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Economic and efficient method of design of a flumed canal fall

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ABSTRACT

Innumerable falls are to be provided in irrigation canals where ground slope exceeds the permissible bed slope of a canal. In the conventional method of design, fluming ratio is fixed arbitrarily irrespective of inflow Froude's number. Long lengths of inlet and outlet transitions are provided to prevent flow separation. Transition and dissipation structures are kept separate resulting in high costs. Hydraulic performance of the conventional fall structure is also not so satisfactory. Analytical and experimental studies were conducted by the author to find an efficient and economic method of design of falls. Optimum fluming ratio and optimum length of transitions are found both for economy as well as efficiency. An efficient and economic stilling basin with rapidly diverging side walls and adversely sloping floor, which act simultaneously as energy dissipater and transition, has been recommended. An example has been worked out to illustrate the design procedure of the proposed canal fall.

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KEYWORDS
Canal fall; fluming ratio; transition; energy dissipation; hydraulic efficiency

1. Introduction

Canal falls are needed for negotiating steep terrain slope. Falls are used as control structures to regulate flow depth, maintain depth–discharge relation, flow diversion, flow measurement, etc. They are often combined with local communication bridges and cross regulators. In unlined canals where canal width is large, they are usually flumed to reduce cost. In all such flumed canal falls, pair of transitions is to be provided both upstream (Contracting Transition) and downstream (Expanding Transition) of the flumed section for smooth flow at entry and exit of the fall. They are to be invariably provided with energy dissipaters to avoid erosion downstream. In the design of a fall in an unlined canal, it is customary to flume the canal by restricting the normal waterway. Extent of fluming will be governed by Froude's number of incoming flow (F_1) and the desired value of Froude's number of flow (F_0) in the flumed section (Mazumder and Ahuja 1978). Any fluming beyond a critical limit (known also as choking limit when $F_0 = 1$) will cause excessive afflux resulting in a long backwater reach where the canal regime and proportionality of flow condition, i.e. the normal depth–discharge relation, are lost (Mazumder and Deb Roy 1999). Cost of connecting the flumed section with the normal canal section by providing classical transition structures is excessively high. The demerits of conventional design of energy dissipaters, inlet and outlet transitions with long length and complicated shapes have been discussed elsewhere (Mazumder 1967). In this paper, the author has suggested an innovative, economic and hydraulically efficient design of a canal fall by employing recent advances in hydraulics.

2. Development in fall design

Depending upon discharge and height of fall, different types of falls have evolved over time. Different types of canal falls, also called drops (USBR-78, Garg-13, IIT-Mod.3), developed over time are:

(a) Sarda-type fall – developed by Sarda project authorities in UP
(b) Inclined Straight Glacis-type Fall with USBR Type-III Stilling Basin
(c) Flumed Curved Glacis Fall with USBR Type-II Stilling Basin
(d) Trapezoidal notch-type fall with Proportional Flow device
(e) Stepped Fall
(f) Well-type fall – with horizontal pipe connecting two wells
(g) Pipe Fall – with inclined Pipe in which hydraulic jump occurs
(h) Baffle-type fall – Flow from an inflow conduit to a Canal

Type of fall to be adopted in a given situation is dependent largely by the height of fall and the discharge. Types (a), (b) and (c) can be used for all discharges and any height of fall. Type (d) is popular where proportionality of flow upstream is to be maintained. Types (e), (f), (g) and (h) are provided where discharge is small. A conventional fall of type (c) is illustrated in Figure 1(a). Detailed design procedures of these falls are available in standard text books (Arora 1996; Aswa 1993; Mazumder 2007). Most of these designs have been developed by project authorities based on the local requirement and knowledge available at the time. The design proposed in this paper is based on the recent developments in hydraulics and the laboratory experiments carried out by the author and the young students working with him with a view to economise the cost and at the same time making the design hydraulically more efficient.

The proposed fall, as shown in Figure 1(b), can be used in place of types (a), (b), (c) and (d) in unlined canals where the mean velocity of flow has to be restricted to avoid erosion. Such falls are invariably flumed for achieving economy. In the conventional design (Figure 1(a)), flume extends up to the

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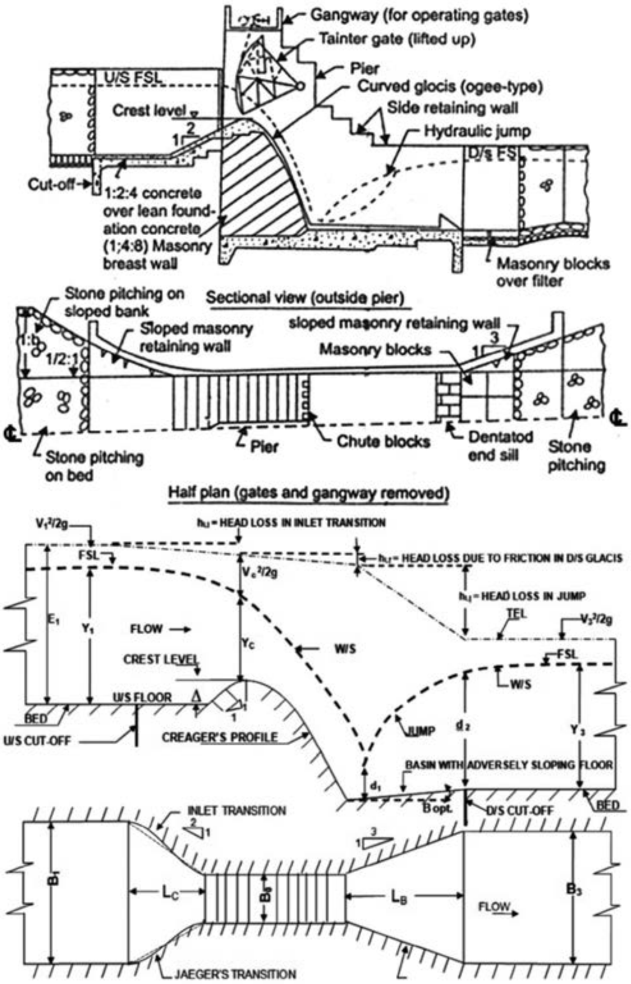

Figure 1. Conventional design of canal fall (type-c). Proposed design of canal fall (in place of types a, b, c and d).

end of stilling basin followed by expanding transition. This makes the fall very costly since the entire length from entry of contracting transition to the exit of expanding transition must be paved and the pavement has to be thick to resist uplift pressure. Long retaining walls are to be provided on either side to prevent slope failure. In the proposed design (Figure 1(b)), the stilling basin is provided with expanding side walls right from the toe of glacis so that no separate expanding transition is needed. There is considerable reduction in the length of paved floor and retaining walls. The proposed fall will not only be economic, it will be hydraulically more efficient when compared to the conventional ones as discussed in the following paragraphs.

3. Hydraulic aspects of fall design

In this section, the different hydraulic aspects of the proposed design of fall in regard to fluming, flow regime, transition and energy dissipation are discussed briefly with a worked out example at the end (Appendix 1) to help the designers.

3.1. Hydraulics of fluming

If B_1 and B_0 are the mean widths of flow at the normal and flumed sections of a canal fall, respectively (Figure 1(b)), it can be proved that the fluming ratio (B_0/B_1) may be expressed as:

$$B_o/B_1 = (F_1/F_o)\left[(2+F_o^2)/(2+F_1^2)\right]^{3/2} \tag{1}$$

where F_1 and F_0 are the Froude's number of flow at the normal and flumed sections, respectively. Figure 2 shows the functional relation between B_0/B_1 and F_0 given by Equation (1) for different values of F_1 for approaching flows. It may be seen that higher the F_1 value, less is the opportunity of fluming to avoid flow choking. F_1 values indicated in the figure were found corresponding to mean width of flow (B_1) for four different canals with varying bed slope and discharge. It may be observed (Figure 2) that there is hardly any advantage/economy if fluming is made such that F_0 exceeds approximately 0.70. Also, flow surface starts becoming wavy when F_0 exceeds 0.70, with highest degree of wave amplitude at critical flow at $F_0 = 1$. Excessive fluming causes high loss in head due to high velocity of flow at the flumed section resulting in large afflux.

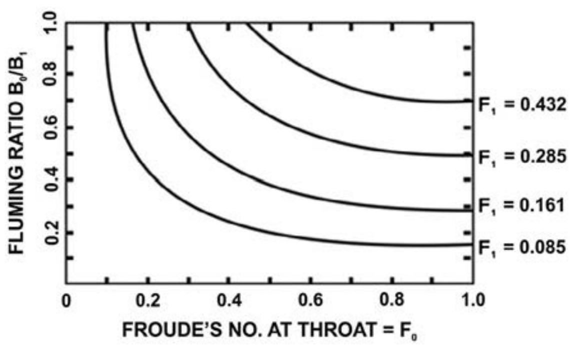

Figure 2. Showing interrelation between F_1, F_0 and B_0 / B_1.

3.2. Flume width (B_0) and crest height (Δ) to maintain depth–discharge relation

Canal falls are control structures which can be used also for measuring flow through the canal. In case the fluming is too high, crest height above the canal bed, Δ (shown in Figure 1(b)) will be low. On the other hand, if fluming is too low, the crest height will be more. An optimum width of throat (B_0) and corresponding crest height (Δ) were determined theoretically (Mazumder and Deb Roy 1999) such that the proportionality of flow can be maintained and there is negligible afflux. Equation (2) gives the optimum width at throat (B_0) and Equation (3) gives the corresponding crest height (Δ) for maintaining proportionality of flow for all discharges passing through the canal.

$$B_0 = \left[0.7\left(Q_{\max}^2 - Q_{\min}^2\right)/\left(E_{1\max} - E_{1\min}\right)\right]^{3/2} \tag{2}$$

$$\Delta = E_{1\max} - 3/2\left[\left(Q_{\max}/B_0\right)^2/g\right]^{1/3} \tag{3}$$

where $Q_{\max}$ and $Q_{\min}$ are the maximum and minimum flow through the canal; $E_{1\max}$ and $E_{1\min}$ are the corresponding maximum and minimum specific energies of flow given by:

$$E_{1\max} = Y_{1\max} + V_{1\max}^2/2g \text{ and } E_{1\min} = Y_1 \min + V_{1\min}^2/2g \tag{4}$$

Here, $Y_{1\max}$ and $Y_{1\min}$ are the normal flow depths and $V_{1\max}$ and $V_{1\min}$ are the mean velocities of flow in the canal upstream of the fall corresponding to $Q_{\max}$ and $Q_{\min}$, respectively. The various symbols used in the equations, the flumed fall, the inlet and outlet transition structures, etc. are illustrated in Figure 1(b). An illustrative example has been worked out in Appendix 1.

3.3. Ogee-type glacis to prevent flow separation

Ogee-type profile (USBR 1968) may be adopted for the downstream glacis to ensure smooth flow over the glacis free from any separation. Coordinates of ogee profile can be obtained from Equation (5) with crest as origin.

$$Y/H_0 = K\left(X/H_0\right)^n \tag{5}$$

where X and Y are the coordinates at any point on the profile, H_0 is the energy head above crest, K and n are coefficients governed by approach velocity head and shape of upstream geometry of the profile. K and n values can be obtained from the text book, 'Design of Small Dams' (USBR 1968).

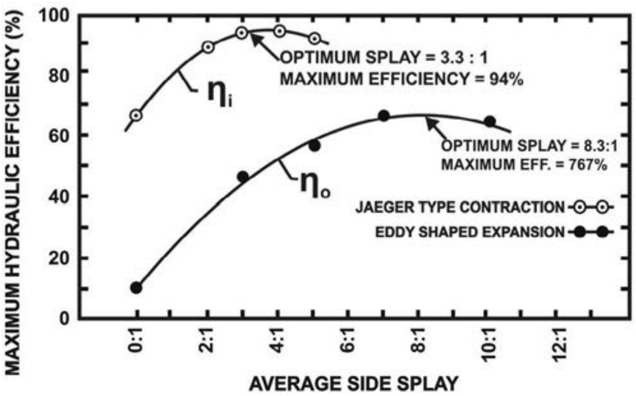

Figure 3. Variation of efficiency in contracting and expanding transition.

4. Design of contracting and expanding transitions

As stated earlier, these transitions, shown in Figure 1(a) and (b), are to be provided for smooth flow at entry and exit of flumed section and to avoid flow separation and consequent head loss.

4.1. Contracting transition

As shown in Figure 1(a) and (b), contracting transition connects the normal section with the flumed section. In a contracting transition, potential energy is converted to kinetic energy of flow. Afflux upstream of a fall is governed by the head loss in the contracting transition. More the head loss, more will be the afflux. Relation between head loss and inlet efficiency (η_i) in a contracting transition can be expressed as:

$$\eta_i = 1/\left(1 + C_i\right) \tag{6}$$

where C_i is inlet head loss coefficient given by the relation:

$$C_i = h_{\mathrm{Li}}/\left[\left(V_c^2 - V_1^2\right)/2g\right] \tag{7}$$

where h_{Li} is the loss in head in the inlet transition, V_c and V_1 are the mean velocities of flow at crest and normal sections of the canal upstream of the fall, respectively.

Different shapes of contracting transitions have been proposed by several research workers from time to time (Chaturvedi 1963; Garde and Nasta 1980; Hinds 1928; Mazumder 1977, 1979; Mitra 1940; Swamee and Basak 1992; Vittal and Chiranjeevi 1983). Shape of Jaeger (1956)-type transition is defined by Equations 8–12 given below. Figure 3 shows the hydraulic efficiency (η_i) of Jaeger-type contracting transition having different axial lengths governed by average rate of flaring varying from 0:1 to 5:1 (Mazumder and Ahuja 1978).

$$V_x = V_1 + a\,(1 - \mathrm{Cos}\Phi) \tag{8}$$

$$\Phi = \pi x / L_c \tag{9}$$

$$y_x = y_1 - a/g\left[\left(a + V_1\right)(1 - \mathrm{Cos}\Phi) - 1/2\, a\, \mathrm{Sin}^2\Phi\right] \tag{10}$$

$$a = \tfrac{1}{2}\left(V_0 - V_1\right) \tag{11}$$

$$V_x B_x y_x = Q = V_1 B_1 y_1 \tag{12}$$

Where V_x, y_x and B_x are the mean velocity, flow depth and mean flow width at any distance 'x' from the beginning of inlet transition, L_c is the axial length of inlet contracting transition and V_0 is the mean flow velocity at throat/flumed section at the exit of inlet transition. Mean width of flow section (B_x) at any axial distance 'x' from entry of inlet transition can be found from the continuity Equation (12). An example has been worked out to illustrate the design procedure (Appendix 1).

4.2. Expanding transition

A pair of symmetric expanding transitions is to be provided for connecting the flumed section with the normal canal section as illustrated in Figure 1(a) and (b). In conventional designs (Chaturvedi 1963; Garde and Nasta 1980; Mitra 1940; Swamee and Basak 1992; Vittal and Chiranjeevi 1983), expanding transition starts from the end of classical stilling basin and ends in the normal canal section. This is necessary since the sub-critical mean velocity of flow [$V_2 = Q / (B_o \times d_2)$] at the exit end of basin (after hydraulic jump) is substantially higher than the normal mean velocity (V_3) in the canal. Main function of the expanding transition is to diffuse the sub-critical flow from V_2 to V_3 so that there is no scour in the tail channel downstream of the fall.

The sub-critical flow in an expanding transition is subjected to an adverse or positive pressure gradient. Flow separates if the axial length of transition is insufficient. It has been established (Gibson 1910; Kline and Cochran 1958) that if the total angle of expansion exceeds about 10°–12°, flow will separate from the boundary resulting in poor outlet efficiency ($\eta_0 = 1 - C_0$) and non-uniform distribution of velocity at the exit end of expansion. Mazumder (1972, 1977, 1979) tested eddy-shaped (Ishbash and Lebedev 1961) expanding transition of different lengths and found that for maximum hydraulic efficiency, the axial length of transition must be about 78 times the offset [1/2 ($B_3 - B_0$)] in order to ensure uniform flow at the end of transition. Figure 3 shows the variation of efficiencies in expanding transitions with different axial lengths governed by average side splays varying from 0:1 to 10:1.

4.2.1. Control of boundary layer separation

Design of an expanding transition in sub-critical flow is essentially a problem of boundary layer separation control. Mazumder and Rao (1971) developed short triangular vanes to control flow separation in a wide angle straight expansion and achieved very high hydraulic efficiency (η_0 = 75%) and highly uniform velocity distribution at exit of expansion (α_3) varying from 1.08 to 1.12, which is almost the same as in a normal flow. Here, α is the kinetic energy correction factor (Corrioli's coefficient) given by the relation:

$$\alpha = \left[1/AV^3\right] \sum u^3 dA \qquad (13)$$

where u is the velocity of flow through an elementary area dA of the flow section and V is the mean velocity through the flow section of area A. When u = V (i.e. an ideal flow with uniform velocity everywhere in the section), α = 1.0. Since velocity is never uniform, α_3 at exit of expansion is always greater than unity. The objective should be to obtain low value of α_3 in order to achieve flow uniformity at exit of expansion and reduce scour in the tail channel. α value for normal flow usually varies from 1.03 to 1.36 (Chow 1973).

5. Design of stilling basin

Stilling basin is provided to dissipate the kinetic energy of flow within the basin. In a classical basin, width of the basin is kept the same as the width of flumed fall (B_0) up to the end of the basin, length of which is usually fixed by the length of a classical hydraulic jump in the rectangular basin. The basin length varies from 4 to 6 times the conjugate depth (d_2) depending on the type of stilling basin determined by F_{t1} and Ut_1 values, where U_{t1} and F_{t1} are the mean velocity and Froude's number of flow at the toe of downstream glacis, respectively. Further details of design of classical hydraulic jump-type stilling basins are given in several text books on hydraulics. (Chow 1973; Hager 1992; Ranga Raju 1993; USBR 1968).

In the conventional design of a canal fall, as indicated by dotted line (in plan) in Figure 4, the cost of stilling basin followed by a classical expanding transition is exorbitantly high. Using different types of appurtenances (like vanes, bed deflector and basin blocks) for preventing flow separation, Mazumder and Sharma 1983, Mazumder and Naresh 1988) developed a unique stilling basin with rapidly diverging straight side walls having axial length equal to three times the offset, i.e. 3 (B–b), as shown in Figure 4. The basin functions both as energy dissipater and flow diffuser simultaneously. Without appurtenances, there will be violent separation of flow and highly non-uniform flow at the exit end of the basin. With appurtenances, there is high hydraulic efficiency and the flow becomes highly uniform at the exit.

To reduce the additional cost of appurtenances, Mazumder (1987a,b, 1994) developed an innovative method of boundary

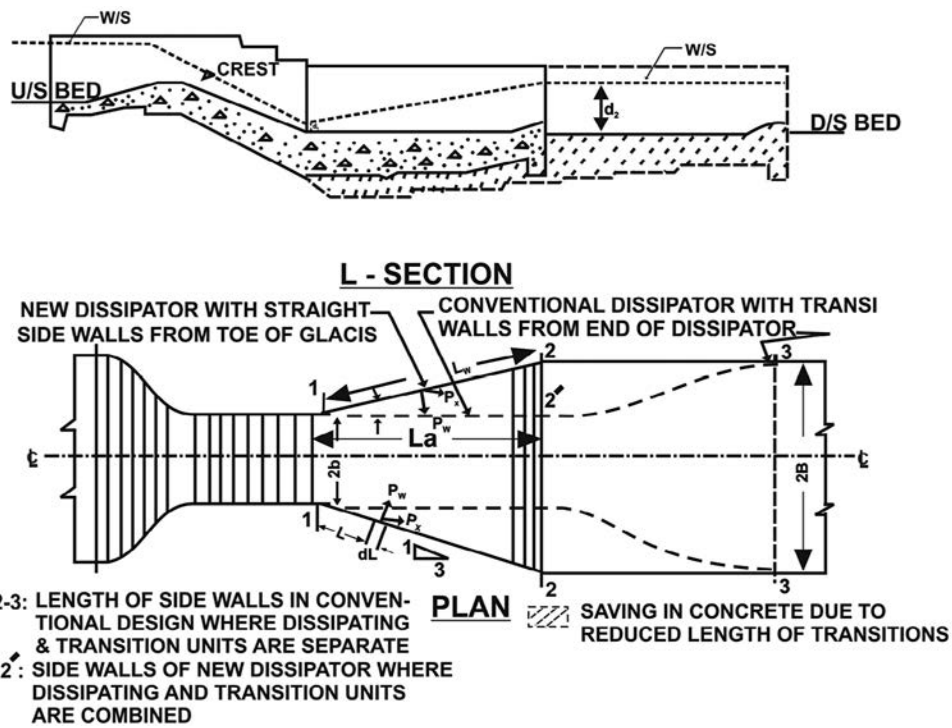

Figure 4. Plan and sectional view of a canal fall showing proposed design (full line) and the conventional design (dotted line).

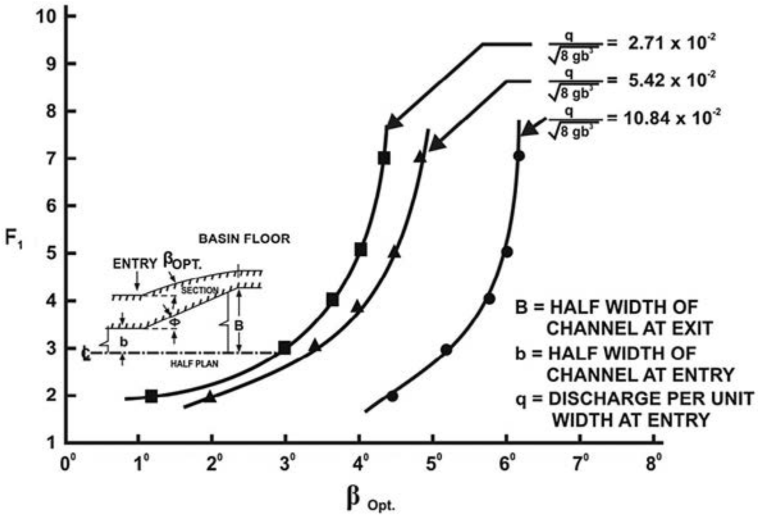

Figure 5. Optimum inclination of basin floor (β_{opt}.) for different values of pre-jump Froude's number $F_{1.}$

layer flow control by providing adversely sloping basin floor. Optimum value of inclination of basin floor (β_{opt}) corresponding to a given angle of divergence of the side wall (Φ), as indicated in Figure 5, can be expressed as:

$$\beta_{opt} = \tan^{-1}\left[\left(d_1^2 + d_2^2 + d_1 d_2\right)\tan\Phi \,/\, \left(b\,d_2 + Bd_1 + 2bd_1 + 2Bd_2\right)\right]$$
$$= \tan^{-1}[2\,(y_1/b)\tan\Phi(1+\alpha+\alpha^2)\,/\,(2 + 2\alpha r + \alpha + r)] \tag{14}$$

where $\alpha = d_2/d_1$, $r = B/b$, d_1 and d_2 are the pre-jump and post-jump depths, b and B are the half widths of the basin at the entry and exit, respectively. The conjugate depth ratio, α, in this non-prismatic stilling basin with adverse bed slope such that the wall reaction is balanced by bed reaction can be expressed by the relation.

$$F_1{}^2 = 1/2\,[(1-\alpha^2 r)/\,(1-\alpha r)]\,\alpha\, r \tag{15}$$

In a prismatic channel of rectangular section when $r = 1$ (i.e. $b = B$), Equation (15) reduces to the conjugate depth relation in a classical hydraulic jump given by Equation (16).

$$\alpha = d_2/d_1 = 1/2\left[\left(8F_1^2+1\right)^{1/2} - 1\right] \tag{16}$$

Experimental values of β_{opt} for best performance of the basin with 3:1 flaring of straight side walls are given in Figure 5. The method of computing the theoretical and experimental values of β_{opt} has been explained through an illustrative example given in Appendix 1.

6. Summary and conclusions

Innumerable canal falls are to be provided when a canal passes through steep sloping terrains. Different types of falls have evolved over time. An economic and efficient method of design of falls has been prescribed in unlined canals where the canal is usually flumed to reduce cost. Based on the research study carried out by the author and co-workers, an innovative method of design of a fall has been evolved. Hydraulic principles of finding optimum fluming ratio to maintain depth–discharge relation have been recommended. A new method of designing flow transitions and energy dissipation with the objective of increasing efficiency and reducing cost has been discussed. An example has been worked out at the end to illustrate the new design principles.

Disclosure statement

No potential conflict of interest was reported by the author.

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Appendix 1. Illustrative example

Design a canal fall with data given below:

Full supply Discharge in the canal, Q_{max} = 99.1 cumec
Full supply depth, Y_{1max} = 3.629 m
Mean Flow width of canal at FSL = 29.87 m
Longitudinal slope of bed = 1 in 8000
Manning's roughness coefficient, N = 0.025
Height of fall = 3 m
Minimum flow in the canal Q_{min} = 21.5 cumec
Corresponding minimum depth of flow $= Y_{1min}$ = 1.38 m

Computation of Flume width at throat (B_o) and Crest height (Δ)

From Proportionate Flow/ Flow Regime Consideration
$V_{1max} = 0.914$ m/s, $E_{1max} = Y_{1max} + (V_{1max})^2 / 2g = 3.672$ m, $F_1 = V_{1max} / (g\, Y_{1max})^{1/2} = 0.153$
$V_{1min} = 0.522$ m/s, $E_{1\,min} = Y_{1min} + (V_{1\,min})^2 / 2g = 1.394$ m
$B_0 = [0.7\,(Q^{2/3}_{max} - Q^{2/3}_{max}) / (E_{1max} - E_{1min})\,]^{\,3/2} = 8.64$ m, $Y_0 = Y_c = 2.374$ m and $F_0 = 1.0$

$V_x = V_1 + a\,(1 - \text{Cos}\Phi) = 0.91 + 1.102(1 - \text{Cos}\Phi)$
$Y_x = Y_1 - a/g\,[(a + V_1)\,(1 - \text{Cos}\Phi) - 1/2\,a\,\text{Sin}^2\,\Phi] = 3.629 - [0.227(1 - \text{Cos}\Phi) - .062\,\text{Sin}^2\,\Phi]$

X(m)	=	0	5	10	15	20
Φ_x(degree)	=	0	45	90	135	180
V_x(m/s)	=	0.914	1.234	2.016	2.795	3.118
Y_x(m)	=	3.629	3.499	3.404	3.272	3.176
B_x(m)	=	29.87	22.90	14.19	10.83	10
F_x(m)	=	0.153	0.211	0.346	0.493	0.588

Jaeger-type Inlet transition curve is obtained by plotting widths B_x at different X values as shown in Figure 1(b).

Design of Ogee-type Glacis

Assuming that there is no regulator over crest, the coordinates of the curved d/s glacis are found from Creager's formula (Equation 5), with $H_0 = E_1 - \Delta = 3.232$
$Y / H_0 = K\,(X / H_0)^n$
K and n values are found to be 0.56 and 1.75 for approach velocity head (ha $= V_1^{\,2} / 2g$) of 0.043 m and design head above crest (H_0 = of 3.232 m (3.672–0.44), respectively, from USBR (1968) publication 'Small dams'.

X(m)	=	0.25	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	4.663
X/H_0	=	0.078	0.155	0.309	0.464	619	0.774	0.928	1.083	1.237	1.392	1.442
Y/H_0	=	0.006	0.021	0.073	0.146	0.242	0.357	0.491	0.643	0.812	0.999	1.064
Y(m)	=	0.019	0.068	0.236	0.472	0.782	1.153	1.567	2.078	2.624	3.220	3.440

Since the flow at critical stage is wavy in the flumed section and from Figure 1, it is noticed that for an approaching flow Froude's number, F_1 = 0.153, there is hardly any economy in fluming beyond
$F_0 = 0.6$, adopt $F_0 = 0.6$ for determining economic fluming ratio given by Equation (1), i.e.
$B_o/B_1 = (F_1 / F_o)\,[\,(2 + F^2_{\ o}) / (2 + F_1^{\,2})\,]^{3/2} = 0.322$ and hence $B_0 = 8.9$ m;
Adopted bed width at flumed section, B_0 = 10 m
Corresponding value of crest height, $\Delta = E_{1max} - 3/2\,[(Q_{max}^{\ \ 2} / B_0^{\,2}) / g]^{1/3}$ = 0.44 m
Assuming no loss in head in inlet transition, i.e. $C_i = 0$ or $h_{Li} = 0$, $E_0 = E_1$ or, $E_0 = Y_0 + V_o^{\ 2} / 2g = 3.672$ and $q_0 = Q/B_0 = 9.9 = V_0\,Y_0$
Solving by trial, Y_0 = 3.176 m and V_0 = 3.118 m/s; $F_0 = V_0 / (g\,Y_0)^{1/2}$ = 0.558
Check: $B_o/B_1 = (0.153/0.558)\,[(2 + 0.558^2) / (2 + 0.153^2)]^{3/2} = 0.335$ and $B_0 = 0.335 \times 29.87 = 10$ m

Design of contracting transition

With 2 : 1 average side splay, axial length of inlet transition, $L_c = ½\,(B_1 - B_0) \times 2 = 19.87$ say 20 m
Adopt Jaeger-type transition given by Equations (7)–(11) as follows:
$a = 0.5\,(V_o - V_1) = 0.5\,(3.118 - 0.914) = 1.102$, $\Phi = \pi x / L_c = \pi\,x / 20$

The X, Y coordinates are plotted with crest as origin to obtain the d/s glacis profile as shown in Figure 1(b).

Design of stilling basin with diverging side walls

Assuming no head loss up to toe of the d/s glacis, specific energy of flow at toe (E_t) is given by:
$E_{t1} = E_{1max}$ + height of drop = 3.672 + 3 = 6.672 $= d_1 + U_t^{\,2}/2g$
$q = Q/B_0 = 9.9 = d_1 \times U_t$
where d_1 and U_1 are the pre-jump depth and velocity of flow at toe of d/s glacis, respectively. Solving the above two expressions by trial
$d_1 = 0.84$ m and $U_1 = 10.72$ and $F_{t1} = 3.73$
Axial Length of the Basin: $L_b = 3\,(B_1 - B_0) / 2 = 29.8$ m say 30 m
Conjugate depth ratio for the non-prismatic basin is given by Equation (15)

$$F_1^2 = \;1/2\,[(1 - \alpha^2 r)/\,(1 - \alpha r)]\;\alpha\,r$$

Putting $F_1 = F_{t1} = 3.73$, $r = B/b$ (Figure 5) = 2.987, the above equation reduces to

$$\alpha^3 \,-\, 9.95\,\alpha \,+ 3.219 \,= 0$$

Solving by trial, α =3 and d_2 = 3 d_1 = 3(0.84) = 2.52 m and submergence =3.629/2.52 = 1.44, i.e. the basin will operate under 44% submergence at maximum flow which is permitted as per test results.

Theoretical value of basin floor inclination, β_{opt} is given by Equation 14

$$\beta_{opt} = \tan^{-1}[2\, y_1 / b\tan\Phi(1+\alpha+\alpha^2) / (2 + 2\alpha r + \alpha + r)]$$

With $y_1 = d_1$ = 0.84 m, b = 5 m, tan Φ = 1/3 and r = 2.987, $\beta_{opt} = 3.36^{\circ}$

Experimental value of β_{opt} can be found from Figure 5 as follows:

$$q / \left(8gb^3\right)^{1/2} = 9 / (8 \times 9.8 \times 5^3)^{1/2} = 0.091 = 9.1 \times 10^{-2}$$

corresponding to above value of $q / (8gb^3)^{½}$ and F_1 = 3.73. $\beta_{opt} = 4.5^{\circ}$ (from Figure 5)

Provide basin Floor slope of $\beta_{opt} = 4.5^{\circ}$ for best performance.

Figure 1(b) is drawn on the basis of the above-mentioned computations.