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Shear Velocity Method to Riprap Sizing at Downstream of Stilling Basins

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Abstract: A model study was conducted to evaluate existing riprap sizing downstream of stilling basins. The US Bureau of Reclamation [21], Pilarczyk [22] and Escarameia [23] procedures were assessed and determined to be conservative in riprap sizing. Flows with and without hydraulic jump were tested where 8 different sizes of stones, ranging from 8.75 to 47.5 mm, were placed at downstream bed of stilling basin. An alternative sizing was developed using the shear velocity, shear stress and shear Reynolds number. The results indicate that ratio between shear velocities at downstream of stilling basin with hydraulic jump to that of uniform flow is 0.54. Furthermore a new and simplified formula for predicting the riprap sizing downstream of stilling basin is developed. The results were verified for discharge intensities up to 0.3 m^3/s/m.

Key words: Riprap protection . stilling basin . shear velocity . hydraulic jump . stone size

INTRODUCTION

Riprap protection is usually used to avoid the bed erosion and bank failure as a consequence of flow and wave action. Riprap materials are commonly selected from large stone particles to create a durable and economic mattress. The characteristics of the riprap particles and layout such as its size, mass density and shape are the important factors in achieving a stable protection. Although the application of large stones guarantees the bed protection against erosion, but having access to such materials needs the employment of heavy machineries and conveyance systems. Therefore, selecting the pertinent smallest riprap size to withstand against the shear stress resulted from the flowing water was one the most important factors in designing of the hydraulic structures since from the early days.

The Federal Highway Administration (FHWA) [1], Maynord and Abt [2], the US Army Corps of Engineers (USCOE) [3] and Maynord [4] have recommended some design codes for riprap protection which was based on the study of shear velocity and critical shear stress using trial and error method. This type of approaches concentrates on bed protection against erosion which seems not being appropriate in designing of riprap protection downstream of hydraulic structures such as stilling basins.

The prediction of scour whole dimensions at its equilibrium stage is often considered in designing of stilling basins encountered with spillways and sluice gates. Formation of hydraulic jump downstream of such structures causes intense turbulent flow which gives rise to scouring capacity in a complex feature. The result will end to a deep scour hole behind the stilling basin which may endanger its stability. Having a trustful information about the characteristics of local scour will therefore, ease the design of stilling basins downstream of hydraulic structures and their adjacent bed protection which is often achieved by ripraps of larger size materials. These techniques are commonly used to locally increase the bed resistance against the flow features working for erosion.

Many researchers such as Breusers [5], Farhoudi and Smith [6, 7], Nik Hassan and Narayanan [8], Chatterjee *et al*. [9], Balachandar and Kells [10], Balachandar *et al*. [11], Kells *et al*. [12], Dey and Westrich [13], Sarkar and Dey [14] and Dey and Sarkar [15] have widely studied erosion of alluvial beds downstream of aprons employed either with spillways or gates. In most investigations the characteristics of riprap layout (such as; stones mean diameter, riprap layer thickness, depth and orientation of particles), stability of riprap layer and the effect of several parameters are studied using the clear water conditions. Riprap design criteria are usually defined to resist against unfavorable situations. The reports of different researchers, under varying conditions, have proposed some criteria to design the stable riprap protection downstream of aprons.

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Posey and Peterson [16] recommended the incipient motion of the first riprap particle as the destruction criterion. Parola [17] used a three-layered riprap with a colored layer in the middle. He accepts the condition for destruction if the colored layer was seen after 30 minutes. Chiew [18] observed 15-minutes duration for the failure of riprap around a bridge pier and considered the instability as soon as the riprap layer was completely destroyed. He did not accept the movement of a few grains of the riprap layer, as a sufficient scale for failure. Lauchlan and Melville [19] considered riprap destruction at the time when 20% of the maximum depth of riprap layer was failed.

One of the design parameters of riprap is its thickness and filtering characteristics. The exit tendency of finer grains located below the riprap layer will result in screen failure [18] which could be solved by thickening the riprap thickness and maintaining a filter layer between the fine bed and riprap mattress. Worman [20] showed that if the thickness of the riprap layer is sufficient, no filter is required. He concluded that the function of a multi-layered riprap with graded materials would be similar to a thin one-layered homogenous riprap.

The investigations by USBR [21], Pilarczyk [22], Wallingford [23], Farhoudi and Valizadegan [24] and Farhoudi and Pourjabbar [25] have been aimed to determine the stable diameter of riprap downstream of stilling basins and recommended some relationships to designing of riprap size.

Peterka [21] studied the stability and failure of riprap downstream of several stilling basins both in the laboratory and field. The outcome was a graph relating riprap diameter to bottom velocity of the flow. He suggested a minimum riprap thickness of 1.5 times of the size of the largest grain.

Based on the effect of flow turbulence, flow depth and velocity, bed slope and material characteristics, Pilarczyk [22] suggested an equation to design the riprap, concrete blocks and box gabions.

Escarameia [23] also suggested an equation to design the riprap and concrete blocks downstream of hydraulic structures. Their relationship relates the riprap size to bottom velocity (ie: velocity in 10% of depth from the bed), turbulence and nature of bed material. In their research, different equations were used to determine bottom velocity at different turbulence levels.

Therefore, it seems to be quite helpful if one could correlate the uniform flow condition (at absence of hydraulic jump) with flow properties at downstream of hydraulic jump to achieve an appropriate riprap size. This approach would facilitate the application of uniform flow in designing of riprap protection downstream of hydraulic jump instead of shear force and velocity.

METHODS AND MATERIALS

The experiments were carried out in a laboratory flume 5.5 m long and 29.7 cm wide as shown in Fig. 1. The height of flume wall was 1.1 m for the first 1.5 m at upstream feeding a sluice gate at the entrance of channel. The gate was of a 2 cm thickness, 29.7 cm width and 1 m height. The stilling tank was shaped in a way to produce streamlined flow in the reach. The experiments were conducted with different gate openings. A 50 cm long riprap tank was installed on the downstream bed of the flume at 2.5 m from the gate. A hinged gate was used at the end of the flume to control the tail water level. Discharge was measured using a calibrated sharp rectangular weir. A Pitot tube with an external diameter of 6 mm was used to measure the flow velocity. A series of piezometers were fixed at the bed of the flume to determine the flow depths along the channel.

Eight different river gravels of 8.75 to 47.5 mm size were used as riprap grains. The grains remaining between two sieves were washed to give uniform, clean and non-cohesive particles. The average size of the two sieves was defined as D_{50}The relative density of the grains was measured in a scaled cylinder using Archimedes method. For each of the riprap sizes D_{50}, the specific gravity SG and geometric standard deviation σ_g were determined. The properties of used materials are shown in Table 1.

Table 1: Riprap properties

No.	Upper sieve (mm)	Lower sieve (mm)	D_{84} (mm)	D_{50} (mm)	D_{16} (mm)	σ_g
1	09.5	08.0	09.26	08.75	08.24	1.06
2	12.5	09.5	12.02	11.00	09.98	1.10
3	19.0	16.0	18.52	17.50	16.48	1.06
4	25.0	19.0	24.04	22.00	19.96	1.10
5	31.5	25.0	30.46	28.25	26.04	1.08
6	37.5	31.5	36.54	34.50	32.46	1.06
7	45.0	37.5	43.80	41.25	38.70	1.06
8	50.0	45.0	49.20	47.50	45.80	1.04

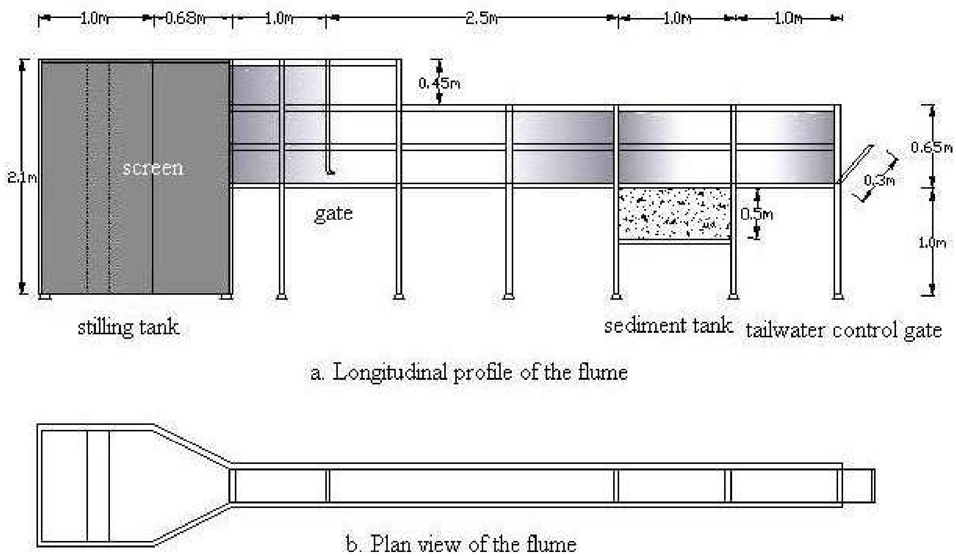


Fig. 1: Experimental layout

With special justifications, the average velocity and flow depth at incipient condition along with critical shear velocity of riprap particles were measured. The sediment tank was filled with riprap materials and its surface was leveled prior to each experiment. While the hinged gate was fully closed, flow control tap was gradually opened to fill the channel where the riprap was at standstill state. The hinged gate was then adjusted to achieve the pertinent velocities for a specified discharge.

In the present study riprap blanket of sufficient thickness (at least 6 times the riprap diameter) was experimentally investigated and the incipient motion of riprap particles was taken as failure scale. This assures a higher factor of safety compared with previous works.

ANALISIS OF EXPERIMENTAL RESULT

At incipient motion of the particles, the mean velocity (V_i) of uniform flow (without formation of hydraulic jump) was determined under each flow stages. The results are compared in Table 2. for D_{50}=8.75 mm with the those suggested by other investigators as shown below.

Isbash (1935) cited by Chiew [26]:

$$V_i = 0.85[2g(SG-1)D_{50}]^{0.5} \tag{1}$$

Niel (1967) cited by Parola [17]:

$$\frac{\rho V_i^2}{(\gamma_s-\gamma)D_{50}} = 2.5\left[\frac{D_{50}}{y}\right]^{-0.2} \tag{2}$$

Shafaei Bajestan [27]:

$$V_i = 2.2[g(SG-1)D_{50}]^{0.5} \Leftarrow D_{50}/y < 0.1 \tag{3}$$

$$\frac{V_i}{[g(SG-1)D_{50}]^{0.5}} = 1.252\left[\frac{y}{D_{50}}\right]^{0.25} \Leftarrow D_{50}/y > 0.1 \tag{4}$$

The results demonstrate an acceptable correlations with those suggested by Neil [28] and Shafaei Bajestan [27].

To determine the critical shear velocity of the riprap particles (either in steady flow condition or downstream of hydraulic jump), first the mean velocity along with flow depth at different incipient conditions were observed. Having V and y the values of V_* were then determined by using the following imperical equations for all riprap particles:

Chezy's Equation (1959) [cited by Maynord [29]]:

$$\frac{V}{V_*} = \frac{18}{\sqrt{g}}\log\frac{12R}{D_{50}+3.3\frac{V}{V_*}} \tag{5}$$

Kolegan's Equation (1993) [cited by Maynord [18]]:

$$\frac{V}{V_*} = 6.25 + 5.75\log\frac{R}{D_{50}} \tag{6}$$

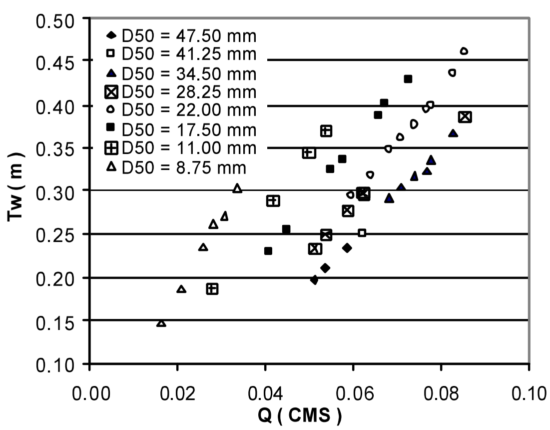
Manning and Strikler's Equation [cited by Chiew [30]]:

Table 2: Comparasion of measured mean velocity of uniform flow (without formation of hydraulic jump) with other suggested methods under different flow depths and discharges at incipient stage of D_{50} = 8.75 mm

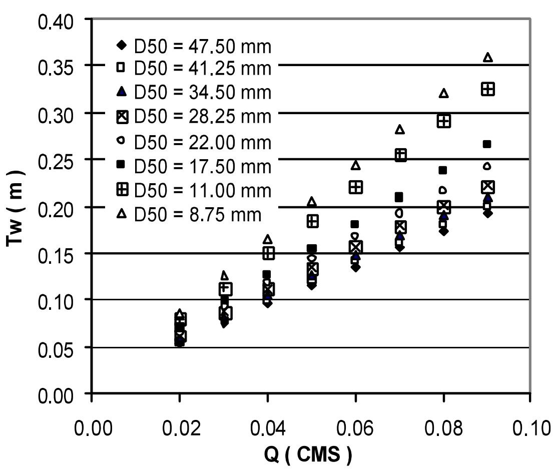
	Isbash (1935)		Niel (1967)		Shafaei Bajestan (1991)		Present study	
Q (CMS)	y (m)	V_i (m/s)	y (m)	V_i (m/s)	y (m)	V_i (m/s)	y (m)	V_i (m/s)
0.0588	0.438	0.452	0.239	0.828	0.239	0.828	0.240	0.825
0.0683	0.508	0.452	0.273	0.840	0.278	0.828	0.275	0.836
0.0854	0.636	0.452	0.335	0.857	0.347	0.828	0.339	0.849

Table 3: Values of V_* for riprap size of D_{50} = 8.75 mm at flow condition of without formation of hydraulic jump

			V_* (m/s)				
D_{50} (mm)	y (m)	V (m/s)	Chezy	Kolegan	Manning and Strikler	Melville	Mean
8.75	0.24	0.82	0.0687	0.0683	0.0622	0.0659	0.0663
	0.27	0.83	0.0684	0.0681	0.0611	0.0645	0.0655
	0.33	0.84	0.0683	0.0679	0.0602	0.0633	0.0649



(A) Downstream of hydraulic jump



(B) Steady flow

Fig. 2: Variation of flow depth with discharge at incipient condition of ripraps

$$\frac{V}{V_*} = 7.66\left(\frac{y}{D_{50}}\right)^{\frac{1}{6}} \qquad (7)$$

Melville's Equation(1988):

$$\frac{V}{V_*} = 5.75\log\frac{5.53y}{D_{50}} \qquad (8)$$

The Computed values of V_* by the above relationships, for riprap size of D_{50} =8.75 mm, are tabulated in Table 3. The results are farely close and the average values of V_* were considered for the flow condition of without formation of hydraulic jump. It was noticed that the calculated V_* values were 40% less than the corresponding values from Shields curve.

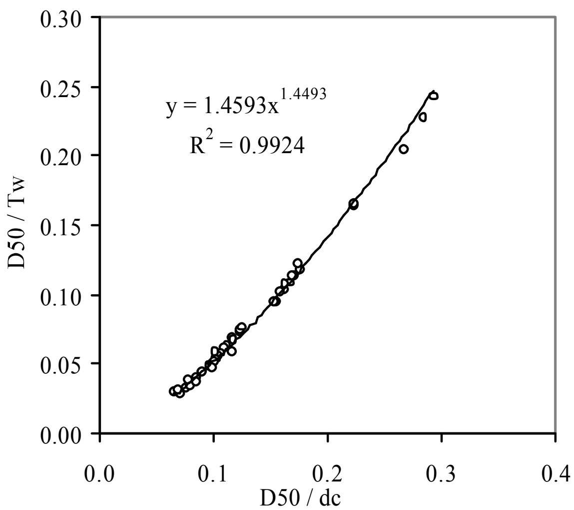
In Fig. 2 the depths at which the ripraps of different sizes fall in motion are depicted for constant flow discharges both at steady flow condition and downstream of hydraulic jump. It would be evident from careful study of this figure that the lager stones move under lesser flow stage which corresponds to higher velocity and effective shear stress of flow. It is also observed that at incipient condition, the flow depth increases as the flow discharge increases.

In Fig. 3 variation of dimensionless flow discharge (D_{50}/ y_c) and flow depth (D_{50}/ T_w), both for steady flow and downstream of hydraulic jump are depicted. It was revealed that the relationship between dimentionless flow discharge (y_c/D_{50}) and flow depth (T_w/D_{50}) follows a power function which could be defined as:

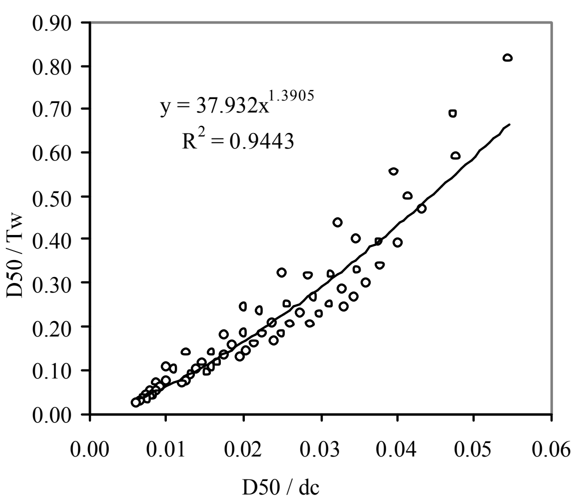
$$\frac{D_{50}}{T_w} = K_1\left(\frac{D_{50}}{y_c}\right)^N \qquad (9)$$

Table 4: Constants in equations (9 and 10)

	Flow	
Factor	Steady	Downstream of hydraulic jump
K_1	1.459	37.932
N	1.449	1.390
R^2	0.992	0.944



(A) Downstream of hydraulic jump

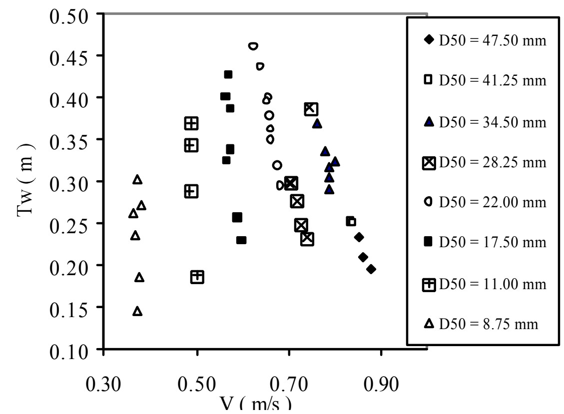


(B) Steady flow

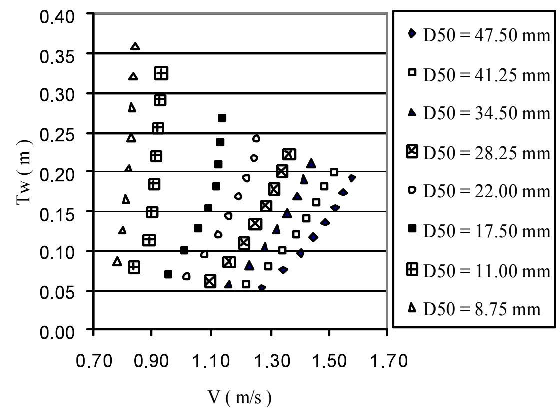
Fig. 3: Variation of nondimentional flow depth with nondimentional discharge at incipient condition of ripraps

Re-arranging the equation (9) will result in an expression which facilitates the calculation of D_{50} with varying discharge and flow depth as:

$$\frac{y_c^{\ N}}{T_w} = K_1 D_{50}^{\ N-1} \tag{10}$$



(A) Downstream of hydraulic jump



(B) Steady flow

Fig. 4: Variation of flow depth with mean velocity at incipient stage

In equations (9) and (10) K_1 and N are constant factors varrieing in steady flow condition and downstream of hydraulic jump as shown in Table 4.

The variation of pertinent mean velocity at constant tail water depth, with riprap size was determined and the results are shows in Fig. 4. It is observed that a direct relationship exists between incipient motion and pertinent mean velocity. It could be also concluded that for a fixed mean velocity, the condition of incipient is directly related to flow depth (i.e.: effective shear stress).

Using the suggested relationships by Chezy, Kolegan, Manning-Strikler and Melville the critical shear velocity (V_{*c}) was determined and then the critical shear stress:

$$\tau_* = \frac{\rho V_{*c}^{\ 2}}{(\gamma_s - \gamma) D_{50}} \tag{11}$$

And shear Reynolds number:

Table 5: Constants in equation (13)

	Flow	
Factor	Steady	Downstream of hydraulic jump
K_2	0.9040	0.0186
M	0.1965	0.2356
R^2	0.9250	0.9340

Table 6: Comparison of computational relations and the observed data

MRE (%)			
Pilarzikh	Peterka (USBR)	Escarmia (Wallingford)	Current Research
9.33	73.8	11.24	8.5

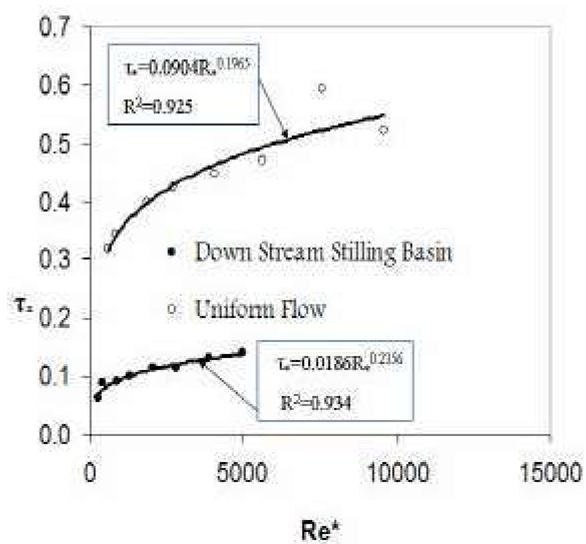


Fig. 5: Variation of t_* with Re_* for observed data

$$R_* = \frac{V_{*c}D_{50}}{\nu} \quad (12)$$

were computed for each flow depth and mean velocity at downstream of hydraulic jump where γ and ρ = specific gravity and mass density of water respectively, ν = kinematic viscosity of water and γ_s = specific gravity of riprap particles.

Based on resulted average critical shear velocities from above mentioned equations, the corresponding shear stress (t_*)and shear Reynolds number (R_{e*}) were then determined for all used riprap sizes and plotted in Fig. 5 In Fig. 5 the observed data in steady flow and downstream of hydraulic jump are also compared. It would be concluded that the variation of t_* and R_{e*} follows a power function as:

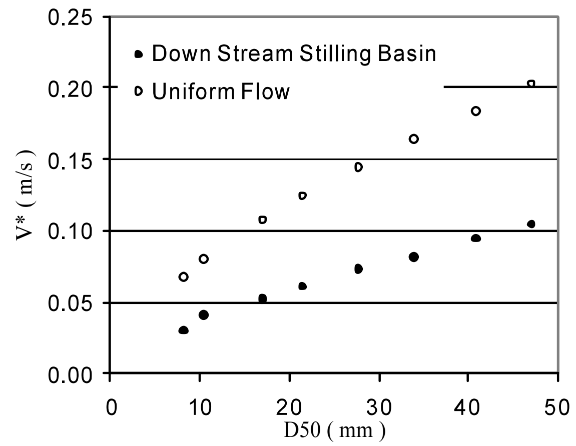


Fig. 6: Relationship between incipient shear velocity V_* and D_{50}

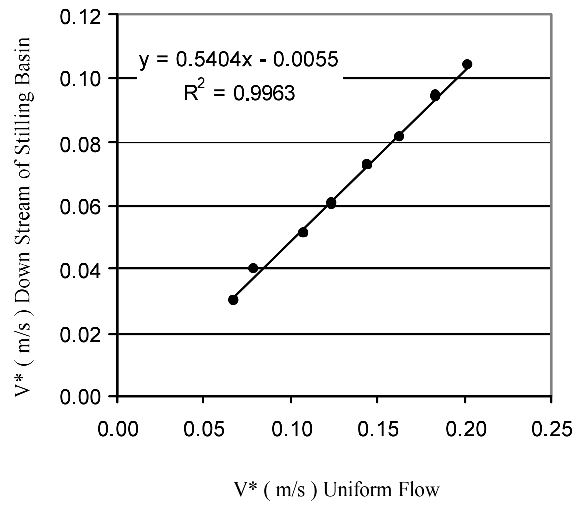


Fig. 7: Comparison of incipient shear velocity with steady flow condition and flow at downstream of hydraulic jump

$$\tau_* = K_2 R_*^{M} \quad (13)$$

where K_2 and M are constants varrieing in steady flow condition and downstream of hydraulic jump as shown in Table 5.

Using equation (13) together with equations (11), (12) and employing trial and error method, one could determine the pertinent size of stable riprap at the end of stilling basins downstream of sluice gates.

In Fig. 6 the relation between V_* and D_{50} is demonstrated for both steady flow and flow at downstream of hydraulic jump. Careful study of Fig. 6 for constant particle sizes was resulted in Fig. 7 which reveals that the incipient shear velocity at steady flow condition is almost twice of that with flow at downstream of hydraulic jump. The observation can be interpreted with the presence of turbulence and local shear stresses which might be still active at the end of hydraulic jump.

For the used ripraps in this research, the incipient conditions (depth, tailwater mean velocity) were determined using the recommended relationships by Peterka [21], Pilarczyk [22] and Wallingford Laboratory [23]. According to the recommendations of these investigators, the following assumptions have been considered :

a) Wallingford equation:

- Turbulence level is high; coefficient of turbulence intensity (TI) is 0.6.
- Since, TI > 0.5, the bottom velocity (u_b) was determined using the following relationship based on average velocity (u_d):

$$u_b = (-1.48TI + 1.36)u_d \tag{14}$$

b) Pilarczyk method:

- Turbulence level is high and the coefficient of turbulence intensity (Kt) equals to 2;

$$\phi = 1.25,\ \Delta = 1.65 \text{ and } \psi_{cr} = 0.035$$

- The flow is highly turbulent with a fully developed velocity profile as:

$$K_h = \left(D_{50} / T_w\right)^{0.2} \tag{15}$$

Where φ is internal friction angle, Δ submerged relative density of particles, ψ_{cr} stability factor and K_h depth factor.

- Apparent mean diameter (D_{n50}) was related to its relevant sieve diameter (D_{50}) by $D_{n50} = 0.9\ D_{50}$ was used which was suggested by Raudkivi (1990).

c) The assumptions mentioned in equation (14) were used for Peterka's method too.

The mean relative error (MRE) values of results obtained from equation 13 and methods of Wallingford, Pilarczyk and Peterka are calculated and compared with experimental data as is shown in Table 6. Equation 13 shows the least error in determining stable riprap diameter while the equations of Pilarczyk and Wallingford fall in a good accordance with the experimental data.

CONCLUSIONS

This research has been conducted to investigate the incipient condition of riprap protection at the end of a stilling basin at downstream of a sluice gate. From the analysis of the results the followings could be concluded:

- Resulted shear velocity from Shields diagram is usually larger than the real value.
- The suggested relationships by Shafaei Bajestan and Niel give true values than those determined by Isbash.
- At a constant discharge, the incipient flow depth decreases as the riprap size increases.
- To determine the required size of stable riprap, knowledge from flow depth and mean velocity would be essential.
- The incipient shear velocity at steady flow condition is almost twice of that with flow at downstream of hydraulic jump which could be attributed to the presence of turbulence and local shear stresses at the end of hydraulic jump.

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REFERENCES

1. Brown, S.A. and E.S. Clyde, 1989. Design of RipRap Revetment. Hydraulic Engineering Circular No, 11, Federal Highway Adminstration. McLean, Va.
2. Maynord, S.T. and J.F. Ruff and S.R. Abt, 1989. Riprap Design. Journal of Hyraulic Engineering, ASCE. Vol: 115 (7).
3. Hydraulic design of flood control channels, 1991. US Army Corps of Engineering. Research. Ctr. Wtrwy. Experimen Station, Vicksburg, Miss.
4. Maynord, S.T., 1993. Corps Riprap Design Guidance for Channel Protection. Preprints of Int. Riprap Workshop Theory. Policy and Practice of Erosion Control Using Riprap. Amour Stone and Rubble. Fort Collins Colorado, USA.
5. Breusers, H.N.C., 1965. Conformity and time scale in two dimensional local scour. Publication 40. Delft Hydraulics Laboratory, Delft, The Netherlands.

6. Farhoudi, J. and K.V.H. Smith, 1982. Time Scale for Local Scour Downstream of Hydraulic Jump. Journal of Hyraulic Engineering, ASCE, Vol: 168 (10).
7. Farhoudi, J. and K.V.H. Smith, 1985. Local Scour Profiles Downstream of Hydraulic Jump. Journal of Hyraulic Research, Vol: 23.
8. Nik Hassan, N.M.K. and R. Narayanan, 1985. Local Scour Downstream of an Apron. Journal of Hyraulic Engineering, ASCE, Vol: 111 (11).
9. Chatterjee, S.S., S.N. Ghosh and M. Chatterjee, 1994. Local scour due to submerged horizontal jet. Journal of Hyraulic Engineering, ASCE, 120 (8): 973-992.
10. Balachandar, R. and J.A. Kells, 1998. Instantaneous water surface and bed scour profiles using video image analysis. Canadian Journal of Civil Engineering, 25(4): 662-667.
11. Balachandar, R., J.A. Kells and R.J. Thiessen, 2000. The effect of tailwater depth on the dynamics of local scour. Canadian Journal of Civil Engineering, 27 (1): 138-150.
13. Dey, S. and B. Westrich, 2003. Hydraulics of submerged jet subject to change in cohesive bed geometry. Journal of Hyraulic Engineering, ASCE 129 (1): 44-53.
14. Sarkar, A. and S. Dey, 2005. Scour hole downstream of aprons caused by sluices. Proc. Inst. Civil Engineering. Water Management, London, 158: 55-64.
15. Dey, S. and A. Sarkar, 2006. Scour Downstream of an Apron Due to Submerged Horizontal Jets. Journal of Hyraulic Engineering, ASCE 132(3): 246-257.
16. Posey, C.J., 1974. Tests of scour protection. Journal of Hyraulic Engineering, ASCE. 100(12), 1773-1783.
17. Parola A.C., 1993. Stability of riprap at bridge piers, Journal of Hyraulic Engineering, ASCE. 119(10), 1080-1093.
18. Chiew, Y.M., 1995, Mechanics of riprap failure at bridge piers. Journal of Hyraulic Engineering, ASCE 121(9): 635-643.
19. Lauchlan, C.S., B.W. Melville, 2001. Riprap Protection at Bridge Piers, Journal of Hyraulic Engineering, ASCE., 127 (5).
20. Worman, A., 1989. Riprap protection without filter layers, Journal of Hyraulic Engineering, ASCE. 115 (12): 1615-1630.
21. Peterka, A.J., 1968. Hydraulic Design of Stilling Basins and Energy Dissipaters. Engineering Monograph No. 25. USBR, USA.
22. Pilarczyk, K.W., 1990. Stability Criterion for Revetments. Proc. of 1990 National conf. on Hyraulic Engineering, ASCE. Chang, H.H. and J.C. Hill (Eds.). San Diego, USA.
23. Escarameia, M., 1998. River & Channel revetment. A design Manual. Wallingford Press, UK.
24. Farhoudi, J. and P. Pourjabbar, 2006. Bed protection criterion downstream of stilling basin based on average velocity of flow. MSc Thesis, Dept. of Irrigation and Reclamation Engineering, Tehran University, Iran.
25. Farhoudi, J. and P. Pourjabbar, 2006. Bed protection criterion downstream of stilling basin based on average velocity of flow. MSc Thesis. Dept. of Irrigation and Reclamation Engineering, Tehran University, Iran.
26. Chiew, Y.M., 1992. Scour Protection at Bridge Piers. Journal of Hyraulic Engineering, ASCE 118 (9): 1260-1269.
27. Shafai-Bajestan, M., 1991. Critical stability number in rock lined channels. Journal of Iranian Agriculture Research, Shiraz University, Iran, 9 (2): 121-138.
29. Maynord, S.T., 1991. Flow Resistance of Riprap. Journal of Hyraulic Engineering, ASCE, 117 (6): 687-696.
30. Melville, B.W. and A.J. Sutherland, 1988. Design method for local scour at bridge piers. Journal of Hyraulic Engineering, ASCE, 114 (10): 1210-1226.
31. Raudkivi, A.J., 1998. Loose Boundary Hydraulics. A.A. Balkema, Rotterdam, The Netherlands.