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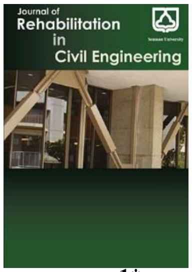

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On the Local Scour Around Group Piers in Series by Experimental Tests

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ABSTRACT

In this article a physical model is presented. A trapezoidal shape of channel with 1.45 m width, 1 meter depth and banks slope of 1H: 1.5V was applied. Circular piers (6, 8 and 10cm diameter) were examined under three dissimilar flow discharges of 50, 65 and 80 lit/s and three different median bed material sizes equal to 0.94, 1.31 and 2.12 mm. Seven different longitude distance to pier diameter ratios (1, 1.5, 2, 2.5, 3, 4 and 5) were examined. Measurement of scour depth (upstream, around and downstream of pier) was accomplished in 10, 15, 20, 30, 60, 120, 240 and 360 minutes from the beginning of each examination. The results indicated that 30%-40%, up to 65% and more than 90% of local scour occurs at first two, 10, 120 and 200 minutes from the beginning of test respectively. The effect of dissimilar parameter especially the distance between piers on scour depth at group piers in series was evaluated and the results were compared to other researchers. It was found that our results is best fitted with the general form of well known Sheppard et al. (2004)'s equation.

1. Introduction

Local scour around the bridge piers is a fast hydraulic-sediment interaction phenomenon which can create a deep hole around bridge pier in a very short period of time during flood season. The phenomenon believes to be the main cause of bridge failure around the world. The flow pattern around the bridge is a three dimensional. A horse shoe vortex which is developed as a result to the down flow created by the stagnation pressure upstream of the bridge can pick-up bed sediment particle and transported further

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downstream by the main flow. The stronger horse shoe vortex resulted in a deep hole. Many parameters have been recognized to affect the horse shoe vortex which includes: the approaching flow characteristics (steady or unsteady flow conditions, flow velocity and flow depth), the geometry of the pier (width, shape and orientation to the flow direction), bed sediment particles (size, density, sediment load) and fluid properties (density and viscosity). During the past decades bridge scour depth studies have attracted the attention of many researchers [1-20]. For more details the results of these studies have been summarized in HEC23..

Since the mechanism of local scour around group piers is quite complicated, few studies have investigated this subject [4, 21-32]. The above mentioned studies resulted in many scour predicted formula which some of them are presented here for comparison our results. These formulas are as follow:

Melville [9]'s equation, as follow:

$$\frac{d_s}{b} = K_L K_y K_d K_s K_\theta \qquad (1)$$

Where: d_s is depth of local scour (m) and b is width or diameter of pier (m). Different expressions (K) are applied for evaluation of the various effective parameters on scour depth.

K_y (depth- size factor for piers) is computed by below equation:

$$K_y = \begin{cases} 4.5 y_1/b & For \quad y_1/b < 0.2 \\ 2\sqrt{y_1/b} & For \quad 0.2 < y_1/b < 1.4 \\ 2.4 & For \quad y_1/b > 1.4 \end{cases} \qquad (2)$$

y_1 is depth of flow (m).

K_L (flow intensity factor) is calculated by below equation:

K_L= $[U-(V_a-V_C)]/V_C$ For $[U-(V_a-V_C)]/V_C<1$ Otherwise $K_L=1$ (3)

U is flow velocity and V_C is critical velocity for mean grain sizes (m/s) and $V_a=0.8V_{ca}$.

V_{ca} is critical velocity for maximum grain size (m/s) ($d_{max}=\sigma_g^m d_{50}$). The values of m are given in Table 1.

Table 1. The values of m (Melville, 1997)

Assumed d_{max}	d_{90}	d_{95}	d_{98}	D_{99}
M	1.28	1.65	2.06	2.34

K_d (sediment size factor) is computed by below equation:

$$K_d = 0.57\log\left(2.24\frac{b}{d_{50}}\right) \quad \text{For } (b/d_{50})<25$$

otherwise $K_d=1$ (4)

The values of K_s (shape nose of pier factor) are illustrated in Table 2.

Table 2. The values of K_s (Melville, 1997)

Shape	Sharp nose	Round nose	Square nose
K_s	0.9	1	1.1

The values of K_θ (pier alignment factor) are depicted in Table 3.

Table 3. The values of K_θ (Melville, 1997)

l/b	K_θ (θ=0)	K_θ (θ=15)	K_θ (θ=30)	K_θ (θ=45)	K_θ (θ=90)
4	1	1.5	2	2.3	2.5
8	1	2	2.75	3.3	3.9
12	1	2.5	3.5	4.3	5

l is length of pier (m) and θ is attack angle of flow to pier.

Ettema et al. [11]'s equation, which is:

$$\frac{d_s}{D}=\left(\frac{y_1}{D}\right)^{0.62}\left(\frac{U}{\sqrt{gh}}\right)^{0.2}\left(\frac{D}{d_{50}}\right)^{0.08} \tag{5}$$

D is diameter of pier (m) and g is gravity acceleration (m/s^2).

3- Richardson and Davis [14]'s equation as follow:

$$\frac{d_s}{b}=2K_sK_\theta K_bK_a\left(\frac{y_1}{b}\right)^{0.65}Fr^{0.43} \tag{6}$$

Where: Fr is Froude number. The values of K_b are given in Table 4.

Table 4. The values of K_b (Ettema et al., 1998)

Bed condition	Dune height, *m*	*Kb*
Clear-water scour	N/A	1.1
Plane bed and anti dune	N/A	1.1
Small dune	0.6 < H < 3.0	1.1
Medium dune	3.0 < H < 9.0	1.1 to 1.2
Large dune	9.0 < H	1.3

K_a is calculated by below equation:

$K_a=0.4(V_R)^{0.15}$ For $d_{50}>2$ mm

otherwise $Ka=1$ (7)

$$V_R=(U-V_{ic\,d_{50}})/(V_{c\,d_{50}}-V_{ic\,d_{95}})>0 \tag{8}$$

$$V_{ic\,d_{50}}=0.645\,(d_{50}/b)^{0.053}\,V_{c\,d_{50}} \tag{9}$$

$$V_{c\,d_{50}}=K_u\,y_1^{1/6}d_{50}^{1/3} \tag{10}$$

$$V_{ic\,d_{95}}=0.645\,(d_{95}/b)^{0.053}\,V_{c\,d_{95}} \tag{11}$$

$$V_{c\,d_{95}}=K_u\,y_1^{1/6}d_{95}^{1/3} \tag{12}$$

K_u is equal to 6.19 in SI system and 11.17 in English system. Minimum K_a is 0.4.

4- Sheppard et al. [16]'s equation, which is:

$$\frac{d_s}{D}=2.5f_1\left(\frac{y_1}{D}\right)f_2\left(\frac{U}{V_C}\right)f_3\left(\frac{D}{d_{50}}\right) \tag{13}$$

$$f_1\left(\frac{y_1}{D}\right)=Tgh\left(\frac{y_1}{D}\right)^{0.4} \tag{14}$$

$$f_2\left(\frac{U}{V_C}\right)=1-1.75\left(Ln\left(\frac{U}{V_C}\right)\right)^2 \tag{15}$$

$$f_3\left(\frac{D}{d_{50}}\right)=\frac{\frac{D}{d_{50}}}{0.4\left(\frac{D}{d_{50}}\right)^{1.2}+10.6\left(\frac{D}{d_{50}}\right)^{-0.13}} \tag{16}$$

At present study the effect of distance between piers in group piers in series on scour depth has been experimentally examined in a relatively large channel under different flow conditions, sediment material, and cylinder piers with different size. Because the flow direction is perpendicular to the piers in experimental tests of this research, the attack angle of flow to pier is not an effective factor. Moreover, width of constructed flume is not sufficient for putting parallel piers and effects of flume walls can reduce accuracy of experimental tests significantly. The accuracy of the above mentioned relations was examined by the experimental data and a modified correlation factor was developed for the existing formula of Sheppard et al. [16].

2. Materials and Methods

2.1. Experimental Tests

Experimental tests were at the water research institute of Iran. Water was supplied by four pumps from underground reservoir. A basket filled with rocks is located at the upstream section of the flume to create more uniform flow within the flume. The cross section of the main flume is trapezoidal with 1.45 m width, 1 m deep and bank slope of 1.5V:1.0V. Flow discharge is control by an inlet butter fly valve and measured by a V-notch weir. A system of rail was installed in order to carry the required apparatus such as point gage for water surface and bed level measurements and flow velocity meter (Valeport 801 made in England). The plan and section of constructed experimental flume is illustrated in Fig. 1.

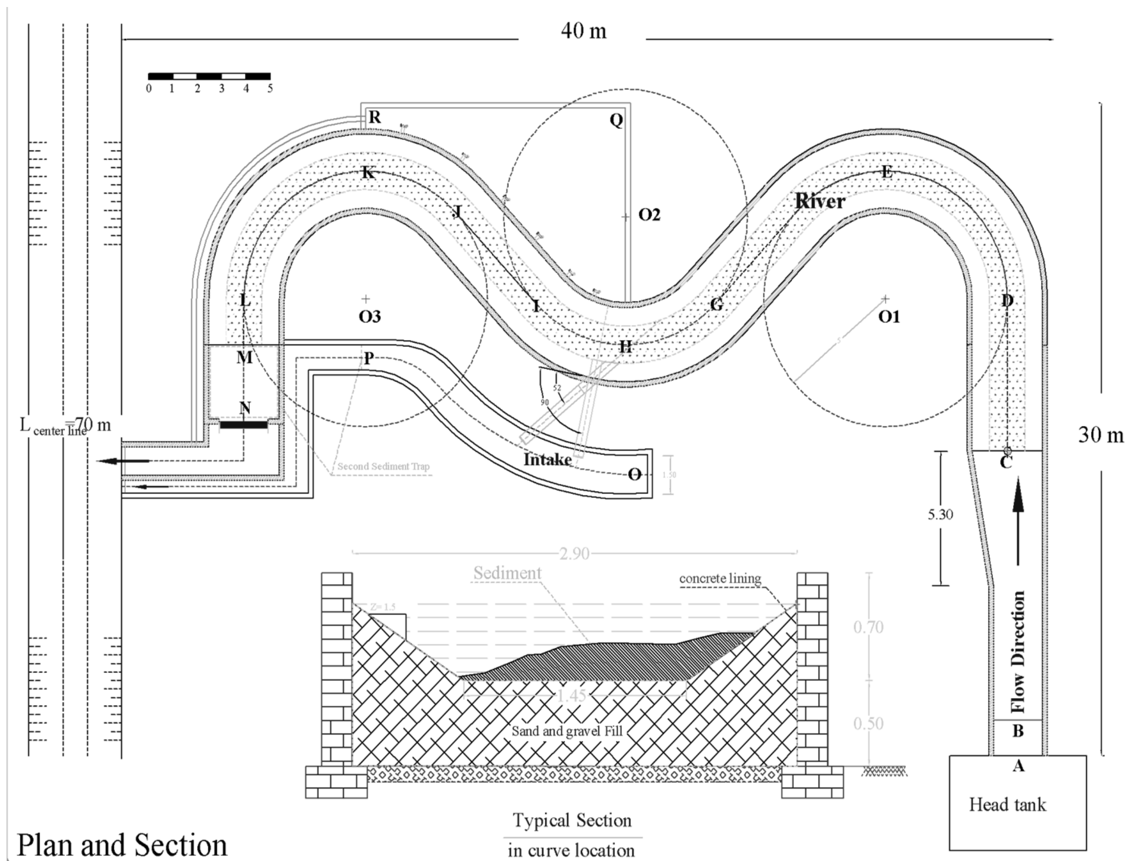


Fig. 1. The plan and section of constructed experimental flume.

Three different sizes of uniform sands ($\sigma_g = \sqrt{\frac{d_{84}}{d_{16}}}$ <1.3 with median sizes of 1.31, 1.47 and 2.12 millimeters) were applied. Ripple forming occurs for non-cohesive alluvial sediments that their particle size is 0.05 -0.7 mm. For these particles bed shear stresses are slightly greater than the threshold value for the initial of motion. With uniform ripple forming sands the scour depth is smaller than with non-ripple forming uniform sediments. Since it is nearly impossible to maintain a flat sand bed at near threshold conditions; thus ripples develop, and a small amount of general sediment transport replenishes some of the sand scoured at the pier. Thus, true clear-water scour conditions cannot be maintained experimentally. Therefore, sediment sizes selected to be more than 0.78 mm in order the bed forms do not develop [33]. Diameters of cylindrical piers are 6, 8 and 10 cm. Group piers (three piers) in series is presented in Fig. 2 schematically. Bridges are built on several piers that they are in series. This research evaluates effects of different parameters as flow discharge, pier diameter and etc on scour depth around bridge piers. For this purpose, piers were classified into three categories (front pier, middle piers and rear pier). First, second and third piers are representatives of front pier, middle piers and rear pier respectively.

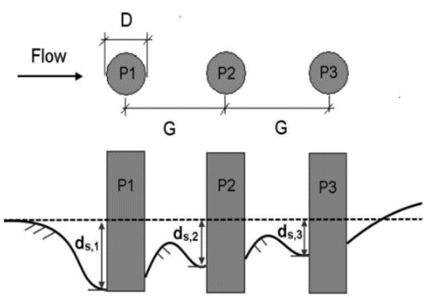


Fig. 2. Group piers

For evaluation of effects of distance between piers a number of tests were performed for bed sediment size of 1.31 mm. Table 5 indicates the characteristics of these tests.

Table 5. Characteristics of performed tests for evaluation of effects of group piers

Flow Discharge (lit/s)	50	65	65
Diameter of pier (cm)	6	6	10
Distance between piers ($\times D$)	1, 1.5, 2, 2.5, 3, 4 and 5	1, 1.5, 2, 2.5, 3, 4 and 5	1, 1.5, 2, 2.5, 3, 4 and 5

Before start of each test, the channel bed was filled with sediment (25 cm thick) and its bed surface was leveled and piers were located at the desired location in the centerline of the flume. Consequently, flow was allowed to enter the flume gradually to prevent the entrainment of the bed sediment. At this time the downstream gate is fully closed. When the flow depth within the flume is high enough the flow discharge increase gradually and at the same time the tail gate was opened. Upon reaching the desired flow discharge, the tail gate was opened further to have the desired flow depth within the flume. On the other word before discharge flow is fixed, flow depth is high in such a way that no scouring occurs. After discharge flow is fixed, flow depth decreases and scouring occurs. Depth scour around piers is measured after discharge flow is fixed. Such condition was kept constant for several hours. During each examination the water surface level and the scour depth in front of first pier was measured periodically. At the end, topography of bed was measured applying an electronic bed profiler.

3. Results and Discussion

Time development of local scour around piers:

The measured scour depth at different time interval was plotted versus time for different tests (Fig. 3). As it can be observed 30% to 40% of total scour depth occurs at first 2 minutes; 55% to 65% of scour depth is developed after 10 minutes and more than 90% of scour depth occurs after 120 to 200 minutes from the start of test. Melville and Chiew [12] stated that at equilibrium time and after this time, variation of depth of local scour is less than 5% of diameter of pier in each time interval. In these examinations, equilibrium time is distinguished that is equal to 6 hours. As an example time variations of depth of local scour in front of the first pier are presented in Fig. 3.

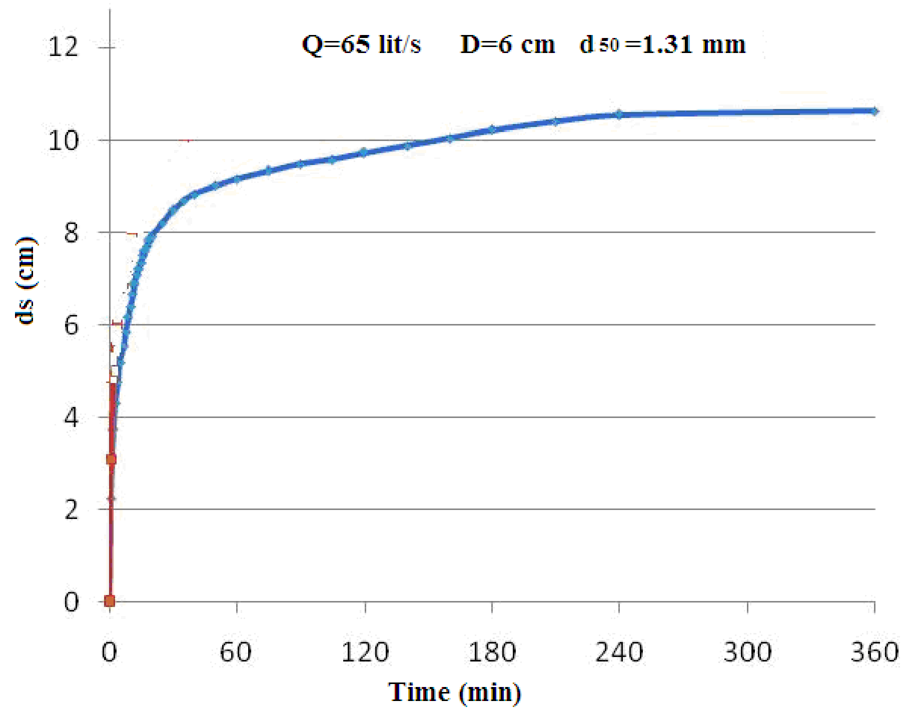


Fig. 3. Comparison between time variations of depth of local scour in front of the first pier

Evaluation of effects of diameter of pier on depth of local scour:

The results revealed that by increasing the size of pier, the scour hole dimensions especially its depth increasing; this is due to the size and the strength of horse shoe vortexes increases. Maximum depth of local scour in front of front pier is given in Table 6 for different tests.

Table 6. Maximum depth of local scour in front of the first pier

Q (lit/s)	50	65	80
D (cm)		H_{max} (cm)	
6	9.08	10.63	11.65
8	11.07	12.94	14.73
10	13.42	15.33	16.92

Evaluation of effects of distance between piers on depth of local scour:

As it was stated earlier, the effect of longitude distance to pier diameter ratio (G/D) has been investigated in this study. Fig. 4 depicted the variations of normalized scour depth (d_{Sf}/d_{Sm}) versus longitude distance to pier diameter ratio (G/D).

Where d_{sf} is depth of local scour in front of front pier and d_{sm} is depth of local scour in front of middle pier.

The results suggest that when the ratio of G/D is equal one, the scour depth in front of other piers is minimum value. The scour depth at each pier compare to the scour at first pier is increase as the distance between the pier increases and reaches to a maximum when the G/D ratio is about 2.2 to 2.5. The results reveal that when G/D=1, the scour depth in front pier is almost 5-12% is larger than the second pier. The difference between front pier and second pier scour will be about 18-24% when the second pier is located about 2.2-2.5 times the pier diameter. The difference then will decrease. Therefore, in order to reduce the scour depth at the second pier the best location of the pier is around 2.2

to 2.5 times the pier size. In this case the first pier is act as sacrificed pier. The strength of horse shoe vortex is mainly decrease by the first pier. When the pier distance is larger than the above mentioned ratio, the scour at second pier increases and it will be the same as the first pier when the distance is long away that each pier act as a single pier.

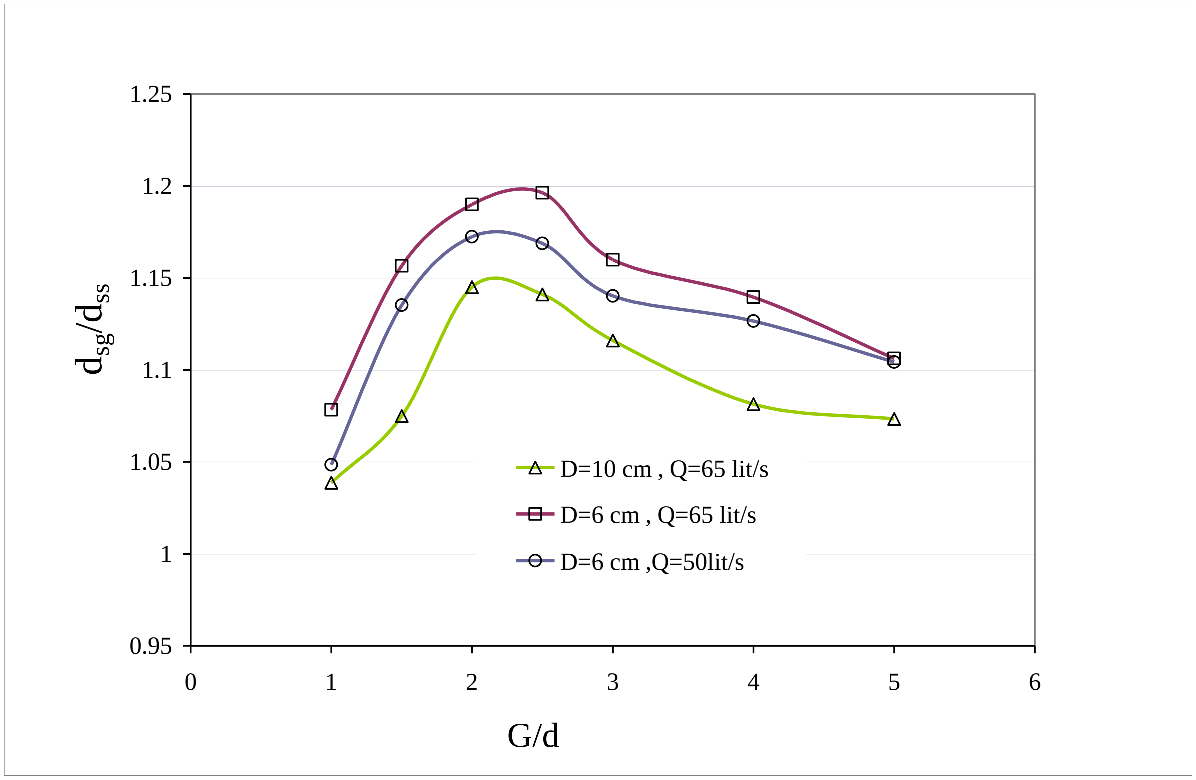


Fig. 4. Variations of d_{SF}/d_{SS} relation to variations of *G/D* (cylindrical group pier and d_{50}=1.31 mm)

To investigate the variation of the scour depth at the middle and the downstream pier compare to each other, Fig. 5 was plotted. This figure illustrates variation of maximum scour depth at middle pier (*Dsm*) and downstream pier (*Dsr*) to the maximum scour depth at the first pier (*Dsf*) for different experimental tests. In each of these experimental tests, flow discharge and diameter of piers are constant. Each of components of Fig. 5 is concern to one experimental test.

Almost for *G/D*>3, depth of local scour in front of middle pier will become more than depth of local scour in front of rear pier.

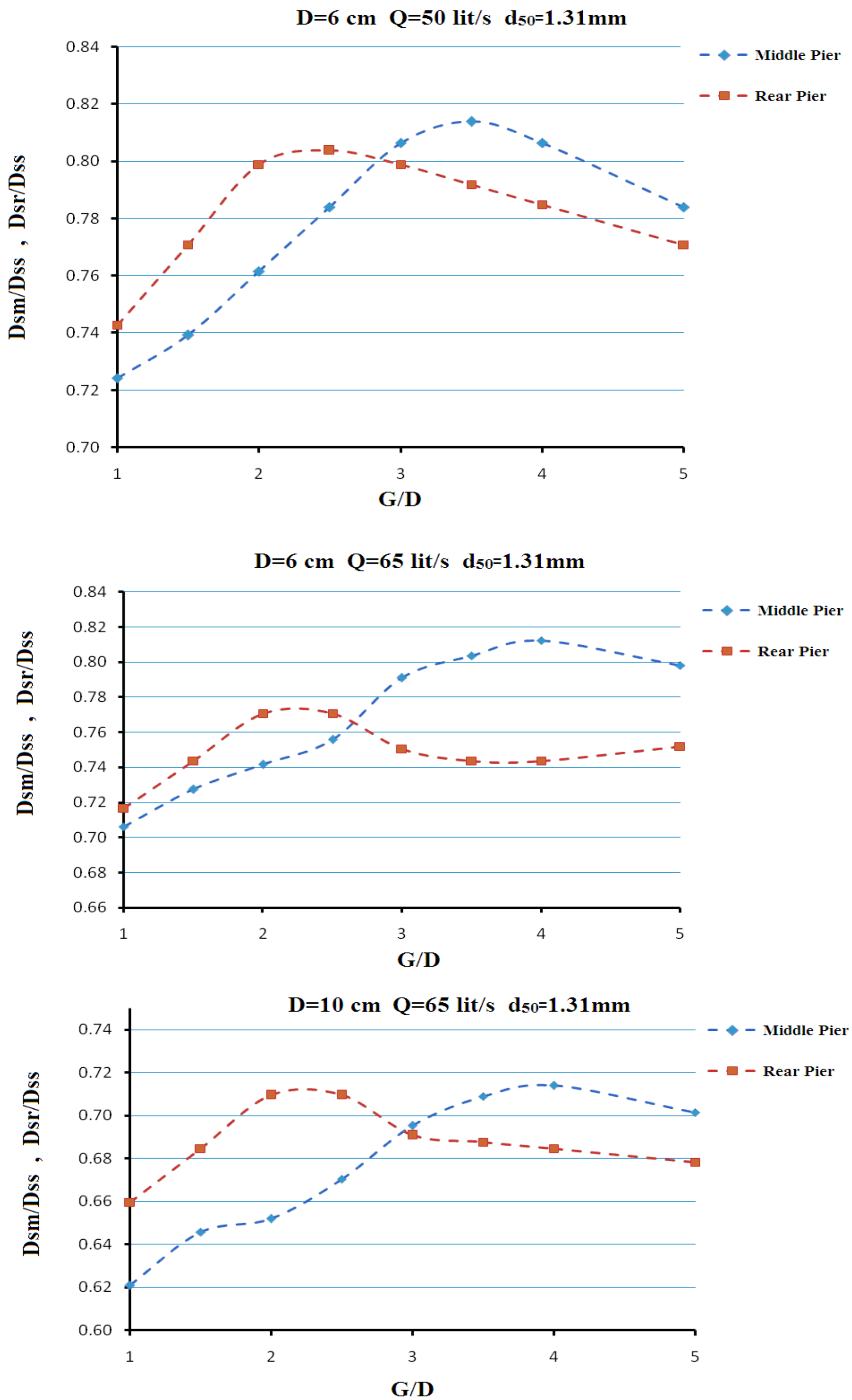


Fig. 5. Comparison between D_{Sm}/D_{Sf} and D_{Sr}/D_{Sf} for different G/Ds and tests

Maximum (equilibrium) depth of local scour for a single pier (d_{SS}) is shown in Table 7.

Table 7. d_{SS} for cylindrical pier and d_{50}=1.31 mm

Q (lit/s)	D= 6 cm	D= 8 cm	D= 10 cm
50	8.06	9.27	10.74
65	9.32	11.31	12.41
80	10.31	12.75	13.97

Maximum depth of local scour in front of front pier (d_{Sf}), middle pier (d_{Sm}) and rear pier (d_{Sr}) is indicated in Table 8 for different group piers.

Table 8. d_{Sf}, d_{Sm} and d_{Sr} for cylindrical group pier, d_{50}=1.31 mm and G=30 cm

Q (lit/s)	D (cm)	d_{Sr} (cm)	d_{Sm} (cm)	d_{Sf} (cm)
	6	7.13	7.31	9.08
50	8	6.39	6.86	11.07
	10	7.49	7.21	13.42
	6	7.91	8.63	10.63
65	8	7.72	8.09	12.94
	10	10.87	10.29	15.33
	6	8.04	8.11	11.65
80	8	8.89	9.47	14.73
	10	11.19	10.52	16.92

New relation for predicting depth of local scour in group piers:

This equation is a modified form of Sheppard et al. [16] equation. In this equation, f_2 of Eq. 15 is converted to below form.

$$f_2\left(\frac{U}{\hat{V}_C}\right)=1-1.105\left(Ln\left(\frac{U}{\hat{V}_C}\right)\right)^2 \tag{17}$$

Where: $\hat{V}_C$ is synthetic critical velocity [18].

$$\hat{V}_C=\sqrt{0.055g\Delta d_{50}}\,f\left(\frac{h}{d_{50}}\right)\approx\sqrt{0.055g\Delta d_{50}}\left[\frac{1}{k}Ln\left(11\frac{y_1}{d_{50}}\right)\right]$$
$$\text{if } \left(\sqrt[3]{\frac{g\Delta d_{50}^3}{\nu^2}}<150\right) \tag{18}$$

Where: $\Delta=(\rho_s-\rho)/\rho$ (ρ_s is mass of unit volume of sediment and ρ is mass of unit volume of water). k is Von Karman constant (equal to 0.4). ν is kinematic viscosity.

Simarro et al. [18] stated that synthetic critical velocity should use for experimental studies instead of critical velocity.

For considering effects of group pier, a parameter adds to Sheppard et al. [16] equation. This parameter (f_4 (G/D)) is an exponential regression relation that is determined by SPSS software. This relation is governing on data corresponding to local scour of cylindrical group pier. This relation is:

$$f_4\left(\frac{G}{D}\right)=\left[1-e^{(1-\frac{G}{D})}\right]+e^{0.725(1-\frac{G}{D})} \quad R^2=0.891 \tag{19}$$

At the end, for calculation of maximum depth of local scour in front of the first pier, below equation is applied:

$$\frac{d_s}{D}=2.5f_1\left(\frac{y_1}{D}\right)f_2\left(\frac{U}{\hat{V}_C}\right)f_3\left(\frac{D}{d_{50}}\right)f_4\left(\frac{G}{D}\right) \tag{20}$$

Comparison between developed equation and other equations are as below:These comparisons are accomplished for different experimental data (data of this research and data of other researchers). These comparisons are stated in below parts.

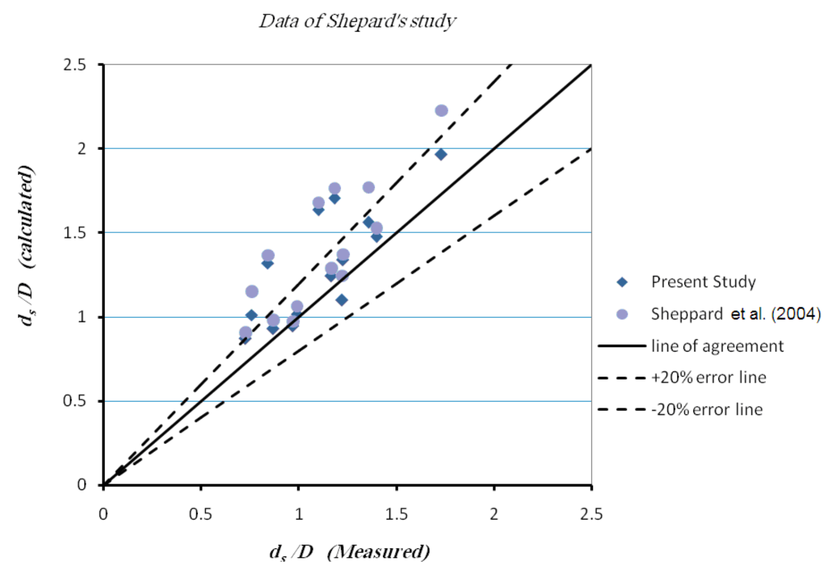
a) Data of Sheppard et al. [34]. Comparison between results of this study (R^2=0.92) and Sheppard et al. [16] equation (R^2=0.89) is illustrated in Fig. 6a.

b) Data of this study for cylindrical piers. For selection the best available equation and modification of it in this research, comparison between results available equations is illustrated in Fig. 6b. In this figure R^2 is 0.91, 0.87, 0.86 and 0.82 for

Richardson and Davis [14], Sheppard et al. [16], Ettema et al. [11] and Melville [9] equations respectively.

c) Data of Sheppard et al. [34]. Comparison between results of this study (R^2=0.92) and other equations is depicted in Fig. 6c. R^2 is 0.91, 0.89, 0.83 and 0.77 for Richardson and Davis [14], Sheppard et al. [16], Ettema et al. [11] and Melville [9] equations respectively.

d) Data of Yanmaz and Altinbilek [8]. Comparison between results of this study (R^2=0.89) and other equations is indicated in Fig. 6d. R^2 is 0.92, 0.87, 0.91 and 0.8 for Richardson and Davis [14], Sheppard et al. [16], Ettema et al. [11] and Melville [9] equations respectively.



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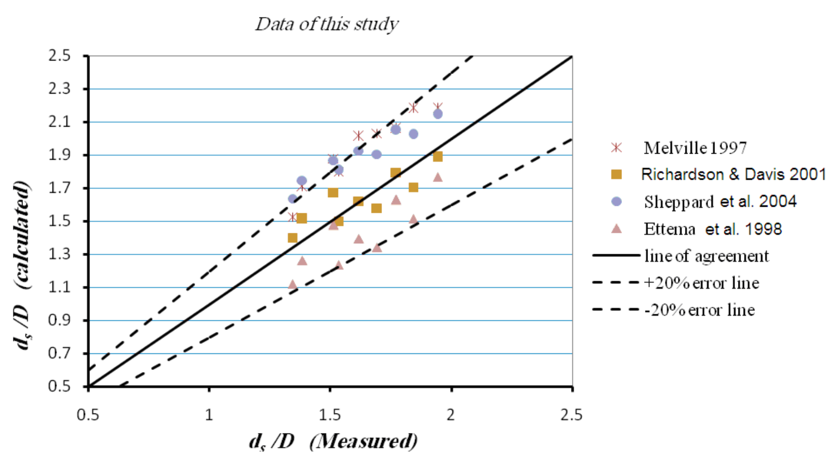

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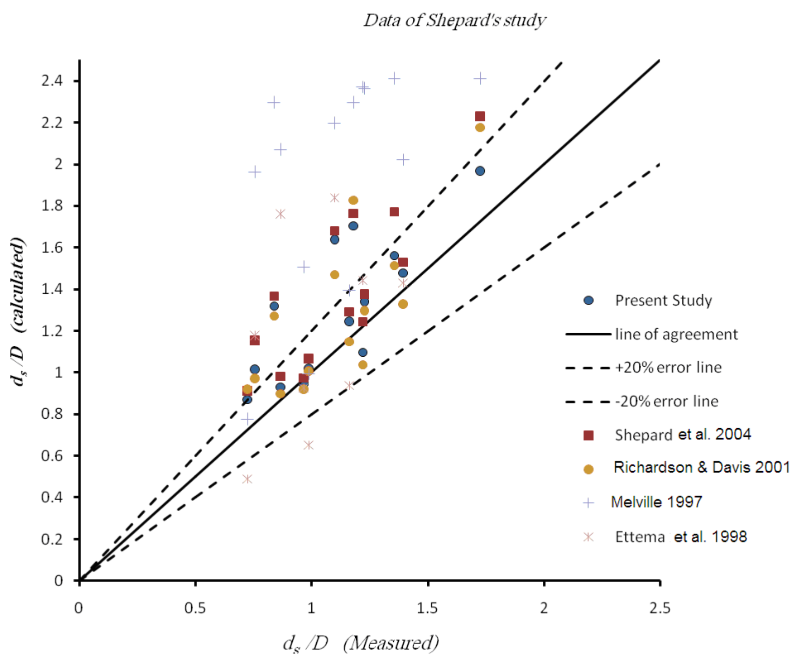

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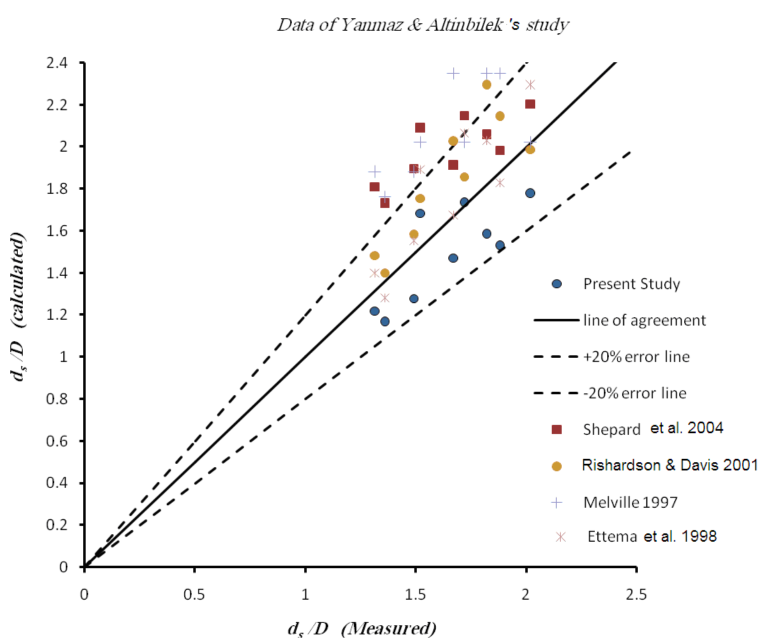

(d)

Fig. 6. a) Comparison between results of this study and Sheppard et al. [16] equation (Data of Sheppard et al. [34]) b) Comparison between results available equations c) Comparison between results of this study and other equations (Data of Sheppard et al. [34]) d) Comparison between results of this study and other equations (Data of Yanmaz and Altinbilek [8])

4. Conclusion

The following conclusions can be highlighted from this study:

1- Tests indicates that 30% to 40% of local scour occurs at 2 minutes from beginning of test and 55% to 65% of local scour occurs at 10 minutes from beginning of test and more than 90% of local scour occurs at 120 to 200 minutes from beginning of test.

2- In the beginning of tests, depth of developed hole by local scour increased very rapid but gradually increasing of surface of hole became faster than increasing of depth of hole.

3- By increasing diameter of pier, size and strength of horse shoe vortexes increased. This subject enlarged depth of developed hole by local scour.

4- By increasing of discharge of clear water, maximum depth of developed hole by local scour increased. For $U/V_C>1$, sediment particles arrived to developed hole by local scour and flow was not clear water flow. In this state depth of local scour was independent of discharge and depth of flow.

5- By increasing of d_{50}, maximum depth of developed hole by local scour reduced. For diameter/ d_{50} >50, depth of local scour was independent of d_{50}. This parameter had slightly effect on maximum depth of local scour.

6- The maximum and minimum depth of developed hole by local scour occurred at *G/D*=2 and *G/D*=1 respectively. For *G/D*>2.2, by increasing *G/D* depth of developed hole by local scour in front of front pier decreased. For great values of *G/D*, difference between depth of developed hole by local scour in front of front pier of group pier and its single pier was negligible.

7- For *G/D*>3, depth of developed hole by local scour in front of middle pier became more than its rear pier. For this *G/D*, effects of sheltering factor decreased depth of developed hole by local scour in front of rear pier.

8- Fig. 6 depicted that results of developed equation at this research were more accurate than results of other equations in calculation of depth of developed hole by local scour. Due the consideration of the effects of group piers and utilized synthetic critical velocity instead of critical velocity in this equation.

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