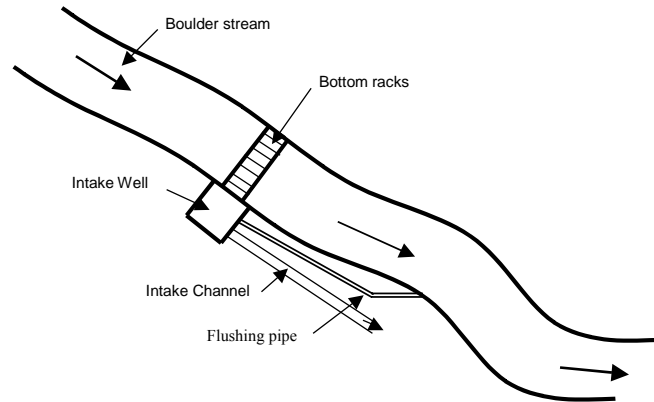


Figure 3: Definition sketch of a trench weir

It is simply a trench built across the stream below its bed level or at the elevated bed at the site (see Figure 4). Old practices were to build the trench on the elevated bed as shown in the Figure 4a (Stakna project, Leh, J&K – Dhillon et al.1986). However, such raising of bed poses the same problems, which are envisaged in the raised crest weir and hence not common. Earlier design of Sipit SHP in Arunachal Pradesh was on the elevated bed. After two- three years, a drop of about 7.5 m was formed in the downstream and finally the trench washed out in the middle portion and the remaining portion was found in the hanging position. Based on these observation and experience gained so far, it is recommended to provide the trench on the bed level of the stream as shown in Figure 4b.

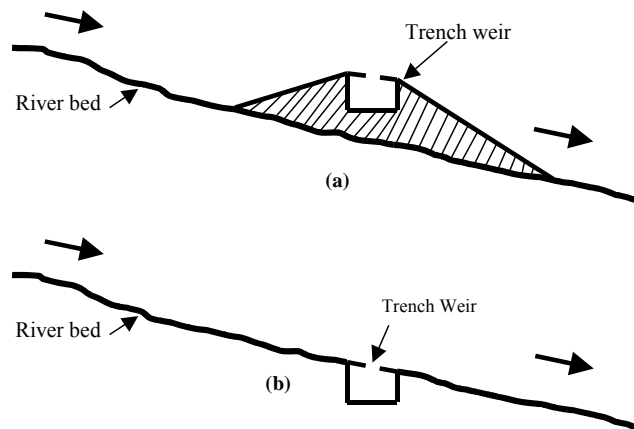


Figure 4: Fixing the position of the trench (a) Trench above the bed of the streams; (b) Trench on the bed of the streams.

The top of the weir is covered with bottom rack bars. Water while flowing over it, passes through the bottom racks and enters into the trench and collected in an intake well located at one of the banks at the end of the weir. The top edge of the diversion weir is almost flushed with the natural bed slope of the stream. The bottom racks consist of heavy rounded steel bars or flats laid on edge and placed parallel with the river flow on the bed level. The bottom rack bars are proportioned to carry the weight of the heaviest boulders that are likely to flow down during the maximum probable flood. This type of weir has the definite advantage as it does not affect the general bed level of the stream. Post-monsoon clearance of the boulders and debris collected in the trench of the weir is obligatory. In spite of the annual maintenance requirement, this type of weir has been widely adopted in the SHP schemes, built in Arunachal Pradesh, Sikkim, Himachal Pradesh, Uttaranchal and J & K.

HYDRAULIC DESIGN OF TRENCH WEIR

Hydraulics design of trench weir is consisted of fixing the dimension of the bottom rack, trench, intake well, flushing arrangement, protection works upstream and downstream of the trench etc. Commonly, the dimensions of different components are standardized on the basis of hydraulic model studies as well as certain empirical methods. Estimation of peak discharge in the stream is required to design the civil works. However, cross-section of the stream and resistance equation with appropriate value of roughness coefficient are needed to analyze the flow in the streams at its different stages. Following sections deal with computation of flow parameters and design of various components of a trench weir:

Estimation of Flow Parameters in Streams

Peak discharge in a boulder stream may be estimated from (a) known stage – discharge curve available at the site; (b) known highest flood level (HFL) using flow resistance equations; or (c) characteristics of the catchments and rainfall. Stage-discharge curve is rarely known for a boulder stream. However, HFL is easily known. Once maximum depth of flow is known, the peak discharge Q_p is calculated using the following Manning's equation:

$$V = \frac{1}{n} R^{2/3} S_0^{2/3} \quad (1)$$

where V = average velocity in the stream; n = rugosity coefficient; R = hydraulic radius; S_0 = bed slope. Manning's rugosity coefficient for boulder streams may be obtained from the equations given in the Table 1.

Table 1: Empirical equations for estimation Manning's rugosity coefficient for boulder streams.

| <i>S.No.</i> | <i>Investigator(s)</i> | <i>Equation</i> | <i>Remarks</i> |
|--------------|------------------------|---------------------------------|---|
| 1 | Golubtsov (1969) | $n = 0.222 S_0^{0.33}$ | data from central Asian rivers; $S_0 = 0.4 - 20\%$ |
| 2 | Anderson et al. (1970) | $n = 0.0395(d_{50})^{1/6}$ | data from flumes and natural rivers; $S_0 < 0.01$; d_{50} in ft |
| 3 | Bray (1979) | $n = 0.104 S_0^{0.177}$ | data from 67 gravel-bed rivers in Alberta; $S_0 = 0.02 - 1\%$ |
| 4 | Jarret (1984) | $n = 0.39R^{-0.16} S_0^{0.38}$ | for high gradient boulder streams |
| 5 | Bathrust (1985) | $n = 5.62 \log(y/d_{84}) + 4$ | y = depth of flow; d_{84} = size of boulders; british mountainous rivers; $S_0 = 0.004 - .04$ |
| 6 | Abt et al. (1988) | $n = 0.0456(d_{50}S_0)^{0.159}$ | d_{50} in inch; and $d_{50} = 26-157$ mm; $S_0 = 0.01 - 0.20$ |
| 7 | Rice et al. (1998) | $n = 0.029(d_{50}S_0)^{0.147}$ | d_{50} in mm; $S_0 = 0.1 - 0.4$ |

Use of the above equations depends upon the parameters available and flow, geometrical and sediment characteristics of the stream.

Relationship between maximum diameter of boulder available in the stream bed and average velocity (Garde and Ranga Raju 2000)

$$d_{\max} \text{ (m)} = 0.023 \text{ to } 0.046 V^2 \text{ (m/s)} \quad (2)$$

is also used to calculate average velocity and hence the peak discharge. It is better to estimate peak discharge using various methods and take the maximum of them as design discharge for trench weir for a conservative design. Flow in the stream at its different stages is analyzed using the Eq. (1) with appropriate value of rugosity coefficient.

Bottom Racks

Water while passing over the bottom rack; some enters into the trench due to gravity and remaining flow in the downstream of the trench in the streams. The flow in the stream with bottom rack is a case of spatially varied flow with decreasing discharge as shown in Figure 5. The rack is usually made of parallel bars placed along the direction of flow. Main variables involved in computing the diverted discharge into the trench are flow characteristics of the stream, geometry and dimension of the rack, free and submerged flow conditions etc.

The racks are placed inclined with reference to the approach bed of the river. The inclination of the rack, which is of the order of 1 in 10, is provided to facilitate easy movement of bed load of stream moving over the rack.

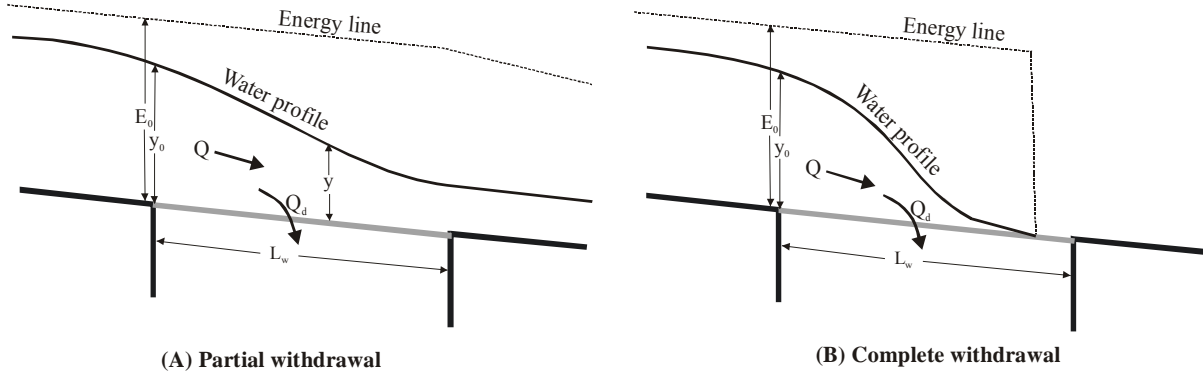


Figure 5: Flow over and through bottom racks

Discharge characteristics of bottom racks

Assuming the specific energy of flow to be constant all over the longitudinal bottom rack, Mostkow (1957) proposed the following equation for the diverted discharge into the trench:

$$Q_d = C_d \varepsilon B L_W \sqrt{2gE_0} \quad (3)$$

Where ε = ratio of clear opening area and total area of the rack; B = length of the trench; L_W = length of the rack bars; g = acceleration due to gravity; E_0 = specific energy at approach; and C_d = coefficient of discharge. Based on limited experimental study, Mostkow (1957) suggested that C_d varies from 0.435, for a sloping rack 1 in 5, to 0.497 for a horizontal rack. However, the influence of approach flow and rack geometry was not considered by him. Nosedá (1956) with an additional assumption of critical approach flow condition, analysed the flow over longitudinal racks and presented a design chart relating the diverted flow to the total stream flow. White et al. (1972) conducted model tests and compared the performance of bottom racks, having different lengths of bars, bar spacing and slope, with those predicted by Nosedá's method. They have shown insufficiency of Nosedá's design chart and suggested a different chart based on their studies, but their design chart is of limited application capability. On the basis of experimental study, Subramanya and Shukla (1988) and Subramanya (1990, 1994) classified the flows over horizontal and sloping racks of rounded bars, which are summarized in Table 2:

Table 2: Nature of flow over a bottom rack (After Subramanya 1990, 1994)

| Type | Approach | Flow over the rack | Downstream state |
|------|---------------|-------------------------|------------------|
| AA1 | Subcritical | Supercritical | May be a jump |
| AA2 | Subcritical | Partially Supercritical | Subcritical |
| AA3 | Subcritical | Subcritical | Subcritical |
| BB1 | Supercritical | Supercritical | May be a jump |
| BB2 | Supercritical | Partially Supercritical | Subcritical |

The functional relationship for the variation of C_d in various types of flows are as follows:

(a) *Inclined Racks* (Subramanya 1994):

$$(4) \quad \text{AA1 type flow} \quad C_d = 0.53 + 0.4 \log D/s - 0.61 S_L$$

$$(5) \quad \text{BB1 type flow} \quad C_d = 0.39 + 0.27 \log D/s - 0.8 V_0^2/2gE_0 - 0.5 \log S_L$$

(b) *Horizontal Racks* (Subramanya 1990):

$$(6) \quad \text{AA1 type flow} \quad C_d = 0.601 + 0.2 \log D/s - 0.247 V_0^2/2gE_0$$

$$(7) \quad \text{AA3 type flow} \quad C_d = 0.752 + 0.28 \log D/s - 0.565 V_0^2/2gE_0$$

$$(8) \quad \text{BB1 type flow} \quad C_d = 1.115 + 0.36 \log D/s - 1.084 V_0^2/2gE_0$$

Where D = diameter of rack bars; s = spacing of rack bars; S_L = slope of rack bars; and V_0 = velocity at approach. The energy loss over the rack is not significant in Type AA3 flows, however, it is significant in other types of flows. Brunella et al. (2003) proposed equations for coefficient of discharge for bottom racks of rounded bars for complete withdrawal of discharge.

From structural consideration, flat bars are preferred over rounded bars as flat bars have more flexural rigidity, i.e., flat bars are commonly used in the bottom rack in place of rounded bars. Ghosh and Ahmad (2006) studied experimentally the discharge characteristics of flat bars. They found that the specific energy over the rack is almost constant. Thus one can use Eqs. (3) for longitudinal bottom racks of flat bars also. They also compared C_d obtained for flat bars with C_d calculated by Subramanya's (1994) relationship, i.e., Eq. (4). Such comparison is shown in Figure 6. It is revealed from Figure 6 that the two sets of C_d values are different and Subramanya's relationship overestimates the value of C_d . It means that Eq. (4) that is proposed for rounded bars cannot be used for flat bars with approximation made above.

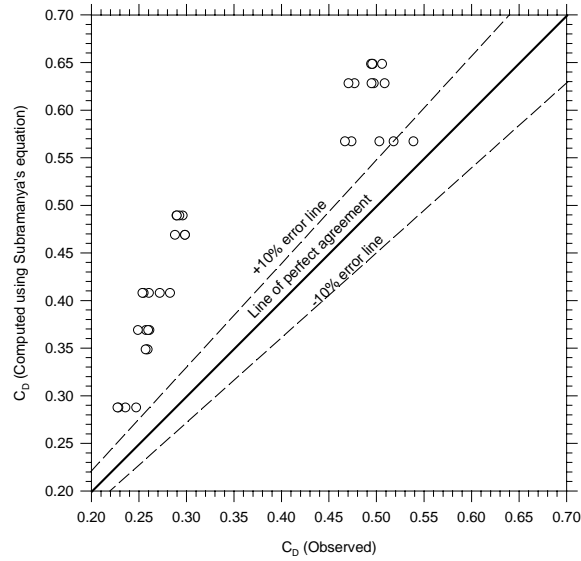


Figure 6: Verification of Subramanya's (1990) relationship for C_d

There is considerable effect of ratio of thickness of bars t and their clear spacing, i.e. t/s and inclination of bars, S_L on C_d . The value of C_d increases with increase of t/s ratio; however, it decreases with the increase of inclination of bars (S_L) for constant value of t/s ratio. Ghosh and Ahmad (2006) proposed the following equation for C_d for flat bars:

$$C_d = 0.1296(t/s) - 0.4284(S_L)^2 + 0.1764 \quad (9)$$

Equation (9) predicts the value of C_d for flat bars within $\pm 10\%$ error. Thus for the design of flat bars bottom racks Eq. (9) is recommended. Recent study at IIT Roorkee revealed that C_d for flat bar decreases with its flatness. Eq. (9) is proposed on the basis of limited data range, more experimental and field data are required to propose a better equation. Moreover, the above equations are applicable for free flow condition in the trench. However, if the discharge in the stream is more than the withdrawal discharge, the flow is submerged and Eq. (3) will be no more applicable. Flow characteristics of trench weir in submerged conditions needs to be studied. Trap efficiency of sediment in the trench is also not studied, so far. Experiments have been conducting at IIT Roorkee to study the above aspects of trench weir.

Once the C_d is known, the length of the bottom rack is calculated using Eq. (3). Ahmad and Mittal (2003) found that the optimum length of bottom rack (width of trench) is obtained when diverted discharge is equal to the incoming discharge in the stream.

Water surface profile over the bottom racks

Flow over the rack is spatially varied flow (SVF) with decreasing discharge. Thus one can solve equation of SVF for water surface profile. Height of the side walls of the trench weir are fixed on the basis of water surface profile. For a rack of parallel bars, the effective head on the

rack is approximately equal to the specific energy, and the energy loss is negligible (Ghosh and Ahmad 2006). The equation of the flow profile under this case is

$$x = \frac{E_0}{C_d \epsilon} \left(\frac{y_0}{E_0} \sqrt{1 - \frac{y_0}{E_0}} - \frac{y}{E_0} \sqrt{1 - \frac{y}{E_0}} \right) \quad (10)$$

Where x = distance along the channel; y = distance in the vertical direction; y_0 = approach depth of flow; and Q = discharge. Ghosh and Ahmad (2006) compared the water surface profile computed using Eq. (10) with the observed water surface profile for flat bars and found a satisfactory result.

Trench

Top width of the trench is taken equal to optimum length of the bottom rack. The shape of the trench may be rectangular or trapezoidal – shape is decided on the basis of economy. If the length of the rack is less and length of the trench is more, rectangular trench is preferred. However, in more length of rack, trapezoidal section is opted. Preferably trench across the stream is provided in the straight reach of the stream to avoid effect of curvilinear flow. The length of the trench is provided over whole width of the stream. If at the site stream width is different than the average width of the streams, the length of trench is provided equal to the average width of the stream with suitable training of stream in the upstream and downstream of the trench. Steep slope of the trench commonly 1:25 is provided to facilitate the movement of the trapped sediment in the trench to intake well. The depth of the trench is fixed on the basis of water surface profile in the trench.

Water surface profile in the trench

Flow in the trench is spatially varied with increasing discharge. For hydrostatic pressure and uniform velocity distributions, the most common equation of spatially varied flow in a prismatic channel is (French 1994).

$$\frac{dy}{dx} = \frac{S_0 - S_f - \frac{2\beta Q}{gA^2} q_d}{1 - \frac{\beta Q^2 T}{gA^3}} = F(x, y) \quad (11)$$

where S_f = energy slope; β = momentum correction factor; A = area of flow; T = top width; and q_d = diverted discharge in the trench per unit length. Computation of profile starts from the critical section. The location of critical section in the trench is given by (French 1994)

$$x_c = \frac{8q_d^2}{gT^2 \left[S_0 - \frac{gP}{C^2T} \right]^3} \quad (12)$$

where x_c = distance of critical section; P = wetted perimeter; C = Chezy's coefficient. To locate the critical section; assume some value of x and calculate A , P , T and C corresponding to the critical section. Substitute these values in Eq (12) and calculate x_c . If $x_c = x$ then O.K., otherwise another value of x is selected and the procedure is repeated till $x = x_c$. Water profiles in the either direction of control section is computed using Eq. (11). If the location of critical section is beyond the downstream end of the trench, the downstream water level in the intake well governs the upstream water level in the trench. In general, Runge-Kutta Method is employed to calculate the water surface profile:

Runge-Kutta Method: Starting from known value of y_i , the depth of flow y_{i+1} at Δx distance is calculated from Eq. (11) by

$$y_{i+1} = y_i + \frac{1}{6} [k_1 + 2(k_2 + k_3) + k_4] \quad (13)$$

$$k_1 = \Delta x F(x_i, y_i)$$

$$k_2 = \Delta x F(x_i + \Delta x / 2, y_i + k_1 / 2)$$

$$k_3 = \Delta x F(x_i + \Delta x / 2, y_i + k_2 / 2)$$

$$k_4 = \Delta x F(x_i + \Delta x, y_i + k_3)$$

Intake Well and Flushing Pipe

Water passing through the racks in the trench meets normally to the flow of water in the trench. This causes generation of vortex flow in the trench, which goes like swirling flow into the intake well. Thus sufficient space in the intake well be provided to stabilized the flow. In general length of the intake well is taken equal to 2-3 times of the width of the trench and width of the trench 2 times the width of the trench. Sufficient space is also provided for accumulation of sediment in the intake well. In general bed level of intake well is fixed at 0.5-1.0 m lower than the bed level of the trench at the intake well. Sufficient space is also required for installing the gates. Water level in the intake channel is fixed 0.15 m below the water level in the intake well to provide sufficient head for flow into the channel.

About 5% of design discharge of trench weir is taken as flushing discharge in the flushing pipe. Generally 4 to 5 m/s velocity in the flushing pipe is sufficient to flush the sediment. Several relations for the estimation of non-deposit velocity in the pipe are available in the literature, which can be used to calculate more accurate value of velocity. One of such relation

is (Garde and Ranga Raju 2000):

$$\frac{V_L}{\sqrt{2gD_p(\rho_s/\rho_f - 1)}} = 1.51C_v^{0.105}d^{0.056} \quad (14)$$

Here V_L = Non-deposit velocity; ρ_s , ρ_f = mass density of sediment and fluid, respectively; D_p = diameter of pipe; C_v = sediment concentration by volume; and d = size of sediment in mm.

Invert level of flushing pipe at its outfall is kept lower than the invert level of pipe at its inlet, so that, in the no flow condition undesirable silting should not occur in the pipe. Flushing of sediment is done in high flow condition in the streams. Under such condition, head available H to flush the sediment is equal to difference of HFL in the intake well and HFL in the streams at the outfall of the pipe and given by

$$H = 0.5 \frac{V^2}{2g} + \frac{fL}{D} \frac{V^2}{2g} + \frac{V^2}{2g} \quad (15)$$

Length of pipe and its diameter is selected such that they satisfy the Eq. (15).

Protection Works

Trench weir has a definite advantage as it does not change the general contour of the stream bed. As an added protection to such weirs, bed pitching in concrete blocks and/or boulders in wire crates and retaining walls on the sides are provided for at least 15 m to 20 m upstream and downstream of the trench. Boulders should be structurally strong enough to sustain the impact of the boulders. Recent trend is also to provide granite layer on the boulders surface to minimize the abrasion. Some stones filled sausages are required additionally to provide the necessary protection in downstream during high floods. The minimum size for stone of relative density 2.65, used for protection works, is calculated using Eq. (2). It is recommended that the higher value of the coefficient in Eq. (2) is adopted in the design.

Validity of Lacey's scour depth equation to fix up the cut-off of the structure in mountain stream is questionable. The cut-off depth is provided more than the calculated by Lacey's equation (Paudyal and Tawatchai, 1987).

CONCLUSIONS

Conventional raised-crest weirs across the streams to divert the water into the intake channel are not suited for boulder streams due to problems outlined in this paper. The most common type of weir adopted in the boulder streams for SHP and also in irrigation and water supply schemes is trench weir. It has a definite advantage as it does not change the general contour of

the stream bed and no structural component of it protrudes out of the river-bed. Trench weir on the elevated bed is not recommended. Recent advances in the design of different components of a trench weir like trench, bottom racks, intake well, flushing pipe etc are discussed in this paper. Authors have designed a number of trench weirs using these design aspects of the weir. Based on the experience gained in field and research related to trench weir, authors recommend the methodology discussed in paper for the design of trench weir.

A further study related to discharge characteristics of flat bars bottom racks, free and submerged flow condition and sediment trap efficiency of the trench weir is needed. Experiments have been conducting at IIT Roorkee to study these aspects of trench weir.

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