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Study on the Effect of Installation Depth of Pipe on Scour Depth Under the River Crossing Pipeline in Unsteady Flow

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Abstract
Water and other fluid pipes that are placed on the seabed and rivers bed change the flow pattern around them. These changes increase the shear stress of the bed and the severity of turbulence around the pipe and create a scouring hole under the pipe. The scouring under the pipe may lead to instability, bending, and even breakage of the pipeline. In this study, local scouring around pipelines across the river in unsteady flow conditions was investigated. In the experiments, three pipe diameters (20, 40, and 60 mm) and three installation depths (on the bed, a quarter of the pipe diameter above the bed, and half of the pipe diameter above the bed) were used. It can be understood from the results of this laboratory study that the amount of final scour depth due to the passage of flood hydrograph is always less than the scour depth due to the steady flow corresponding to the peak flow of the hydrograph. For different pipe installation depths, the amount of scouring for the installation depth of one-quarter of the pipe diameter above the bed in both steady and unsteady flow conditions is higher than other installation depths.

Keywords: River, Local scour, Pipe diameter, Pipe installation depth, Unsteady flow.
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Introduction

Pipes are the most common means of transporting water, gas, oil, and other fluids across the river. The passage of pipelines through rivers and canals has the potential to cause serious damage to the river and the surrounding environment. The pipeline crossing the river may be at the top of the bed, on the bed, or under the bed. When pipeline crosses the river and crosses its bed, the hydrodynamic force starts acting on it, as well as the pressure gradient between the upstream and downstream of the pipe, which creates vortices that cause scouring under the pipeline. The scour mechanism under the pipe in a steady flow condition in the study by Sumer (2002) asserts that the development of scour under the pipeline may cause the pipeline to fail, leading to disruptions in operation, reconstruction costs, and environmental damage. On the other hand, since the mechanism of sediment transport and scouring are different in steady and unsteady flows, and that, according to the related literature, there is no document about scouring around pipelines across the riverbed during unsteady flow and floods, it is very important to study the mechanism of the occurrence of this phenomenon and to predict the maximum scour depth around the pipelines in unsteady flows.

Maza (1987) showed that the scour created under the pipe is a function of the Froude number and the relative distance between the pipe and the initial bed. Sumer and Fredsoe (1991) reviewed previous research and showed that pipe roughness has little effects on the scour depth. Chiew

(1991) also studied the depth of scouring in shallow flows. Ghodsian et al. (2000) performed several experiments on scouring around pipelines and measured the scour depth under the pipe and at different distances of the pipe from the sediment bed with uniform sediments. They obtained scour depth and length for the average particle size of 1.8 and 0.72 mm. They stated that the final scour depth increases with the increasing flow depth in shallow water, and for flow depths greater than five times the pipe diameter, flow depth will not affect scour depth. Dey and Singh (2007) performed several tests to calculate the maximum scouring depth under pipelines in clear water conditions under steady flow despite upward leakage on five pipe diameters and four different sizes of inpatient materials, and found that scour depths below pipelines with upward leakage were generally less than without leakage. Wu and Chiew (2013) investigated the scour hole and the flow field around a pipeline under a steady flow. The flow field in these experiments was measured by an acoustic Doppler velocimeter. The results of this study showed that the presence of vortices which created upstream and downstream of the pipe as a consequence of pressure difference, causes the formation of a force for the movement and displacement of sediments. Also, the flow from under the pipe into the scour hole causes it to expand further. Azamathulla et al. (2014) investigated the effects of the angle of the collision of flow lines with the pipeline on the scour below it, and examined four angles of 30, 45, 60, and 90 degrees. The results of this study showed that changing the angle of the pipeline relative to the flow causes the scour hole to move towards the channel wall. Also, changing the angle of the pipeline had little effects on the maximum scour depth. Zhao et al. (2015) performed laboratory and numerical study on scouring under two consecutive pipelines with different distances from each other. It was observed that, in moving bed conditions, the depth of the scour hole under the upstream pipe is slightly greater than the scour hole under the single pipe, while the depth of the scour hole under the downstream pipe is much greater than the scour depth compared to the single pipe. According to the observations of this study, the depth of the scour hole in the state of clear water is greater than the state of the mobile bed. The reason for this is the occurrence of scouring and the lack of replacement with upstream sediments in clear water. Also, the closer the two pipelines are, the lower the maximum scouring depth.

During the recent years, studies on unsteady flows have accelerated due to their greater similarity to the natural conditions of rivers. There are various discussions concerning the unsteady flows which are addressed by various researchers. In the remainder of this section the studies that have been done in the field of unsteady flows and the issues which they had to deal with are reviewed. One of the oldest studies on the transport of bed sediment during unsteady flow under equilibrium and nonequilibrium sediment conditions is related to Griffiths and Sutherland (1977). In their research, they used hydrographs with a base time of 30 to 120 minutes. They reported that there was little difference between the bed sediment transfer rate under the corresponding unsteady and steady flow conditions. The steady-state corresponding to a flood hydrograph is the same as the step model of the corresponding hydrograph so that the flow conditions include the depth and the flow rate over a specified period (step continuity time) in each step. Of course, this conclusion was challenged later by researchers such as Graf and Suszka (1985) and Lee et al. (2004). They attributed this conclusion to the very high base of production hydrographs in Griffiths and Sutherland's (1977) study. Graf and Suszka (1985) performed laboratory studies on the amount of sediment transported under steady and unsteady conditions. They concluded that the sediment rate carried under unsteady flow conditions is always higher than the corresponding steady flow. In a study by Kothyari et al. (1992), they found a relationship which helped find the maximum scour depth around bridge piers in unsteady flow. This identified relationship was obtained using the data of previous research. The unsteady flow in the experiments of this study included a normal triangular hydrograph with a peak flow of 80 liters per second and a duration of 30 minutes. They divided the hydrograph into separate time steps to find the maximum scouring depth in the unsteady flow and they took each of these steps as a steady stream. Then, using the obtained equations for continuous flow, they obtained the maximum scouring depth in each part. Finally, the scour depths obtained for each of the steady flows were summed together to estimate the maximum scour depth in the unsteady flow. Chang and Yen (2004) performed experiments to find scour depth around bridge piers in a steady and unsteady flow. Their experiments were performed using uniform and non-uniform bed materials under clear water conditions. They proposed a method for calculating the temporal variation of scour depth in unsteady flow. They conducted the experiments by stepping on the hydrograph. To produce an unsteady flow, the flow was regulated through the inlet and end gates. Their experiments were performed to examine two cases: the effect of peak hydrograph flow at a constant time and the effect of peak flow time at a constant peak flow. Oliveto and Heger (2005) investigated the scouring around bridge piers under unsteady flow and non-uniform sediments. They examined two types of floods with different flow conditions. The first type of floods with the same peak flow and different peak times and floods with the same peak time and different peak flows were used. The second type of floods used in this study had two identical and different peak discharges. Like other researchers, they achieved the maximum scouring depth around the bridge piers in the unsteady flow by stepping the hydrograph and the relationship identified by themselves (Oliveto and Heger, 2002). Lu et al. (2011) carried out a case study on scouring around bridge piers in Taiwan's rivers. The collected field data, which were the sum of local and general scour depth, were used to evaluate the model presented by them. Comparison of collected data and calculated data with steady flow relationships based on peak flood discharge showed that these relationships overestimate the scour depth. Tabarestani and Zarrati (2017) conducted analytical laboratory studies on the estimation of time development of local scour around a rectangular bridge pier in an unsteady flow condition. Experiments performed under steady and unsteady flows were compared. The results of this study showed that although the shape of the scour depth change curve is a function of the time of occurrence of the hydrograph peak, it has little effects on the final scour depth. Also, in this study, using the results of previous research, a relationship was identified in order to estimate scour parameters.

Due to the complexity of simulating unsteady flows in the laboratory, no study has so far been performed on scouring under a pipeline across a river under unsteady flow conditions. Currently, with the development of laboratory facilities, limited studies have been carried out in the field of scouring around bridge piers and breakwaters under unsteady flows. The purpose of the present study is to investigate the effects of insertion depth and diameter of pipe crossing the waterway on the scouring pattern under the pipe, maximum scouring depth, and the trend of scouring time changes under steady and unsteady flow conditions.

Materials and methods

All experiments of the present study were performed in the hydraulic laboratory of the Faculty of Water and Environmental Engineering of the Shahid Chamran University of Ahvaz in a rectangular flume 10 m long, 74 cm wide, and 60 cm high. The walls of the flume are made of glass and have inlet and outlet tanks and a part to calm the flow. Figure (1) shows a view of the flume used, the pipe installation location, the test zone, and the flow calming system.

According to the test program, an electric pump device of 23-200 Pumpiran type was prepared, which was coupled with an electromotor device of 18.5 KW that provided a flow rate of up to 120 liters per second. To investigate the scouring phenomenon around the pipe crossing the waterway, in the middle of the flume, in an area, 1.5 m long and 15 cm thick, uniform sediments with Medium size (d50) 0.7 mm, relative density (Sg) 2.65, and standard deviation (σg) 1.4 were poured. Figure (2) shows the sediment granulation curve used in the experiments. Two metal mesh grid plates were used to eliminate the turbulence and excess energy of the input stream to the laboratory flume. In the laboratory flume, the velocity profile was taken by a JFE ALEC 3D electromagnetic velocimeter to find a suitable location for the test zone and pipe installation location. The first section was taken at a distance of 3 meters from the inlet and the next sections were taken at a distance of half a meter from the previous section. As can be seen from the profiles drawn in Figure (3), the adaptation of the velocity profiles taken from 3 m onwards indicates the development of the boundary layer and the uniformity of the flow in the flume. In this study, pipes with a diameter of 20, 40, and 60 mm at bed installation depths (e/D= 0.), a quarter of the pipe diameter above the bed (e/D= 0.25), and half of the pipe diameter above the bed (e/D= 0.5) were used. The pipes were made of PVC and were installed perpendicular to the flow across the flume. The experiments of the present study were performed under steady and unsteady flow conditions. The total number of experiments was 18. Due to the unstructured state, it was observed that, in unsteady flow experiments, the movement of sediment particles starts at discharges of more than 33 liters per second. For this reason, the peak discharge of the hydrograph in the conditions of unsteady flow was selected as 33 liters per second. Therefore, a normal hydrograph with a peak discharge of 33 liters per second and a base time of 120 minutes was used. steady flow tests were performed for the corresponding peak discharge of the hydrograph (33 liters per second) and for the time equivalent to the base time of the hydrograph of unsteady flow (120 minutes). In steady and unsteady experiments, clear water conditions prevailed. Figure (4) shows the hydrograph used in the present study. Figure (5) shows the installation depth of the pipe and its related parameters, schematically.

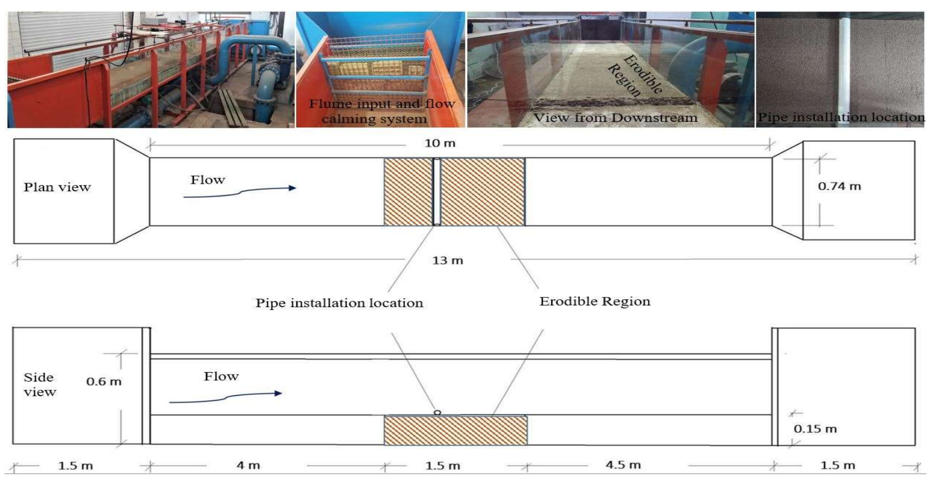

Fig. 1- View of the laboratory flume used in this research

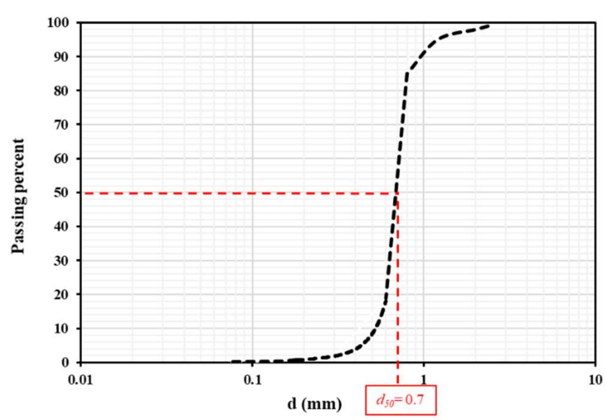

Fig. 2- Sediment granulation curve

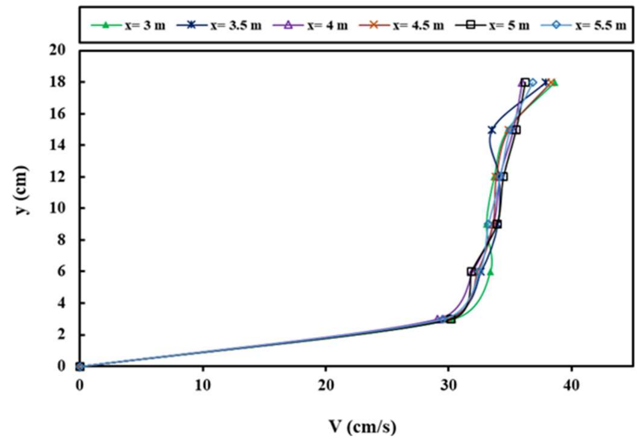

Fig. 3- Depth velocity distribution

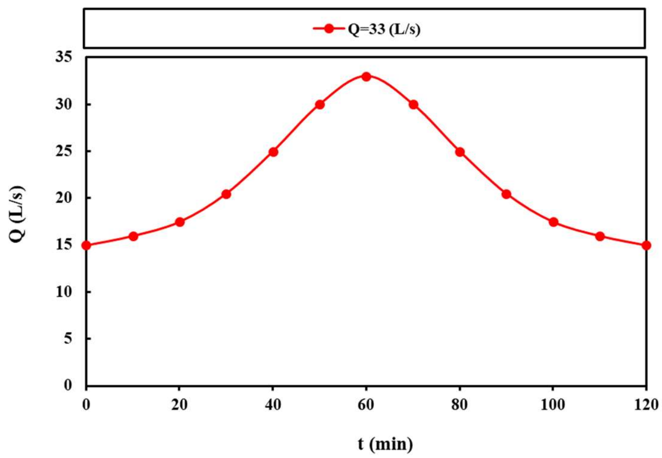

Fig. 4- Normal hydrograph.

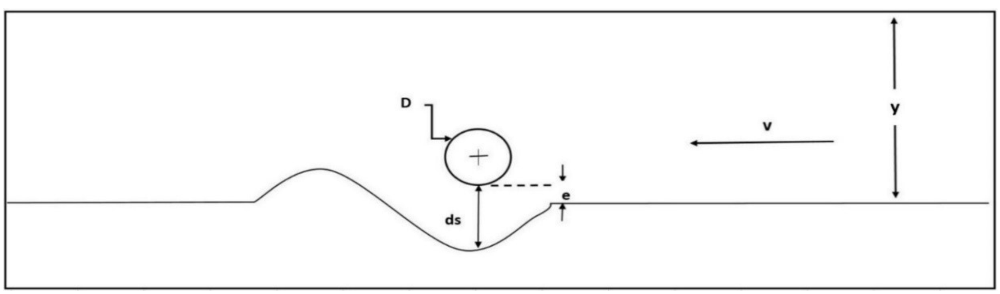

Fig. 5- Schematic figure of pipe installation depth and scour depth under a pipeline

In order to prevent bed formation, at the beginning of each experiment the downstream flume gate is closed and the flume is filled with water. The inlet flow to the flume was then cut off. The pipe is then installed and stabilized perpendicular to the flow and at the desired level. After installing the pipe, by turning the pump on, the flow in the flume is established. Shortly after the start of the test, scouring under the pipeline began. At the end of each test, the pump was switched off and the canal drains were opened. Also, the maximum depth of scouring under the tube was collected using a laser meter.

Table (1) presents the specifications of the experiments of the current research, where Q is the flow rate, y is the flow depth, V / Vc is the velocity to critical velocity ratio, Fr is the Froude number, D is the pipe diameter and e/D is the pipe installation depth.

Results and discussion

Temporal changes in scour depth under steady flow conditions

Scouring always occurs around structures located in the course of the river. The amount and extent of scouring depend on the flow conditions and geometric conditions of the structure. Despite significant advances in the study of scouring, limited research has been conducted to investigate scouring around the pipeline crossing the riverbed. The results of this figure indicate that for each installation depth, the scouring process has its own behavior. For example, at an installation depth of e/D= 0.5, for all three diameters tested, the slope of scouring changes was very high in the initial moments and a significant part of scour depth development occurs at the very beginning of the experiment. A comparison of each of the installation depths shows that at the installation depth e/D= 0.5, the scouring depth in the first 5 minutes is close to 83% of the final scouring depth. Figure (6) shows a view of the test process and the start of scouring under the pipeline and a view of the formation of the scouring hole and the flow pattern around the pipe. Figure (7) shows the temporal variations of scour depth for different diameters.

Table 1- Specification of Experiments

Row	Test	D(mm)	e/D	Q (l/s)	Q_{peak} (l/s)	Fr	V/V_c	y(cm)
1	Steady	20	0	33	-	0.26	0.95	14
2	Steady	40	0	33	-	026	0.95	14
3	Steady	60	0	33	-	0.26	0.95	14
4	Steady	20	0.25	33	-	0.26	0.95	14
5	Steady	40	0.25	33	-	0.26	0.95	14
6	Steady	60	0.25	33	-	0.26	0.95	14
7	Steady	20	0.5	33	-	0.26	0.95	14
8	Steady	40	0.5	33	-	0.26	0.95	14
9	Steady	60	0.5	33	-	0.26	0.95	14
10	Unsteady	20	0	-	33	0.26	0.95	14
11	Unsteady	40	0	-	33	0.26	095	14
12	Unsteady	60	0	-	33	0.26	0.95	14
13	Unsteady	20	0.25	-	33	0.26	0.95	14
14	Unsteady	40	0.25	-	33	0.26	0.95	14
15	Unsteady	60	0.25	-	33	0.26	0.95	14
16	Unsteady	20	0.5	-	33	0.26	0.95	14
17	Unsteady	40	0.5	-	33	0.26	0.95	14
18	Unsteady	60	0.5	-	33	0.26	0.95	14

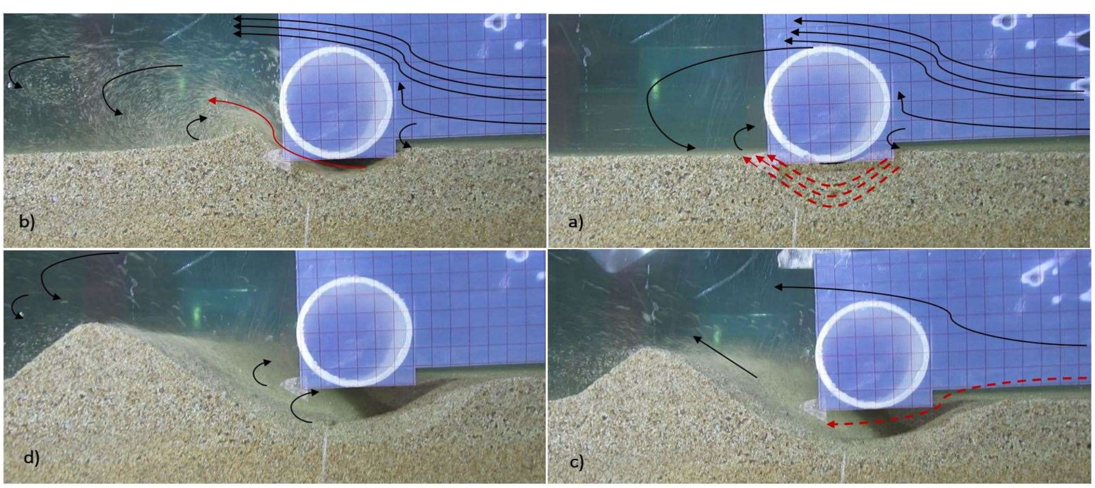

Fig. 6- Schematic figure of flow pattern and scour under the pipeline.

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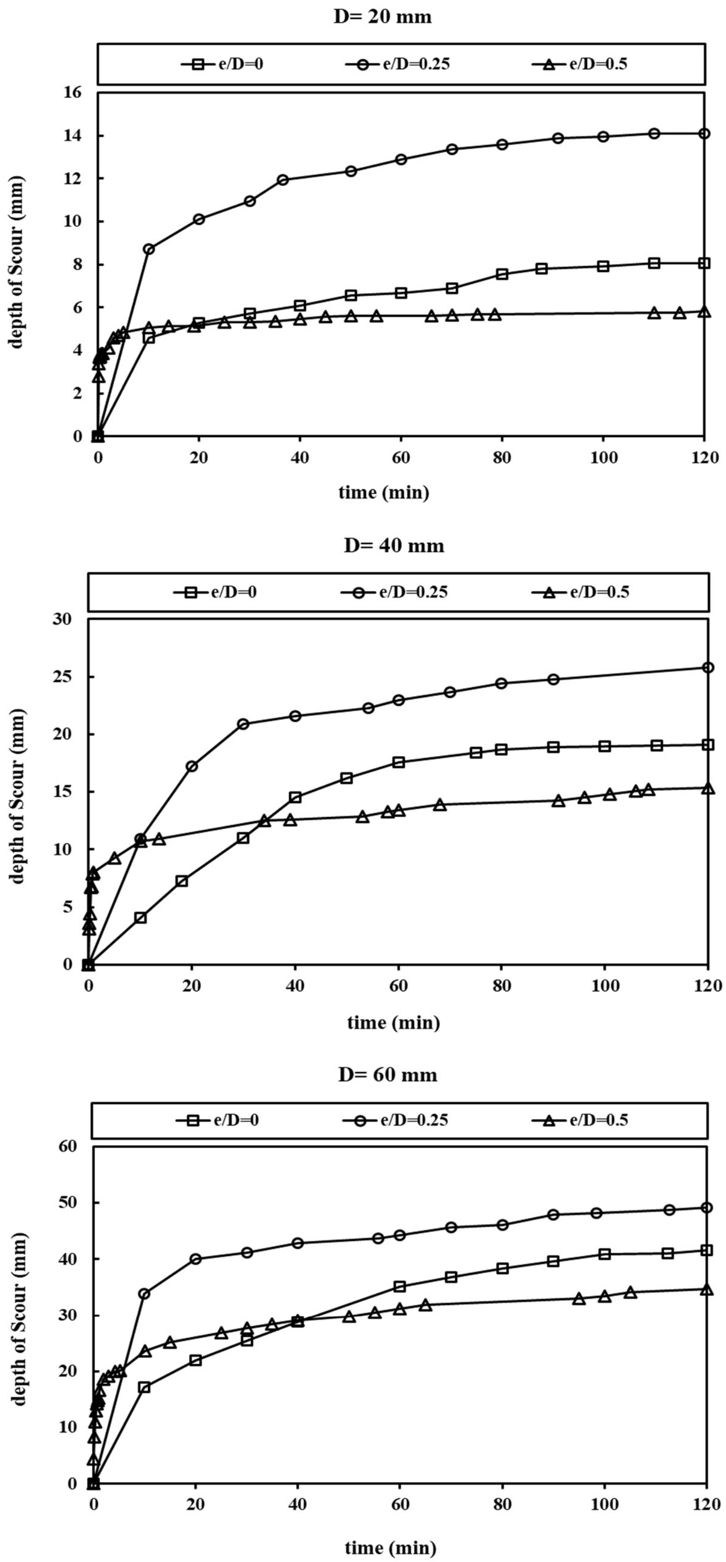

Fig. 7- Temporal scour development for different diameters and installation depth of pipe in steady flow condition

At the installation depth e/D= 0.25, the scouring depth occurs at about 90% of the final scouring depth in the first hour of the experiments. At this installation depth, unlike the installation depth e/D= 0.5, the temporal progression curve of the scouring depth has a gentle slope. In general, the results of Figure (7) show that the highest scouring depth has occurred at the installation depth e/D= 0.25. After that, the installation depth e/D= 0, has the highest scouring rate. So that, in the first hour of each experiment, at this installation depth, about 85% of the maximum scouring depth occurs. Installation depth of e/D= 0.5 has the lowest scouring depth compared to other installation depths but the temporal development of this implantation depth is faster than the rest of the depths. The results related to the effect of installation depth for different scenarios are compared in equation (1). The ds index displays values for an installation depth of pipe (e/D). As shown in Figure (7), as the diameter of the pipe increases, the effect of the insertion depth decreases.

$$d_{s_{e/D=}0.25} > d_{s_{e/D=}0.} > d_{s_{e/D=}0.5} \qquad (1)$$

Flow pattern and scouring under the pipe in unsteady conditions

According to the observations made in steady and unsteady experiments, these flows can be compared with each other and the differences and similarities of each flow and scouring pattern around the pipeline in both steady and unsteady conditions can be studied and compared. The flood hydrograph used in the present study has a duration of 120 minutes. Since this duration is relatively long, changes in flow characteristics such as flow rate and depth do not change significantly during the test. For example, in conditions of unsteady flow in an instant, there is little difference between the flow depth at the beginning and end of the channel, in such a way that most changes in unsteady flow are observed over time. However, in case of short periods, the flow pattern formed by the unsteady flow is very similar to the steady flow pattern. In the case of unsteady flow, the approaching flow is converted into a backward and downward flow and after passing under the pipe, they interfere with the upstream approaching flow. The mechanism of flow and scouring under the pipe of the rising vortices that cause the sediment bed to be removed is very similar to the steady flow. The main difference between the flow pattern in steady and unsteady flow conditions is the changes in the amount of stress applied to the sediment bed during each of the unsteady flow tests. The hydrograph used in unsteady flow consists of two branches, namely ascending and descending. In the ascending branch of the hydrograph, the intensity of the stresses on the sedimentary bed gradually increases and then decreases after the peak flow of the hydrograph. As a result, compared to steady flow conditions in which the amount of stress applied to the bed is consistently the same, in the unsteady flow, the amount of vortex power that causes scouring is strengthened and weakened in proportion to changes in flow rate and intensity. In the descending branch of the hydrograph, the flow intensity decreases so much that in some cases it is not able to move the washed sediments in the scouring hole. Examination of the scour under the pipe in an unsteady flow showed that changes in scour depth stop shortly after the passage of the hydrograph peak. This can be attributed to the reduction of flow rate and flow intensity in the descending branch of the flood hydrograph and the weakening of the main and rising vortices.

Investigating the effect of passing a hydrograph with a peak flow corresponding to a steady flow rate

This part of the present study is presented to investigate the scour under the pipe under the effect of a flood hydrograph with a peak flow which corresponds to the steady flow rate. The maximum ratio of shear velocity to critical shear velocity of the hydrograph is 0.95 and according to the experiments, no scouring around the pipe was observed at flow rates of less than 15 liters per second, therefore, the base discharge of the hydrograph was 15 liters per second. To find the peak flow of the hydrograph in the unstructured state, it was observed that the movement of sediment particles begins at flows higher than 33 liters per second; for this reason, the peak discharge of the hydrograph

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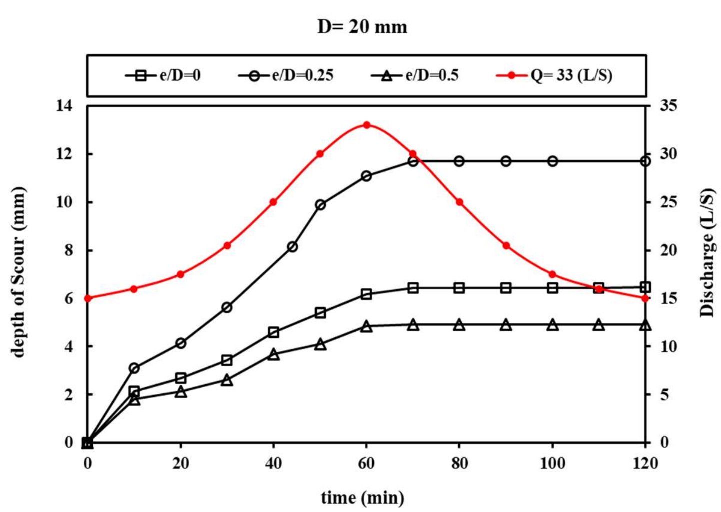
at unsteady flow conditions was 33 liters per second. The results of the unsteady flow tests are shown in Figure (8). Figure (8) illustrates the temporal variations of scour depth for different diameters in unsteady flow conditions. The vertical axis of this shape represents the scouring depth in millimeters and the horizontal axis shows the time in minutes. The results of this figure indicate that the process of scour depth changes is such that, sometime after the passage of the hydrograph peak, the development of the scour hole stops, this is due to the reduction of applied stresses on the sediment bed in the descending branch of the hydrograph after passing the peak of the hydrograph. As shown in Figure (8), like steady flow conditions, the highest values related to scour depth were observed at the depth of installation e/D= 0.25. The maximum final scour depth at the installation depth e/D= 0.25 for diameters of 20 and 40 mm is 48 and 35% higher than other installation depths. Given that the equilibrium depth and scour stop time in each of the pipes for different installation depths were investigated, it was observed that in experiments related to 20 mm diameter and installation depth e/D= 0.5, approximately after the passage of the hydrograph peak, the development of scouring dimensions, including scouring depth, stopped, and continued with the same amount until the end of the experiment. At the depth of installation e/D= 0.25, in the initial 20% of the time from the continuation of the descending branch of the hydrograph, scouring stops. A diameter of 40 mm also follows a trend close to a diameter of 20 mm and for almost all cases, scour development stops after a small amount of hydrograph peak passes. At a diameter of 60 mm, it can be seen that for experiments related to the installation depth e/D= 0.5, the scouring starts to stop at about 30% of the duration of the descending branch of the hydrograph. According to the diagrams shown in Figure (8), it can be seen that each of the pipe installation depths during the test demonstrates its specific behavior over time in order to extend the maximum amount of scouring. The final results of the maximum scour depth are shown comparatively for different install depths in equation (2). As shown in Figure (8), The compression of the streamlines under the pipe installed at a depth of e/D= 0.25 that, this compression leads to increased shear stress on the bed and the consequence is more scouring at this installation depth than at other installation depths.

$$d_{s_{e/D=}0.25} > d_{s_{e/D=}0.} > d_{s_{e/D=}0.5} \quad (2)$$



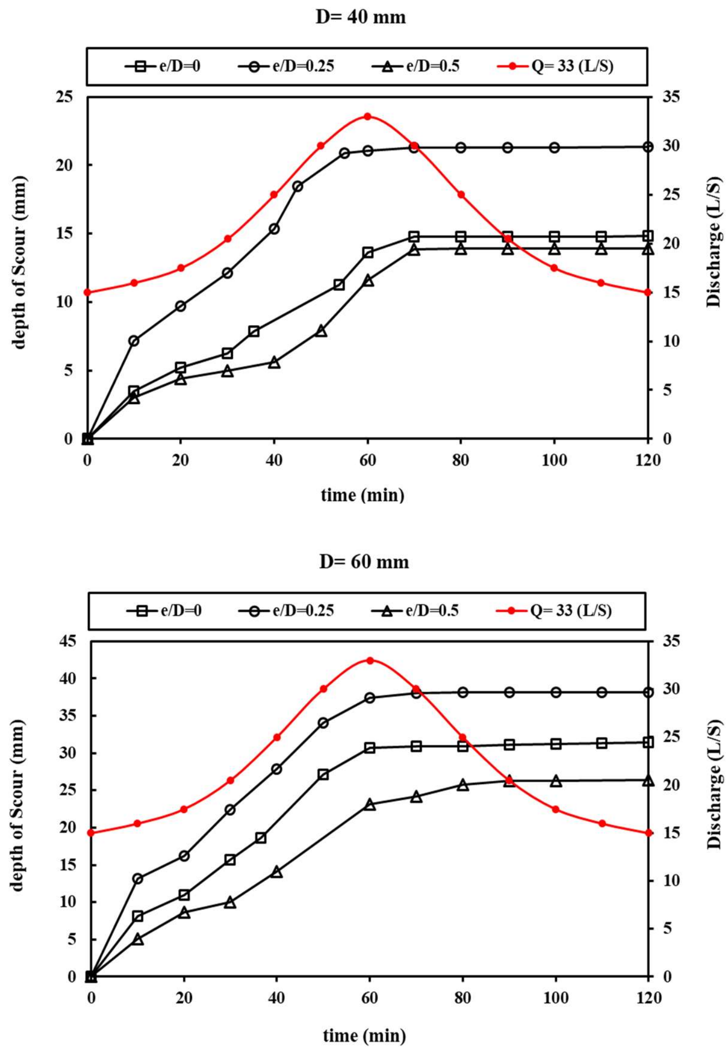


Fig.8- Temporal scour development for different diameters and installation depth of pipe in unsteady flow condition

As a result, the design of structures crossing the river should be done according to the maximum scour depth due to the maximum flow rate, equilibrium time, and sediment granulation characteristics. To reach the maximum equilibrium scouring depth, the unsteady flow must continue with the maximum flow rate and the equilibrium time. But, given that floods are a kind of transient flow over time, their persistence time is less than when the sediments reach equilibrium. Therefore, according to the results obtained in the present study and according to the studies conducted by Tabarestani and Zarrati (2017) and Link et al. (2017), in most cases, the scour around the designed structures crossing the river after the unsteady flow of flood is less than the equilibrium scour.

Under steady conditions and for a time similar to the duration of the hydrographs under study, the depth of scouring increases by the end of the test time. However, in unsteady conditions, after passing the hydrograph peak due to the reduction of shear stresses on the bed, the scour depth remains constant. The main difference in the scouring mechanism in the conditions of steady flow compared to unsteady flow is the difference in the intensity of stresses on the sediment bed, thus, in the steady flow, the intensity of applied stresses is constant and uniform, but in the conditions of unsteady flow, the intensity of these stresses on the sedimentary bed increases with an increase of the hydrograph flow. When the hydrograph reaches the peak discharge, these stresses reach their maximum values and decrease after the passage of the hydrograph peak discharge. Therefore, the strength of the vortices and the intensity of the flow are equally attenuated in the reducing branch of the hydrograph. Therefore, in the descending branch of the hydrograph, the sediment has no power to transfer into the scour hole and the sediment accumulates in the hole and finally, the scour depth becomes less than steady, under unsteady conditions.

Conclusion

The general results of this laboratory research explains that the conditions and the amount of scouring in the conditions of the steady flow are different from the conditions of the unsteady flow. For installation depth e / D = 0.25 in the unsteady flow, scour depth decreased by 13% compared to the steady flow and this value decreased by 35% and 23% for e/D= 0 and e/D= 0.5, respectively. After analyzing the results, at the depth of installation e/D= 0.25, the highest rate of final scouring, as well as the high rate of scouring progress over time, was observed. This process indicates that at this depth, the streamlines approaching the pipe and passing under the pipe have a high shrinkage and that the stress on the bed has increased compared to other conditions and as a result, has caused more erosion. This condition was observed in both steady and unsteady conditions, which is less in unsteady conditions for the reasons mentioned above. Most of the scouring under the pipe in steady conditions occurred within the first few minutes, and due to unsteady flow conditions, the scour depth reached its final level shortly after the hydrograph peak and after that, there was no change in the depth of scouring. The final scour depth at unsteady flow conditions was lower than the steady flow conditions in all cases. Scour ratio for unsteady to steady flow is between 35 and 60%.

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