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ORIGINAL ARTICLE

Application of Bed Sill to Control Scouring Around Cylindrical Bridge Piers

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ABSTRACT: Results are presented from laboratory experiments to investigate the effectiveness of bed sills in reduction of scour depth at cylindrical piers under clear water flow condition. The experiments were performed in a flume with 6.0-m-long, 0.8-m-wide, 0.5-m-deep and zero longitudinal slope. During experiments a bed sill with 10-mm-thick, 0.6-m-length and 0.15-m-height equal to sand bed deep was used. The scour depth in front of the piers becomes negligible as the bed sill placed upstream the piers, but it is to be noted that the scour downstream of the piers in this case is more than when the bed sill placed the downstream of the piers, i.e., risk of reduction of bed surface because of local scour and particularly general scour will be increased as the bed sill placed upstream the piers. The best location for the bed sill is suggested downstream and about snap of the piers. In this case the reduction percent of scour depth in front of the piers is about 29% in the best configuration.

Keywords: Scouring, Cylindrical Piers, Bed sill, Scour reduction

INTRODUCTION

Scour is defined as the erosion of streambed sediment around an obstruction in a flow field [1]. On the other hand scour is a natural phenomenon caused by the erosive action of flowing water on the bed and banks of alluvial channels [2]. In the United States, general and local scour are the major cause of hydraulic factors. Stream instability, long-term streambed aggradations or degradation, general scour, local scour and lateral migration cause 60% of all U.S. highway bridge failures [3]. The methods used for control of scouring around bridge piers can be divided into two types: (1) Direct methods, through increasing streambed resistance. This is usually done by utilizing riprap, geobag, cable-tied blocks and etc around the piers [4-6]. (2) Indirect methods. In these methods, the flow pattern around the piers is changed to reduce shear stresses on the riverbed, and therefore reduce the depth of the scour hole. Flow altering devices that have been used to protect piers against local scour include sacrificial piles, Iowa Vanes, collars and slots [7-9]. The indirect method can be more economical, especially when the required amount of stone is not available near the bridge site.

Mechanism of scouring

The flow pattern and mechanisms of scouring around a bridge pier are very complex and have been reported by various investigators [10-13]. The vortex is generated by a combination of main flow at the upper stream portion of the cylinder and potential flow in the downstream direction along the front surface of the cylinder. The size of local scour is related to strength and size of the vortex flow [14]. Local scour around a pier results from the down flow at the upstream face of the pier and the horseshoe vortex at the base of the pier [15,16], this vortex is often referred to as horseshoe vortex because of its great similarity to a horseshoe [17]. Separation of flow at the sides of the piers also creates the so-called wake vortices [10]. These vortices are unstable and shed alternatively from each side of the pier, however in choosing the elevation of piers foundation, only maximum depth of scouring is important [18]. Various investigators used the bed sills as a common solution to stabilize degrading bed rivers [19-21]. Often such structures are used to control erosion in the vicinity of bridge piers and abutments or in channels downstream of dams. Grimaldi et al. [22] used the bed sill downstream of the piers in order to reduce the scour depth in front of the piers as well. Their results are presented in the following pages. The main aim for this study was to find an appropriate location for bed sill around bridge piers. The bed sill was placed in three location; downstream, middle and upstream of the piers, respectively. For first two location of the bed sill at piers, bed sill acts as an obstacle against the wake vortices and the wake vortices loses its strength to lifting the materials and transport to downstream the piers. When the bed sill placed in front of the piers, it is up to as an obstacle against the horseshoe vortex.

Framework for Analysis

To avoid wall effect on scouring, pier diameter should not be more than 10% of flume width [6], already Raudkivi [11] had offered that for avoiding wall effect on scouring, the distance ration of pier axis until flume wall to pier diameter should be bigger than 6.25. By the way in order to unaffected the particle size on the depth of scour hole should be b/ d_{50}>20-25. If the so-called "pier Reynolds number" R_p=U_b / v, is bigger than 7000,

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it doesn't affect on local scouring process [22]. To negligible flow depth effect on scouring, should be h/b>3.

The relation between the depth of local scour at bridge pier d_s and its dependent parameters can be written:

$$h_s = f(g,\rho,\upsilon,U,h,B,\rho_s,d_{50},b,L,t) \qquad (1)$$

Choosing as basic variables and Buckingham theorem, we have:

$$\frac{d_s}{b} = f_1\left(\frac{U}{\sqrt{gb}},\frac{Ub}{\upsilon},\frac{h}{b},\frac{B}{b},\frac{\rho_s}{\rho},\frac{b}{d_{50}},\frac{L}{b},\frac{Ut}{b}\right) \qquad (2)$$

Where $U/(gb)^{0.5}$ = so-called "pier Froude number" F_p; and □s/□= sediment specific gravity, being constant for sand and gravel (equal to about 1.65). In accordance with the above conditions, Equation (2) can be simplified as follows:

$$\frac{d_s}{b} = f_2\left(F_p,\frac{L}{b},\frac{Ut}{b}\right) \qquad (3)$$

At equilibrium, i.e., when the dimensions of scour hole remain constant, Equation (3) becomes:

$$\frac{d_{se}}{b} = f_3\left(F_p,\frac{L}{b}\right) \qquad (4)$$

MATERIALS AND METHODS

The experiment reported herein was conducted in a rectangular flume, with 6.0-m-long, 0.8-m-wide, 0.5-m-deep and longitudinal slope equal to zero (Fig.1). The test area of the flume is made up of an iron bottom and glass sidewalls along two sides for most of its length to facilitate observations. In upstream and downstream of the flume two false bottoms was installed with 2-m-long and 0.15-m-high. Some coarse sand was spread on upstream of false bottom in order to distribute the inflow uniformly. At the end of this flume a controlling gate was designed to adjust the water surface height at the desired levels. All the experimental tests were conducted under the same flow depth and discharge equal to 0.19 m and 41 lit/s, respectively.

The sediment used in all tests was sand, its grain size distribution curve is shown in Fig.2, with median sediment diameter d_{50}=0.45 mm and geometric standard deviation of the grain size distribution equal 1.48. d_{50} and geometric standard deviation were selected to maintain clear water condition without formation of ripple. Since the uniformity coefficient $C_u=(d_{60}/d_{10})=1.56$ is less than 2, the sand can be considered as uniform [8].

The decreasing rate of sediment supply into the scour hole from upstream leads to an increased scour depth [11], therefore in flume no sediment feeding was provided during the experiments.

Cylindrical pipes with diameters of 18 mm, 30 mm and 60 mm were used as piers model. Piers diameter were selected so that the effect of sediment size and flume sidewalls on the depth of scour becomes negligible. The bed sill in tests was 10 mm thick plastic plate, as long as the working cross section. It was placed flush and deep with bed, downstream, middle and upstream of the piers, respectively.

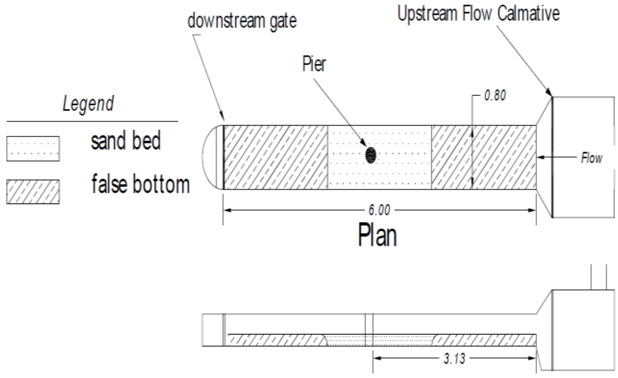

Figure 1. Flume - plan view and longitudinal profile

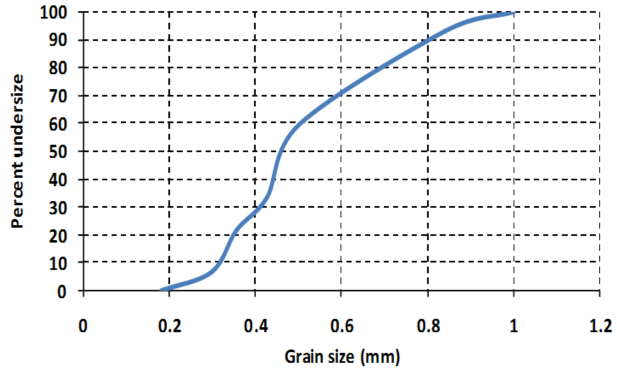

Figure 2. Grain size distribution curve of the sediment used in tests.

The working section in which bed sill and piers were placed, was about 3.0-m downstream from the entrance of the flume. The piers were first placed in the flume at the desired location. For each test the bed sill was placed at different location of the piers. To start the test, the flume was slowly filled with water to the desired flow depth. It should be noted that too care is require when filling the flume no sediment movement is allowed. The pump was then started and the downstream gate slowly opened and was adjusted until the desired flow rate had been achieved. Development of the scour depth in front of piers was measured by a transparent ruler installed in front of piers.

A point gauge with 0.1 mm accuracy was used to measure the depth of the scour hole around piers. In clear water scour the bed material upstream of the scour area is at rest and the maximum scour depth is reached when the flow can no longer remove particles from the scour hole. For each test the study approach flow was adjusted so the velocity to critical velocity ratio U/Uc was equal to 0.9, on the center line of the flume. The critical velocity was estimated form Melville's equations [6] as follows:

$$U_c/U_{*c} = 5.75\log(5.53h/d_{50}) \qquad (5)$$

$$U_c = 0.0115 + 0.0125 d_{50}^{1.4}$$
$$0.1\,mm < d_{50} < 1\,mm \qquad (6)$$

Where U_c is in m/s and d_{50} is in mm. One Froude number of 0.199 was applied in order to investigate the effect of flow conditions on the scouring. At the end of each test, the pump was shut down to allow the flume slowly drains without disturbing the scour topography, and the conclusion the hole of scour was recorded by

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point gauge. Experiments were carried out at hydraulic laboratory of university of Tabriz (Iran) during Jan 2010 to Mar 2010.

Duration of scour test

It is presently established that local scour depth increases progressively with time and reaches equilibrium. Equilibrium scour occurs when the scour depth does not change appreciably with time. Various criteria have been reported in the literature in order to identify the equilibrium state. Melville [7] defined the time equilibrium as the time at which the scour hole develops to a depth (the equilibrium depth, d_{se}) at which the rate of increase of scour does not exceed 5% of the piers diameter in the succeeding 24 hr period, i.e., $d(d_s)/dt<0.05b/24hr$. Cardoso and Bettess [23] defined the time equilibrium in the flowing way by plotting the scour depth against logarithm of time a change of slops is identified in the plot and scour depth follows an almost horizontal attitude. Experiments were carried out until the rate of scouring was negligible. The temporal variation of scour was measured and the runs were carried out until such time as the scour did not change by more than 1 mm over a period of 3 hours [14].

RESULTS AND DISCUSSION

Preliminary tests: Piers without any countermeasures

One long-term experiment was carried out in one of the tests of each pier without countermeasure to study the development of the scour hole and to establish a basis for comparison. The results of the scour depth in front of the piers were indicated in Fig.3. The equilibrium clear water scour depth is always < 2.3 times the pier diameter while the equilibrium live-bed scour depth is always < 2.0 times the pier diameter [18]. Fig. 3 corroborated that the experiments were carried out at clear water condition. Whole of the present experiments were performed under clear water conditions at one flow intensities of 0.9.

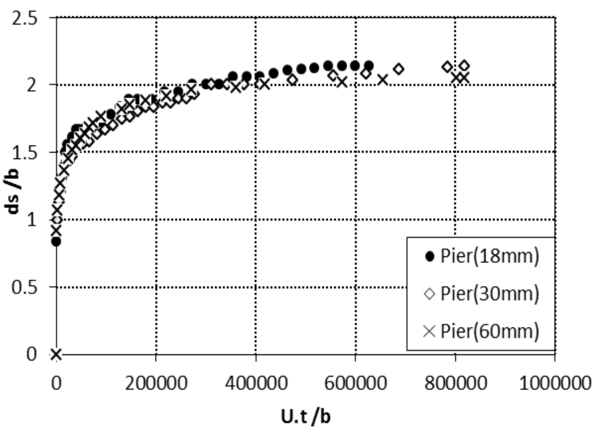

Fig. 3 Dimensionless time evolution of the scour depth in front of piers

The curves of scour depth were dimensioned relative to the piers diameter so that indicates same experiment condition and scour depth for piers. The size of local scour is related to strength and size of the vortex flow, i.e., the larger piers is the bigger vortex flow [19]. By the way the time equilibrium is related to the piers diameter, i.e. the bigger piers diameter has long time equilibrium [6].

The ratio of scour depth in front of the piers to piers diameter with diameters of 18 mm, 30 mm and 60 mm were achieved in the values about 2.13, 2.14 and 2.1 in 13, 25 and 50 hours, respectively, i.e. the larger the piers had larger scour depth and the longer the time is required to achieve the equilibrium. The deepest scour holes in these tests were at the edge and in front of the piers (see Fig. 4).

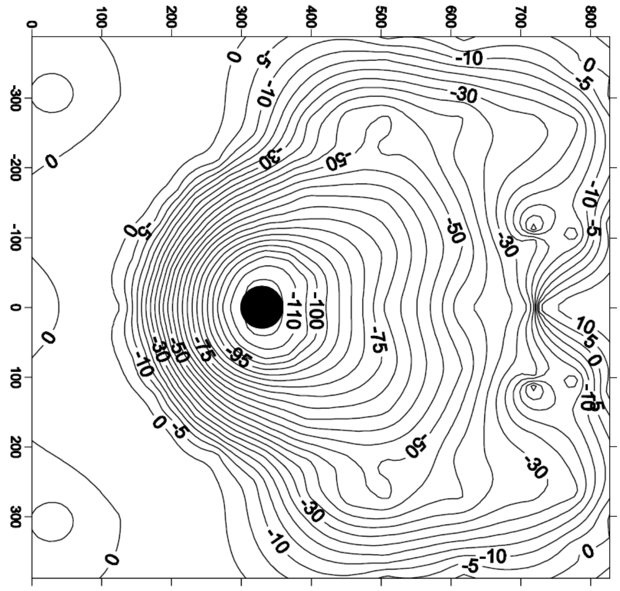

Fig. 4 Scour pattern around a pier without countermeasure
(Numbers in mm and flow from left to right)

Experiments with the bed sill in downstream of the piers

The bed sill was located downstream of the piers in distances of L=0, b/2, b, 1.5b, respectively. Fig.5 indicates the increase of scour depth in front of the piers because of increase in distance between the bed sill from the piers. These results were achieved already by Grimaldi et al. [22]. Their investigations indicated that the smaller the distance between the piers and the bed sill, the larger the effectiveness of this countermeasure about 26% in scour reduction in front of the piers.

The interaction between flow, bed sill and piers produce three local scours, first one in front of the piers and bed sill, and other two holes downstream of the bed sill. About two holes downstream of the bed sill, a possible hypothesis reported by Graf and Istiarto [24] is that; they could have been generated by the lower part of the wake vortex system. Minimum distance between the piers and the bed sill has maximum scour hole in downstream of the bed sill. Because large part of the wake vortices will be transferred to downstream of the bed sill as it has shorter distance with the piers. When the distance of the bed sill and the piers is large the wake vortices placed between them and downstream scour holes of the bed sill will be reduced.

The scour pattern for two situations of bed sill; (a) L=0 and (b) L=1.5b were indicated in Fig.6a,b, respectively. Figure 6 shows that the maximum downstream scour depth of the bed sill in case of L=0 is more than as the bed sill placed in distance L=1.5b of the piers. This problem is the result of that the portion of the wake vortices which is led to downstream of the piers in case of L=0. But by increasing distance of the bed sill from the piers, the whole of the wake vortices would be placed between the piers and bed sill, finally the downstream scour of the bed sill will not be more affected by the wake vortices.

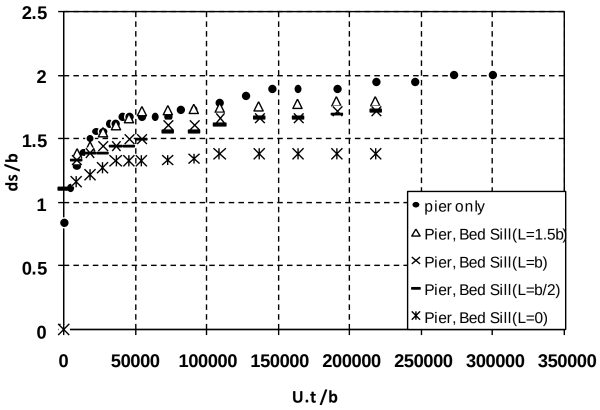

Figure 5. Dimensionless time evolution of the scour depth in front of piers for different locations of bed sill in downstream of the piers

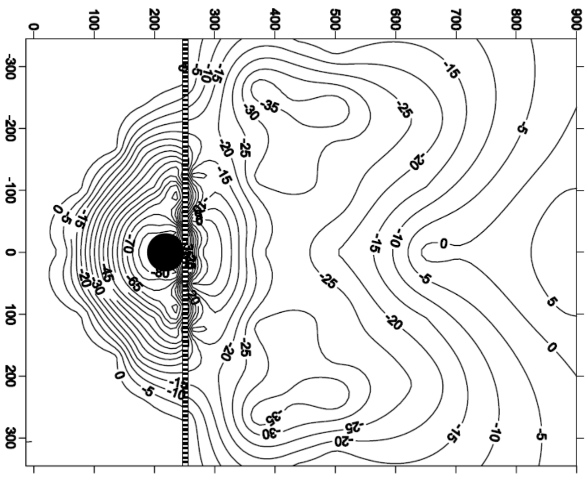

Figure 6a. Downstream scour depth of the bed sill in case of L=0

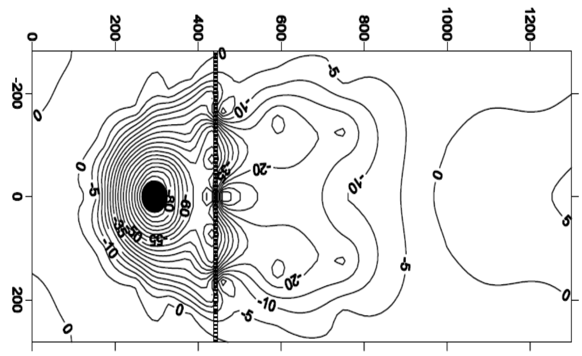

Figure 6b. Downstream scour depth of the bed sill in case of L=1.5b

Experiments with the bed sill at the middle of the piers

In continuing the tests, according to Fig.7 the bed sill was cut until the piers were placed between the bed sill. i.e., the bed sill was placed in L= -b/2. Fig.8 indicates the scour depth in front of the piers when bed sill is in L=0 and L= -b/2. As is found from figure by aligning the bed sill in L= -b/2 the scour depth in front of the piers is less than the bed sill in downstream and stick to the piers. Physical justification of this phenomenon is that once the bed sill transported across the upstream, this makes to front of wake vortex and horseshoe vortex soon blocked.

Fig. 9 shows scour depth comparison in front of piers for bed sill in modes of L=0 and L= -b/2. It is observed that scour process for piers is identical with bed sill in L=0, but the conclusion about the exposure of bed sill in L= -b/2 is different. The ratio of b/2 in bigger piers is more than smaller piers, so we can say that the effect of bed sill in L= -b/2 will be more at greater piers.

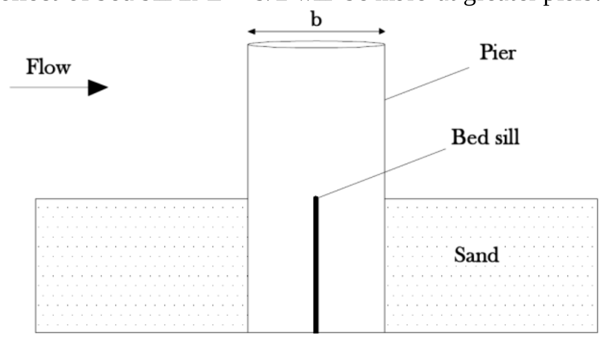

Figure 7. Bed sill installed as a countermeasure in location of L= -b/2

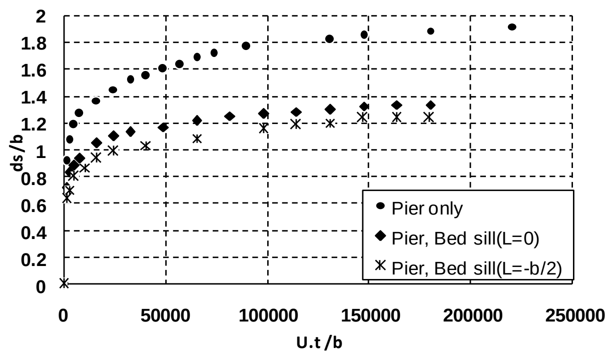

Figure 8. Dimensionless time evolution of the scour depth in front of piers for bed sill locations in L=0 and L= -b/2

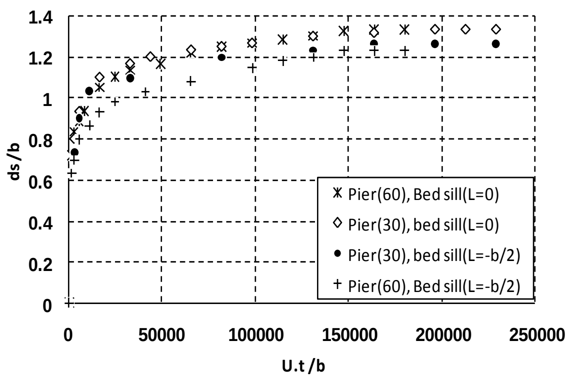

Figure 9. Dimensionless time evolution of the scour depth in front of the piers with countermeasure

Considering the curve of Fig. 9, the maximum scour depth in front of the piers is equivalent about to 1.4b when the bed sill is placed in L=0, i.e., the minimum installation depth for bed sill can be considered equivalent to maximum scour depth in front of the piers.

According to Fig.10b greater part of wake vortices moved to downstream of the piers that resulting an increase downstream scour of the piers is. i.e., the risk of local and general scour in downstream of the piers will be more with transfer the bed sill to upstream. On the other hand used bed sill length in L= -b/2 is less than L=0, hence duo to economic issues the placement of bed sill in the range of L= 0 to -b/2 should be study more and many researches are needed in order to identify the flow field around the piers and bed sill.

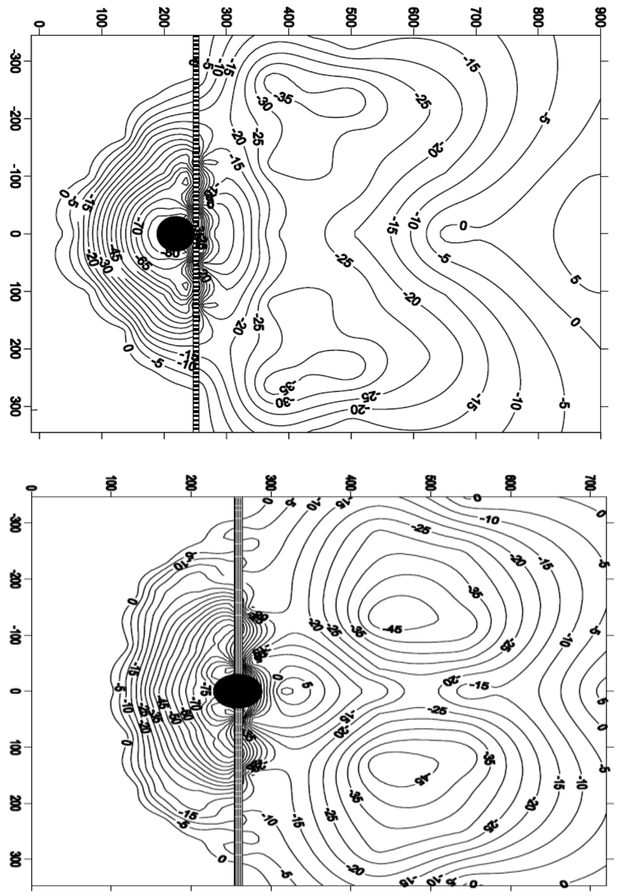

Fig. 10. Scour pattern for two cases (a) L=0 and (b) L= -b/2
(Numbers in mm and Flow from left to right).

Table 1. The reduction percent of scour depth in front of the piers

L / Size of the piers (mm)	L= -b/2	L=0	L=b/2	L=b	L=1.5b
18	32	29.5	16	12	11
30	33	29	15	14	11
60	35	29.2	19	13	12

The results of different position of the bed sill have indicated in table 1. Scour reduction percent in front of piers were achieved about 29% in the best configuration. To investigate the relation expressed through Eq. (4) the dimensionless equilibrium scour depth in front of the piers d_{se}/b were plotted versus F_p, with U and U/U_c kept approximately constant (Fig. 11). As U is approximately constant, Fig. 11 in effect indicates how d_{se}/b varies with piers constant, b. The data suggest that larger values of d_{se}/b occur for smallest piers tested, b=18 mm.

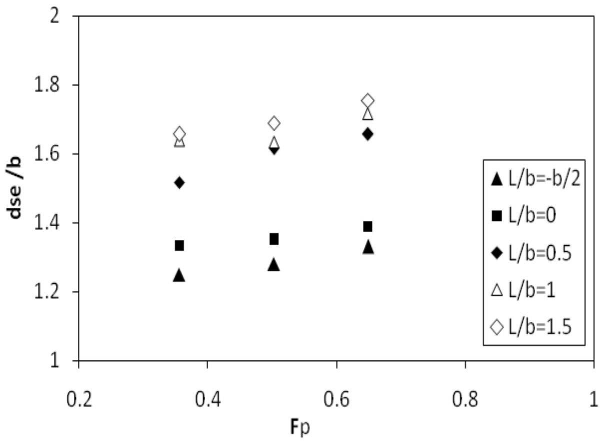

Fig. 11. Variation of d_{se}/b with F_p

Experiments with the bed sill in upstream of the piers

In continuing the tests the bed sill was placed upstream the piers i.e., in position of L= -b. With insert the bed sill in this position for each pier, it was found that scour depth in upstream side of piers is much smaller than pervious modes, because in this position for the bed sill horseshoe vortices couldn't more continue to upstream the piers. On the other hand since downstream the piers was without any countermeasure therefore big portion of wake vortices washed particles as a downhill and this makes the downstream local and general scour of the piers to be more, so in this position the downstream scour depth of piers will be so close to the piers (Fig. 12).

Fig. 12 indicates scour pattern for bed sill in L= -b. By comparing the figures 10 and 12 can be concluded that scour risk in downstream of the piers whatever bed sill to be transferred to upstream the piers will be more. The scour around a pier develops upstream to and downstream of the pier itself owing to the action of both the horseshoe and the trailing vortices.

The bed-sill mainly acts like a flow separator: if the bed sill is located upstream to the piers the trailing vortex can act behind the piers, which is directly exposed. In the case of bed-sill located downstream of the piers the piers is not directly exposed to the trailing vortex action. Note that, in this second case, behind the bed sill, the scour hole does not develop so close to the bed-sill itself and the scour depth is always lesser than in the case of unprotected piers. On the other hand whit position of the bed sill in upstream of the piers occur amount scour in upstream the bed sill.

Becomes depleted in front of bed sill, hydrostatic pressure in front of this structure will be more. Also being of general and local scour in downstream of bed sill (L= -b), the foundation stability in location of piers becomes loose and will be existence an add force from bed sill to piers that threat piers stability. So bed sills upstream to bridge are commonly avoided.

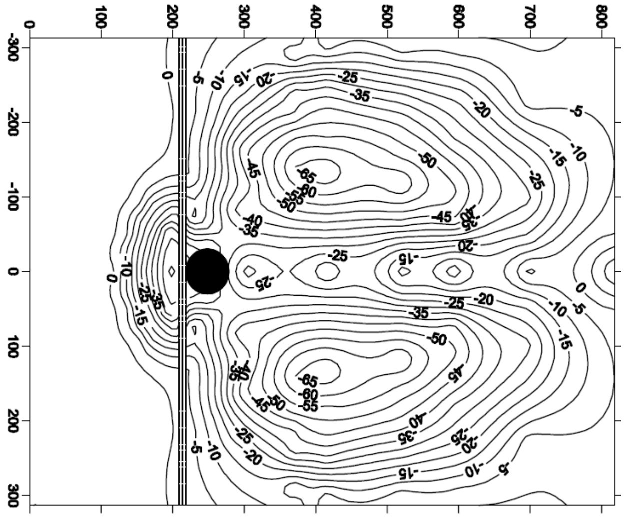

Figure 12. Scour pattern for case of bed sill in L= -b (Numbers are in mm and flow is from left to right)

Notation

B = top width of the flume cross section;

b= piers width;

d_s = scour depth in front of the piers;

d_{se} = equilibrium scour depth in front of the piers;

d_{10} = grain size for which 10% by weight of the sediment is finer;

d_{16} = grain size for which 16% by weight of the sediment is finer;

d_{50} = median grain size of the sediment;

d_{60} = grain size for which 60% by weight of the sediment is finer;

d_{84} = grain size for which 84% by weight of the sediment is finer;

F_p=piers Froude number;

R_p=piers Reynolds number;

g = acceleration due to gravity;

h = approach flow depth;

L = distance between piers and downstream bed sill;

t = time;

U = approach flow velocity;

U_c = mean approach velocity at the threshold condition;

U_{*c} = mean approach velocity in the flood channel at the threshold condition;

v = water kinematic viscosity;

ρ = water density;

z_g=geometric standard deviation of the grain size distribution.

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