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EFFECT OF INLET AND OUTLET GEOMETRIC SHAPE VARIATIONS OF A PIPE CULVERT ON LOCAL HEAD-LOSSES

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ABSTRACT

This research investigates the losses resulting from the sudden contraction and expansion of a pipe culvert. A four hundred runs were carried out considering various angles and contraction ratios at inlet and outlet of the pipe culvert and downstream submerged ratios with different flow conditions. Results were analyzed and graphically presented. The results indicated that the inlet angle of 15° with width contraction ratio of (b_u/D = 2.33) gives the least values of losses at the transition from a free-surface channel to the pipe culvert. For the transition from a conduit to a free-surface channel, the outlet angle of 60° with a submerged ratio of (h_d/D = 0.20) and the outlet angle of 30° with submerged ratios of (h_d/D=0.4, 0.6 and 0.8) give the least values of losses at width contraction ratio of (b_d/D = 2.33).These results agree well with the results obtained from previous studies.

Keywords: Culvert; Energy Loss; Pipe Entrance; Pipe Exit; Physical Model.

1. Introduction

The culvert consists of three main parts, namely the inlet, barrel and outlet. Flows in the culvert may be pressurized flow or open channel flows. Its cross section may be circular or box in shape. The different types of flow through the culvert have been previously described by Bodhaine [4], Chow [1], Blasidell [2], Dasika [6], Montes, [7], Hager and Giudice [9], Meselhe and Hebert [11] and Nguyen et al. [15].

In hydraulics, two kinds of energy losses may be take place, [12]. The first kind is due to shear stresses along the boundary walls, and is designated as friction loss due to surface resistance and the second one is intimately linked with the variations in the flow-path geometry, resulting in local-flow transitional. The local losses are known to be proportional to the dynamic pressure or kinetic energy of the flow. Usual formulations to compute a local head-loss coefficient from conduct velocity head is related to the flow velocity downstream of the conduit expansion/contraction, Gardel [3] and Idel'cik [5]. Idel'cik [5] considered wide spectra of geometry variations and varied circular conduit inlet configurations from a reservoir with negligible flow velocity. He [5] also proposed local head-loss coefficients for square conduit inlet protruding in a reservoir, depending on the conduit location and sidewall thickness. He [5] found the head-loss coefficient is equal to 0.63 when the conduit bottom is aligned with the reservoir bottom. When a sidewall and

the bottom of the conduit are those of the reservoir, the head-loss coefficient is equal to 0.77. Each conduit had a wall thickness that equals 0.03–0.04 times the width of the square conduit. Hager [12] summarized several local head-loss coefficient expressions for a conduit expansion/contraction. Hager [12] and Gardel [3] developed a head-loss coefficient formula for a conduit contraction when the wall angle of the contraction is equal to 90°. Norman et al. [10] provided detailed information about the hydraulic design method of highway culverts, considering varied geometries of the inlet. This design method was published in a report entitled Hydraulic Design of Highway Culverts. Gisonni and Pfister [17] investigated experimentally the flow features of combining flows at 45° and 90° junction manholes on circular conduits, with various diameters. They applied energy and momentum conservation along with their experimental results to provide expressions for the head loss coefficients. Tullis et al. [13] experimentally determined the entrance-loss coefficients for circular/elliptical buried-invert culverts in both un-submerged and submerged culvert inlet conditions. These experiments have been carried out for circular culverts with invert burial depths of 20%, 40%, and 50% and an elliptical culvert with 50% invert burial depth. The obtained coefficients varied depending on the geometry and the culvert inlet end treatments.

The present study aims to investigate the local head losses using different geometries of the sidewall at the inlet and outlet of the culvert (upstream/downstream sidewall thickness, angles) and downstream submergence ratios with different flow conditions.

2. Theoretical approach

The energy loss at the transition section from an open channel to a pipe culvert at upstream side of the culvert, ΔE_{T1}, and the energy loss at the transition section from a conduit to an open channel at downstream side of the pipe culvert, ΔE_{T2} depend on a large number of flow variables as follows:

$$\Delta E_{T1} = f(A, B, b_u, D, g, h_d, h_i, L, L_u, p_i, S, V_i, \rho, \theta_1, \mu) \quad (1a)$$

$$\Delta E_{T2} = f(A, B, b_d, D, g, h_d, h_i, L, L_d, p_i, S, V_i, \rho, \theta_2, \mu) \quad (1b)$$

Where, A is wetted area of cross section, B is upstream and downstream open channel width, b_u or $b_d = (B - 2b_w)$ is width of the contracted section at upstream or downstream, b_w is width of upstream or downstream sidewall, , D is the diameter of the pipe culvert, g is gravitational acceleration, h_d is submergence of the pipe culvert at downstream sides of the culvert, h_i is water depth at cross section i of the free-surface channel, L is pipe culvert length, L_u and L_d are lengths of the sidewalls at upstream and downstream sides of the culvert, p_i is pressure at cross section i of the conduit, S is bed slope in the free-surface channel/conduit reach, V_i is mean flow velocity at cross section i, θ_1 and θ_2 are inlet and outlet angles of the sidewalls, ρ is density of the water and μ *is* dynamic viscosity of water, see Fig 1.

Since ρ, μ, D, B, L, L_u and L_d were kept constant throughout the experimental program. The various parameters affecting the local head loss that taking place at the transition from a rectangular free-surface channel to a pipe conduit or vice-versa may be written as follows:

$$\left(\frac{\Delta E_{T1}}{D} \, or \, \frac{\Delta E_{T2}}{D} = f\left(F_r = \frac{V}{\sqrt{gD}}, R_e = \frac{\rho V D}{\mu}, \frac{b_u}{D}, \frac{b_d}{D}, \frac{h_d}{D}, \theta_1, \theta_2\right) \right) \quad (2)$$

The values of Reynolds number in this study are beyond 10^4, so, it is not incorporated. Montes [8] reported that for Reynolds numbers higher than 10^4, the viscous effects become less important.

Fig. 1. General view of used flume and considered (sidewalls)

3. Experiments

The experiments were conducted in a 40 m long, 0.40 m wide and 0.60 m deep glass flume, see Fig. 1. The experiments were carried out at the hydraulics Research Institute of National Water Research Center. All the tested configurations consider a 6 m long one PVC pipe between two free-surface-flow channels, 5 m long upstream and downstream, respectively. The culvert bed level is constant along the system with the flume bed. The channel and the pipe culvert had the same axis in plan. The inner diameter of the pipe D= 0.15 m. The upstream and downstream sidewalls with a length of 0.4 m were made in a rectangular cross section with a width, b_w of (2.5, 5, 7.5, 10, and 12.5 cm) depending on the configuration. Steady discharges of (4, 7, 10, 12, and 15 L/s) were flowing upstream of the flume through a permeable screen ensuring uniform velocity distribution over the cross section. In the downstream, a tailgate is used to control the water level in the flume, depending on the downstream submergence of the culvert.

Four hundred runs were carried out considering aforementioned five discharges, for each discharge four submergence ratios of (h_d/D = 0.2, 0.4, 0.6, and 0.8) were used. For each submergence ratio and at a constant downstream contraction ratio of (b_d/D = 1.67), Norman et al. [10], the upstream contraction ratios are changed six times (b_u/D = 2.33, 2.00, 1.67, 1.33, and 1.00) to reach the best upstream contraction ratio. The best upstream contraction ratio is used and then the downstream contraction ratios are changed six times (b_d/D = 2.33, 2.00, 1.67, 1.33, and 1.00) to reach the best downstream contraction ratio. With a constant angle of 90° at downstream sidewalls, the angle of the upstream sidewalls is changed five times (θ_1= 15, 30, 45, 60, and 90 ^{o}) to reach the best inlet angle. The optimum upstream angle (15 ^{o}) is used and then the angle of the downstream sidewalls is changed five times (θ_2 = 15, 30, 45, 60, and 90 ^{o}). In this study, the upstream water depth h_1 varies depending on the discharge Q and downstream submergence. Specific cross sections have been selected to compute the flow energy upstream and downstream of the transition. The location of these sections is shown in Fig. 2. Sections 1, 3, and 5 are far enough from the transition to ensure uniform flow-velocity conditions and thus to be used to compute the flow energy from water depth and pressure measurements, respectively, in the free-surface channel and in the pipe culvert. The difference in energy values between sections 1

and 3 and sections 3 and 5 provide the local head loss. Energy in sections 1, 3, and 5 is computed from the measurements in sections 1, 3 and 5 considering friction losses between the measurements in sections 1 and 3, 3 and 5. Each test has been repeated three times to ensure the consistency of the results.

The flume has been equipped with electromagnetic flow-meter to measure the discharge (flow rate). The water levels were measured in free surface channels by two precise digital point gauges. They mounted on carriage moving in the flow and the perpendicular directions in addition to four ultrasound sensors placed in Sections 1, 2, 4, and 5 in Fig. 2 with an accuracy of ±0.5 mm. The pressure in closed conduct pipe was measured by a piezoresistive pressure transducers placed in section 3, see Fig. 2.

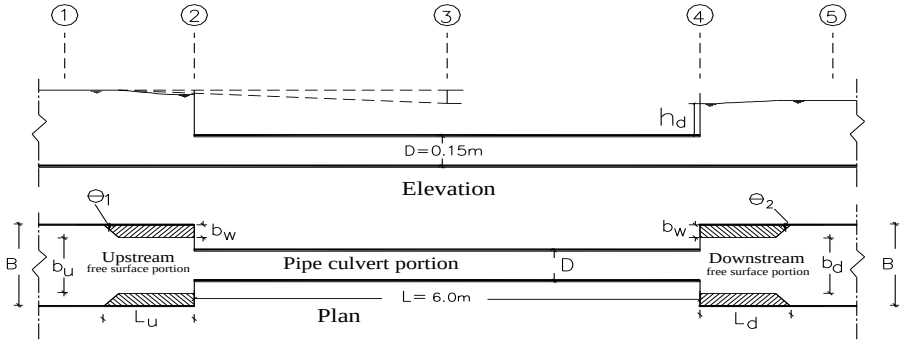

Fig. 2. Layout of geometric configuration and measurement cross sections

4. Results and discussions

The flow depths upstream and downstream of the pipe culvert were measured and the mean flow velocity is computed as follows:

$$V_i = \frac{Q}{b_i h_i} \tag{3}$$

And at the closed conduit portion, the mean flow velocity is computed as follows:

$$V_3 = \frac{Q}{\pi D^2 / 4} \tag{4}$$

The mean flow energy E are computed in open cross sections and at section 3 (with an elevation reference at the channel bottom) as follows:

$$E_i = h_i + \frac{V_i^2}{2g} \tag{5}$$

$$E_3 = \frac{p_3}{\gamma} + \frac{V_3^2}{2g} \tag{6}$$

In which p_3/γ is pressure head at cross section 3 of the pipe culvert. The energy losses at the entrance ΔE_{T1} and at the exit ΔE_{T2} are as computed as:

$$\Delta E_{T1} = \Delta E_{1\text{-}3} - \Delta E_{1\text{-}2} - \Delta E_{2\text{-}3} \tag{7}$$

$$\Delta E_{T2} = \Delta E_{3\text{-}5} - \Delta E_{3\text{-}4} - \Delta E_{4\text{-}5} \tag{8}$$

Where $\Delta E_{1\text{-}3}$ and $\Delta E_{3\text{-}5}$ are energy difference from section 1 to 3, and from section 3 to 5.

$\Delta E_{1\text{-}2}$, $\Delta E_{2\text{-}3}$, $\Delta E_{3\text{-}4}$ and $\Delta E_{4\text{-}5}$ are friction losses between these sections. The friction losses may be estimated using Darcy– Weisbach formula, Eq. 9 as follows:

$$\Delta E_j = \frac{f_j x_j}{D_{h,j}} \frac{V_j^2}{2g} \tag{9}$$

Where the subscript j represents free-surface channel reaches or the conduit reaches, x is reach length; V and D_h are flow velocity and hydraulic diameter, respectively, f is friction factor and is equal to 0.016 for the free-surface channel, 0.004 for the pipe culvert reach. For the considered wall materials, these values are obtained from the results presented by McGovern [14].

4.1. Effect of upstream contraction ratio (b_u/D) on Local Head-Loss (ΔE_{T1})

Results were grouped into dimensionless terms and the relationships were drawn to study the effect of the upstream contracted widths (Contraction ratios b_u/D) on relative values of the local head-loss ΔE_{T1}/D at the transition from a free open channel to a pipe culvert and were presented graphically in Figs.(from 3 to 6) as a function of the tested Froude number, F_r. for different values of downstream submergence ratio (h_d/D), ($\theta_1=\theta_2=90^o$), and downstream contraction ration (b_d/D = 1.67)

For all considered downstream submergence ratios h_d/D with all tested values of F_r, the value of b_u/D = 2.33 gives the lower values of ΔE_{T1}/D, meaning that, increasing the value of b_u/D decreases the value of ΔE_{T1}/D. Also, for all considered values of upstream contraction ratios b_u/D with all tested values of h_d/D, it is clear that ΔE_{T1}/D is increased as F_r increases. For lower values of F_r, all used values of b_u/D had small influence on the values of ΔE_{T1}/D. Finally, by comparing these figures, it is easy to observe that, the value of downstream submergence ratio h_d/D had small effect on the values of ΔE_{T1}/D for all considered values of b_u/D with all tested values of F_r. Generally, for all considered values of submergence ratio (h_d/D) with all tested values of F_r, using the smallest value of upstream contraction (b_u/D = 2.33) gives the minimum values of the local head-loss ΔE_{T1} at the transition from free flow to pipe flow.

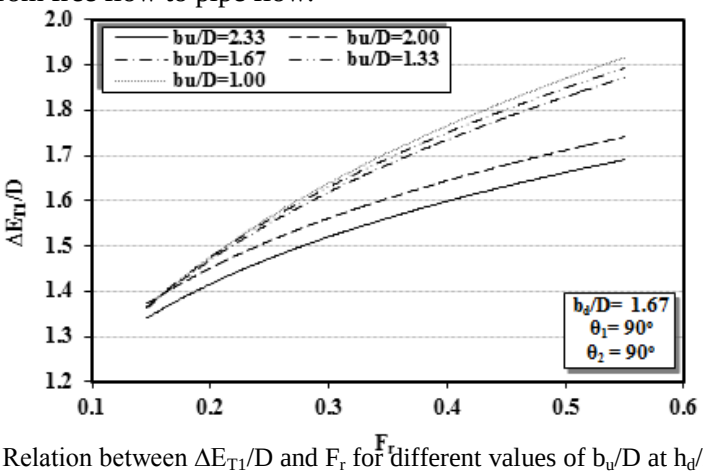

Fig. 3. Relation between ΔE_{T1}/D and F_r for different values of b_u/D at h_d/D = 0.2

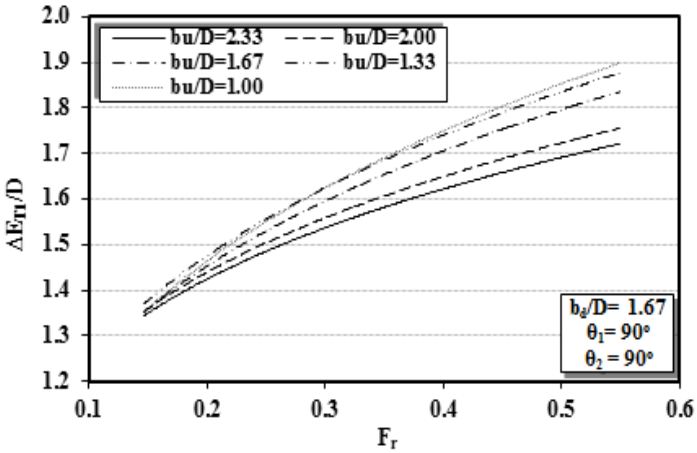

Fig. 4. Relation between $\Delta E_{T1}/D$ and F_r for different values of b_u/D at $h_d/D = 0.4$

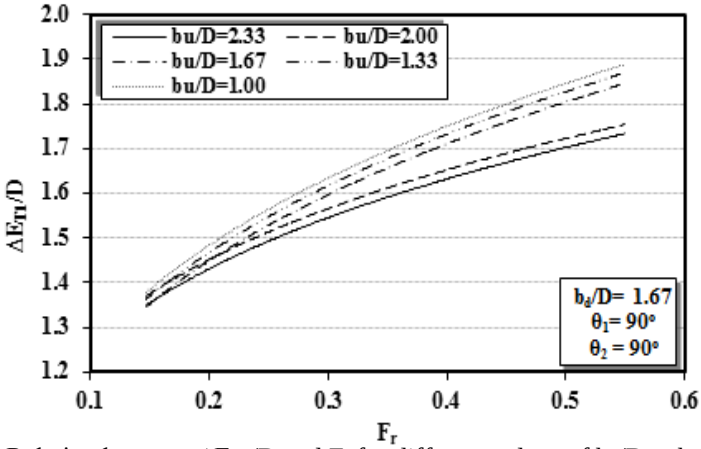

Fig. 5. Relation between $\Delta E_{T1}/D$ and F_r for different values of b_u/D at $h_d/D = 0.6$

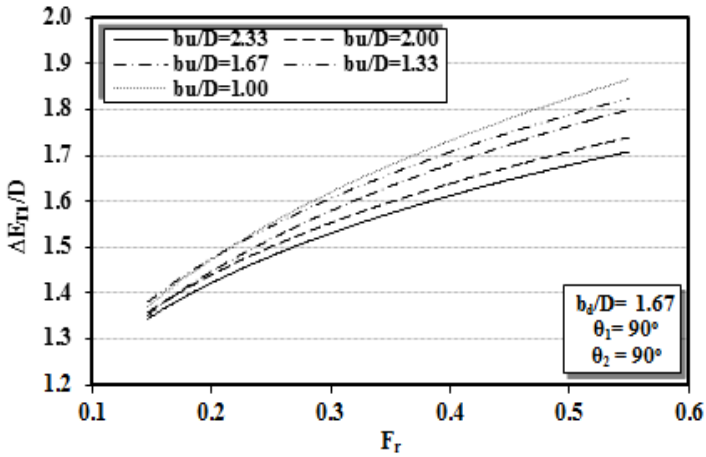

Fig. 6. Relation between $\Delta E_{T1}/D$ and F_r for different values of b_u/D at $h_d/D = 0.8$

4.2. Effect of upstream angle of sidewall (θ_1) on Local Head-Loss (ΔE_{T1})

The relationships were graphically presented to study the effect of the inlet angle of the upstream sidewalls θ_1 on relative values of the local head-loss $\Delta E_{T1}/D$ at the transition from free flow to pipe flow as shown in Figs. (from 7 to 10) as a function of the tested

Froude number, F_r. for different values of (h_d/D) and, θ_1 was taken as a third dimension. θ_2 was kept constant at 90° and also b_u/D and b_d/D equal to 2.33.

For all considered downstream submergence ratios h_d/D with all tested values of F_r, the value of θ_1= 15° gives the smaller values of $\Delta E_{T1}/D$ and the value of θ_1= 90° gives the higher values of $\Delta E_{T1}/D$, meaning that, decreasing the value of θ_1 decreases the value of $\Delta E_{T1}/D$. This result agrees well with the results obtained by Awad M. [16].

For all considered values of the inlet angle θ_1 with all tested values of h_d/D, it is clear that $\Delta E_{T1}/D$ is increased as F_r increases. By comparing these results, it is easy to observe that for all considered values of θ_1 with all tested values of F_r, the value of downstream submergence ratio h_d/D had small effect on the values of $\Delta E_{T1}/D$. Finally, for lower values of F_r, all tested values of θ_1 had small influence on the values of $\Delta E_{T1}/D$. Generally, for all considered values of submergence ratio (h_d/D) with all tested values of F_r, using the smallest value of inlet angle of the upstream sidewall (θ_1= 15°), gives the minimum values of the local head-loss ΔE_{T1} at the transition from free flow to pipe flow.

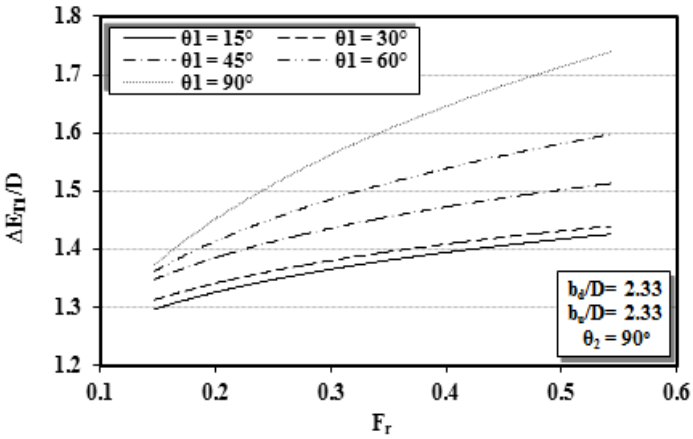

Fig. 7. Relation between of $\Delta E_{T1}/D$ and F_r for different values of θ_1 at h_d/D = 0.2

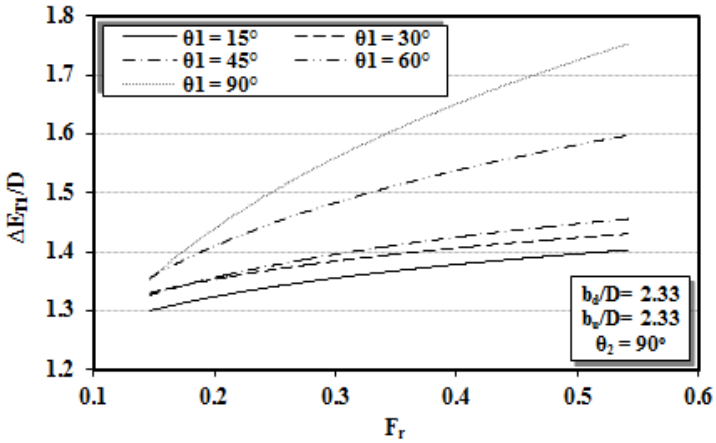

Fig. 8. Relation between $\Delta E_{T1}/D$ and F_r for different values of θ_1 at hd/D = 0.4

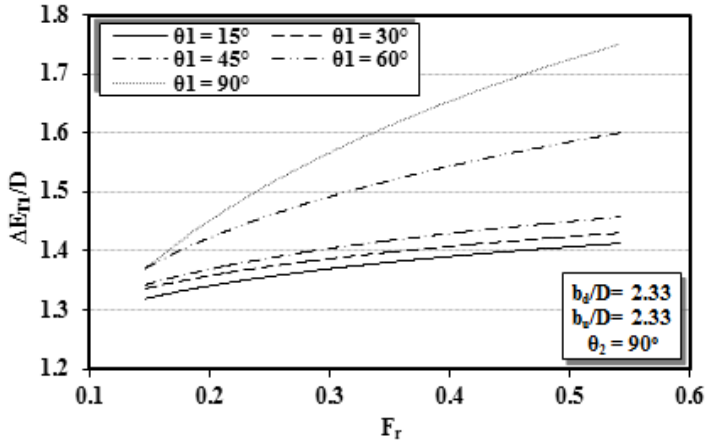

Fig. 9. Relation between $\Delta E_{T1}/D$ and F_r for different values of θ_1 at hd/D = 0.6

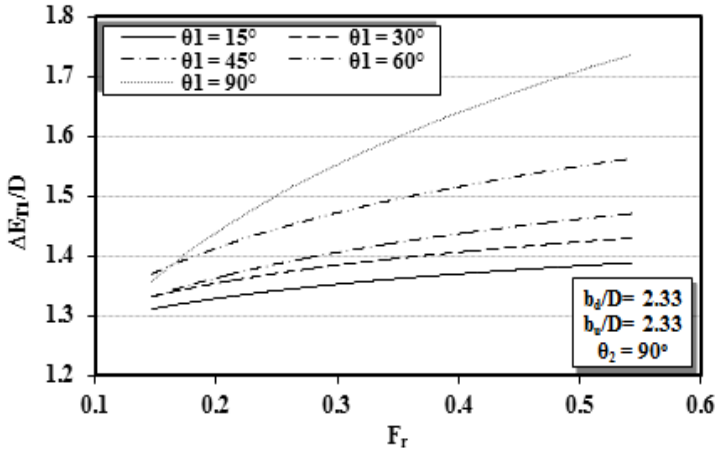

Fig. 10. Relation between $\Delta E_{T1}/D$ and F_r for different values of θ_1 at hd/D = 0.8

4.3. Effect of downstream contraction ratio (b_d/D) on Local Head-Loss (ΔE_{T2})

The effect of the downstream contracted width ratios b_d/D on relative values of the local head-loss $\Delta E_{T2}/D$ at the transition from a pipe culvert to a free open channel were presented graphically in Figs. (from 11 to 14) as a function of the tested Froude number, F_r. for different values of downstream submergence ratio (h_d/D), (θ_1=15°), (θ_2=90°), and upstream contraction ratio (b_u/D = 2.33)

For all considered downstream submergence ratios h_d/D with all tested values of F_r, the value of b_d/D = 2.33 gives the smaller values of $\Delta E_{T2}/D$, meaning that, increasing the value of b_d/D decreases the value of $\Delta E_{T2}/D$. Also, for all considered values of b_d/D with all tested values of F_r, increasing the value of downstream submergence ratios h_d/D leads to increase the values of $\Delta E_{T2}/D$ as shown in Figs. (from 11 to 14). Generally, for all considered values of submergence ratio (h_d/D) with all tested values of F_r, using the downstream contraction ratio of 2.33, gives the minimum values of the local head-loss ΔE_{T2} at the transition from a pipe flow to a free open channel flow.

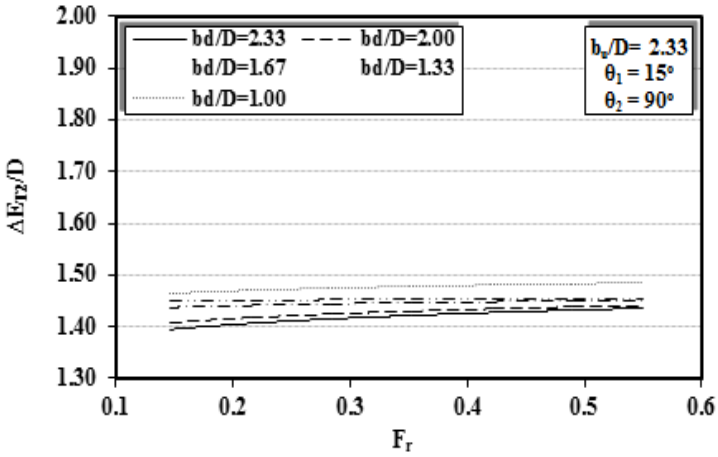

Fig. 11. Relation between $\Delta E_{T2}/D$ and F_r for different values of (b_d/D) at h_d/D =0.2

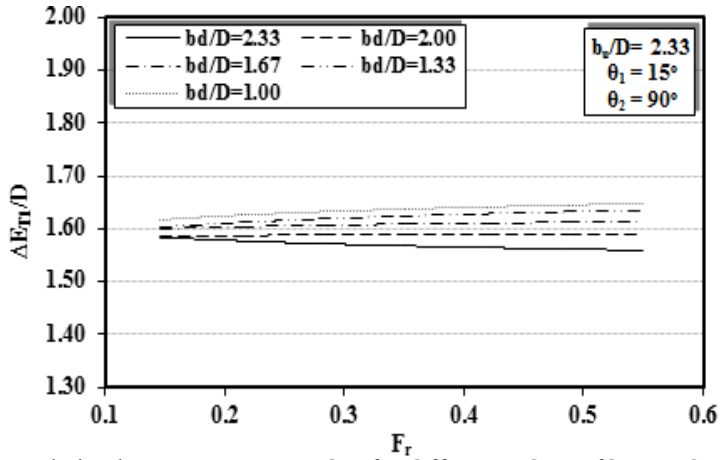

Fig. 12. Relation between $\Delta E_{T2}/D$ and F_r for different values of b_d/D at h_d/D =0.4

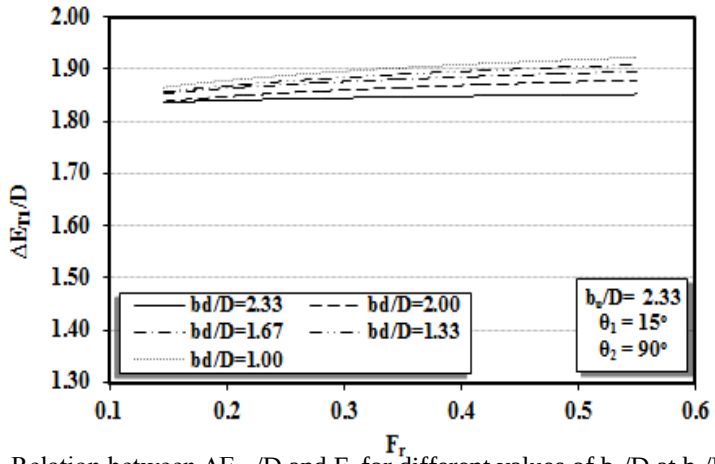

Fig. 13. Relation between $\Delta E_{T2}/D$ and F_r for different values of b_d/D at h_d/D =0.6

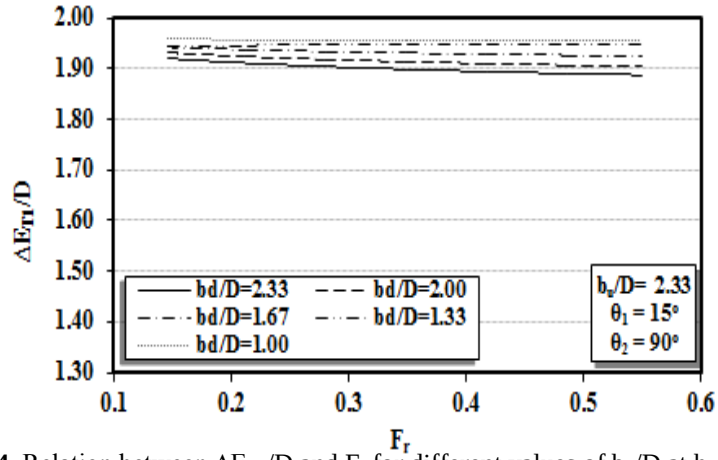

Fig. 14. Relation between $\Delta E_{T2}/D$ and F_r for different values of b_d/D at h_d/D =0.8

4.4. Effect of downstream angle of sidewall (θ_2) on Local Head-Loss (ΔE_{T2})

The relationships were graphically presented to study the effect of the outlet angle of the downstream sidewall θ_2 on relative values of the local head-loss $\Delta E_{T2}/D$ at the transition from pipe flow to free flow as shown in Figs.(from 15 to18) as a function of the tested Froude number, F_r. for different values of downstream submergence ratio (h_d/D), (θ_1=15°), and $b_u/D = b_d/D$ =2.33.

From these figures, it can be concluded that for submergence ratio of h_d/D = 0.2 with all tested values of F_r, the value of θ_2= 60° gives the smaller values of $\Delta E_{T2}/D$ but for submergence ratios of (h_d/D = 0.4, 0.6, and 0.8), the value of θ_2= 30° gives the smallest values while that the value of θ_2= 90° gives the higher values of $\Delta E_{T2}/D$ for all values of h_d/D. These results agree well with the results obtained by Awad M. [16]. Also, it is clear that the $\Delta E_{T2}/D$ is increased as F_r increases and for lower values of F_r, all tested values of θ_2 had small influence on the values of $\Delta E_{T2}/D$. Finally, for all considered values of θ_2 with all tested values of F_r, increasing the value of downstream submergence ratios h_d/D leads to increase the values of $\Delta E_{T2}/D$, see Figs.(from 15 to 18). Generally, at a specific value of Froude number F_r the outlet angle θ_2 = 60° gives the minimum value of $\Delta E_{T2}/D$, these results match at (h_d/D = 0.2). But with the increase of downstream submerged ratios of (h_d/D = 0.4, 0.6 and 0.8), the outlet angle of θ_2 = 30° give the minimum values of $\Delta E_{T2}/D$.

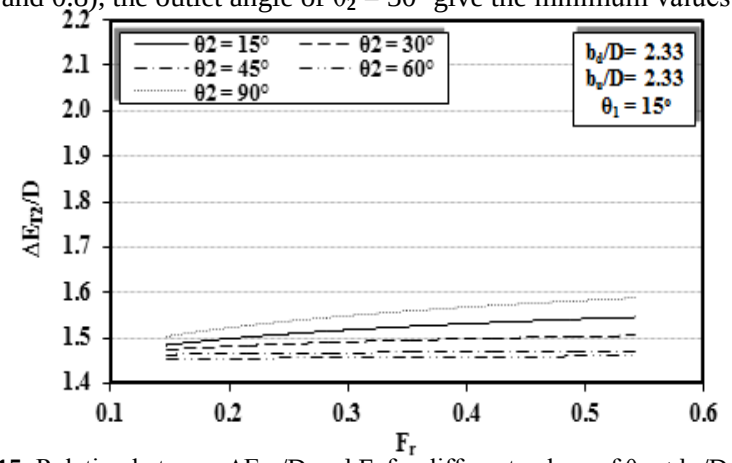

Fig. 15. Relation between $\Delta E_{T2}/D$ and F_r for different values of θ_2 at h_d/D = 0.2

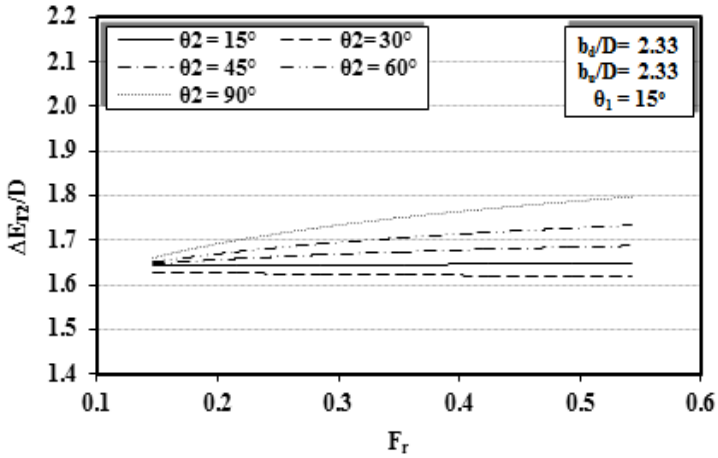

Fig. 16. Relation between $\Delta E_{T2}/D$ and F_r for different values of θ_2 at $h_d/D = 0.4$

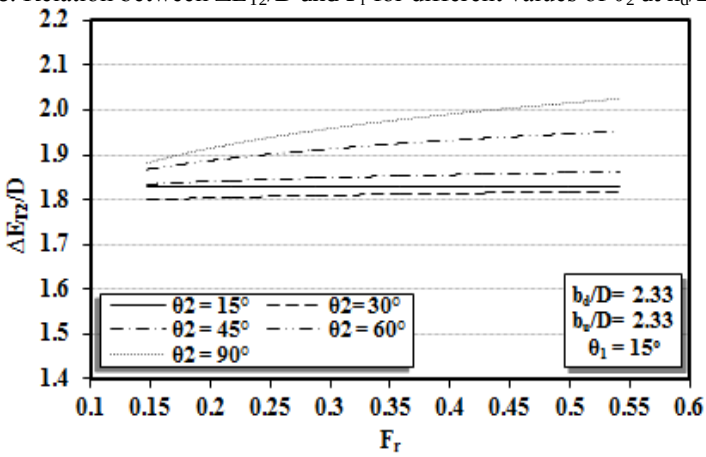

Fig. 17. Relation between $\Delta E_{T2}/D$ and F_r for different values of θ_2 at $h_d/D = 0.6$

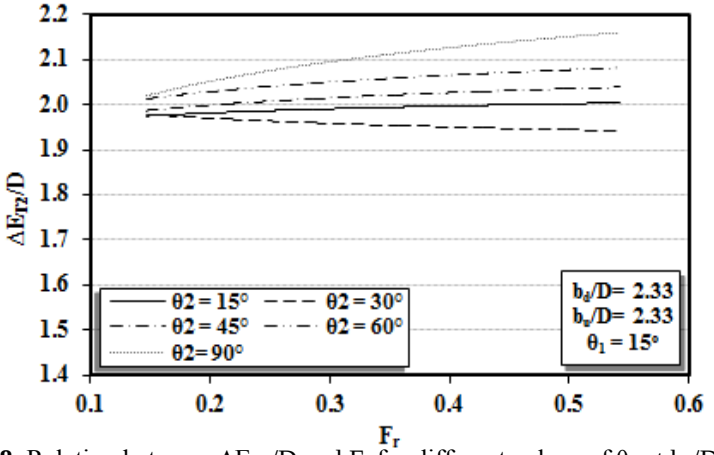

Fig. 18. Relation between $\Delta E_{T2}/D$ and F_r for different values of θ_2 at $h_d/D = 0.8$

5. Local Head-Loss coefficient

The calculated friction coefficient in the free-surface channel is very small. So, the local

head-loss coefficient k at the transition may be computed as a function of the flow kinetic energy as follow:

$$k = \frac{2g\Delta E_T}{V^2} \tag{10}$$

The reference velocity V can be selected so that no problem arises for its determination in further applications. Throughout this study, the values of the reference velocity V are referred to the Cross Section 3, computed from Eq. 4, whatever the discharge and geometrical configurations are, as the downstream boundary condition has been regulated to always create pressurized flow along the whole culvert length, Nguyen et al. [15].

The entrance local head-loss coefficient (k_u) was calculated as a function of the ratio of the downstream cross section area, A_3 to the upstream one A_1, (Sections 3 and 1, respectively in Fig. 2) as follow:

$$k_u = a\,(1 - A_3 / A_1) \tag{11}$$

The downstream local head-loss coefficient k_d was calculated as

$$k_d = b\,(1 - A_3 / A_5) \tag{12}$$

Where, A_3/A_5 is ratio of the cross section area A_3 to the downstream one A_5, (Sections 3 and 5, respectively in Fig. 2), a and b are coefficients that should be determined experimentally.

Based on the experimental results, the coefficient a, was found between 0.426 and 1.213 and the coefficient b, was found between 0.322 and 0.976. These experimental results are in close agreement with the findings by Tullis et al. [13].

5.1. Derivation of Local Head-Loss

It is important to predict local head loss at the transition from open channel flow to pipe flow ΔE_{T1}, and local head loss at the transition from pipe flow to open channel flow ΔE_{T2} for the different cases being under investigation. Based on the experimental data and using the statistical methods with the presence of the different flow conditions, several models were proposed and their coefficients were estimated. The best equations predicting ΔE_{T1} and ΔE_{T2} may be as:

$$\frac{\Delta E_{T1}}{D} = 1.32\, F_r^{0.18} \left(\frac{h_d}{D}\right)^{-0.005} (\theta_1)^{0.098} \left(\frac{b_u}{D}\right)^{-0.08} \tag{13}$$

$$\frac{\Delta E_{T2}}{D} = 2.17\, F_r^{0.013} \left(\frac{h_d}{D}\right)^{0.224} (\theta_2)^{-0.007} \left(\frac{b_d}{D}\right)^{-0.006} \tag{14}$$

Figure (19) shows a comparison between the measured relative local head loss at the transition from open channel flow to pipe flow, ($\Delta E_{T1}/D$) and the calculated one using Eq. 13. Figure (20) compares the measured relative local head loss at the transition from pipe flow to open channel flow, (($\Delta E_{T2}/D$) and the calculated one using Eq. 14. It can be noticed that, the predicted data agrees well with the measured one. The developed model of the equation has been validated through the tested condition. The regression statistics for Eqs. 13 and 14 define the coefficients of determination (R^2) as 0.86 and 0.92 respectively.

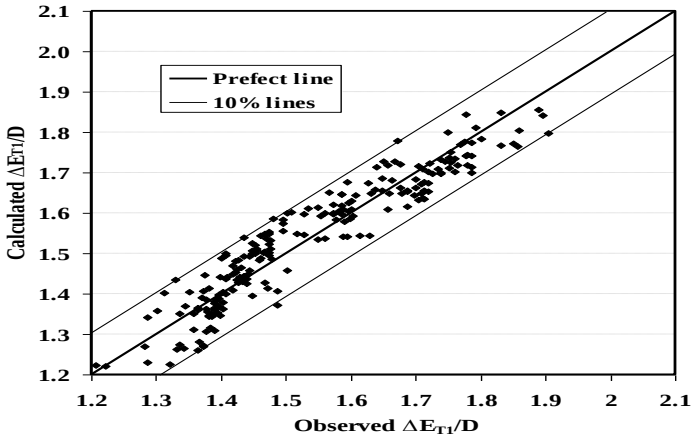

Fig. 19. Comparison between calculated and measured values of $\Delta E_{T1}/D$

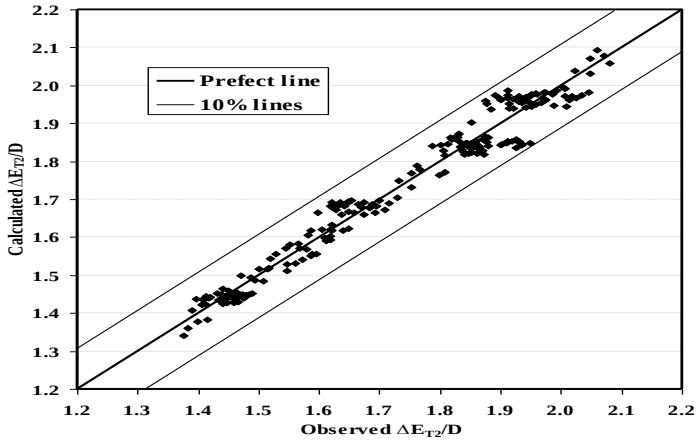

Fig. 20. Comparison between calculated and measured values of $\Delta E_{T2}/D$

6. Conclusions

The conclusions from this study may be as:

- The values of $b_u/D = 2.33$ and $b_d/D = 2.33$ give the lower values of $\Delta E_{T1}/D$ and $\Delta E_{T2}/D$ respectively.
- The value of $\theta_1 = 15^o$ gives the smaller values of $\Delta E_{T1}/D$ while the value of $\theta_1 = 90^o$ gives the higher values of $\Delta E_{T1}/D$.
- For lower values of F_r, the angle of sidewalls and the value of contraction ratios at the inlet and outlet of the pipe culvert had small effect on the values of Local Head-Losses.
- With all tested values of F_r and all tested values of submergence ratio of h_d/D, the value of $\theta_2 = 90^o$ gives the higher values of $\Delta E_{T2}/D$. But the value of $\theta_2 = 60^o$ for submergence ratios of ($h_d/D = 0.2$) and the value of $\theta_2 = 30^o$ for submergence ratios of ($h_d/D = 0.4$, 0.6, and 0.8) give the smaller values of $\Delta E_{T2}/D$
- Increasing the value of downstream submergence ratios h_d/D leads to increase the values of $\Delta E_{T2}/D$. But it had small effect on the values of $\Delta E_{T1}/D$.

Upstream and downstream local head-loss coefficients were determined for the tested geometrical configurations. Equations predicting the local head loss at the transition from open channel flow to pipe flow and the local head loss at the transition from pipe flow to open channel flow are presented with limitation to tested conditions.

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NOTATION

The following symbols were used through this research:
A is wetted area of cross section (m^2);
B is upstream and downstream open channel width (m);
b_d is width of contracted open section at downstream (b_d= B-2b_w), (m);
b_i is bed width at cross section i of the free-surface channel (m);
b_u is width of contracted open section at upstream sidewall (b_u= B-2b_w), (m);
b_w is width of upstream or downstream sidewall (width of abutment) (m);
D is diameter of the pipe culvert (m);
D_h is hydraulic diameter (m);
$\Delta E_{1\text{-}3}$ is energy difference from section 1 to 3 (m);
$\Delta E_{3\text{-}5}$ is energy difference from section 3 to 5 (m);
ΔE_j is friction loss in the free-surface channel/pipe reach (m);
ΔE_{T1} is local head loss at the transition from open channel flow to pipe flow (m);
ΔE_{T2} is local head loss at the transition from pipe flow to open channel flow (m);
F_r is the Pipe Froude number (-);
f is friction factor (-);
g is gravitational acceleration(m/s^2);
h_d is submerged depth above the pipe culvert at downstream side of the culvert (m);
h_i is water depth at cross section i of the free-surface channel (m);
L is pipe culvert length (m);
L_d is length of the sidewalls at downstream side of the culvert (m);
L_u is length of the sidewalls at upstream side of the culvert (m);
k is local head-loss coefficient (-);
p/γ is pressure head at cross section 3 of the pipe culvert (m);
S is bed slope in the free-surface channel/conduit reach (m/m);
V_i is mean flow velocity at cross section i (m/s);
x is reach length (m);
θ_1 is inlet angle of the upstream sidewall at upstream side of the pipe culvert;
θ_2 is outlet angle of downstream sidewall at downstream side of the pipe culvert;
μ is dynamic viscosity of water ($N.s./m^2$).
ρ is density of the water (kg/m^3); and

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تأثير التغيرات الهندسيه لمدخل ومخرج ماسورة بربخ على الفواقد الهيدروليكية المحلية

الملخص العربى:

يهدف هذا البحث إلى دراسة السريان بماسورة بربخ من خلال دراسة الفواقد الهيدروليكية الناتجة عن التضييق أو التوسيع المفاجئ لمدخل ومخرج البربخ. وقد تم إجراء أربع مائة تجربة معملية لدراسة تأثير زوايا المدخل والمخرج و نسب تضييق مدخل ومخرج البربخ وذلك بوجود نسب غمر مختلفة فوق مخرج البربخ وبمرور تصرفات مختلفة خلال البربخ. وقد تم تحليل النتائج وعرضها بيانيا. أشارت النتائج إلى أن زاوية مدخل 15° مع نسبة تضيق العرض تساوى 2.33 عند مدخل البربح ($b_u/D = 2.33$) يعطي أقل قيم للفواقد الهيدروليكية أثناء إنتقال السريان عند مدخل البربخ (من قناة مكشوفة إلى ماسورة البربخ). وعند إنتقال السريان من ماسورة البربخ إلى القناة المكشوفة عند مخرج البربخ تكون زاوية المخرج 60° مع نسبة غمر ($h_d/D = 0.20$) وزاوية مخرج 30° مع نسب غمر (h_d/D=0.4, 0.6 and 0.8) تعطي أقل قيم للفواقد الهيدروليكية وذلك مع نفس نسبة تضييق العرض عند مخرج البربخ ($b_d/D = 2.33$).