Article

Evaluation and Prediction of the Scour Depth of Bridge Foundations with HEC-RAS Numerical Model and Empirical Equations (Case Study: Bridge of Simineh Rood Miandoab, Iran)

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Abstract. Today, scouring is one of the significant issues in river and coastal engineering. Each year, many bridges around the world destroy due to neglecting hydraulic elements. In the present study, scour depth around the piers of the Simineh Rood Bridge in Miandouab, Iran were investigated using empirical relationships and the HEC-RAS numerical model, and the results are compared with each other. Firstly, a hydraulic software model was created from the river where the bridge was located using field data. Then, by entering the scouring data of bridge piers for discharges with a return period of 5 to 1000 years, changes in flow discharge were investigated for scouring around the middle and lateral sides of the bridge. Results of the empirical equations showed that some of equations are not sensitive to increases in flow discharge, and for each return period, the results are near each other. Also, numerical model results showed that with an increase in discharge, scouring increases in the bridge's middle and lateral piers. In all discharges, the first and the seventh pier had the lowest and highest scour depth, respectively. Also, the left and right abutments are heavily influenced by increasing discharge. In discharges with a return period of 1000 years, the scour depth was 11.19 and 6.32 m. The Frohlich method is not as sensitive as the CSU method to an increase in discharge when calculating scour depth. Finally, the results of the numerical model were compared with experimental empirical equations.

Keywords: Scour depth, Simineh Rood Bridge, empirical equations, HEC-RAS.

1. Introduction

Bridge abutment scour in river engineering, as well as scouring around sea piles, is an important topic. Many reports indicate that the scouring phenomenon as a factor in bridge destruction. Therefore, estimating bridge scour depth is crucial. Scouring is caused due to the erosion power of running water, transporting material in the vicinity of flow-damping structures. The obstacle in the water stream changes the flow pattern in its vicinity [1]. In this case, changing the distribution of the flow velocity and the emergence of complex patterns of turbulence in the flow field (vortices) will be a factor in exacerbating bed erosion [2]. The main system of the vortex is generated from the collision of flow with the front part of the pier and its downward deviation (Fig. 1). Investigations show that horseshoe vortices and afflux play a major role in creating a scour hole around the bridge pier [3].
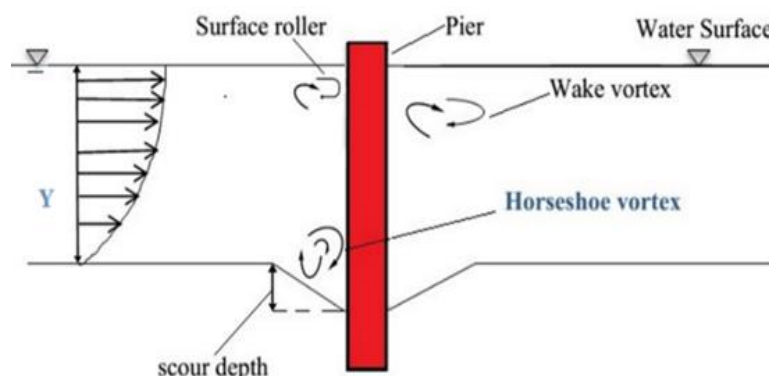

Fig. 1. Local scouring and how to create horseshoe jerks around the bridge.

There are numerous experimental and numerical studies on bridge pier scouring and control sedimentation of basin [4, 5, 6]. Mohammad et al. (2005) [7] have estimated the CSU equations, Melville and Sutherland (1998) [8], Larsen and Toch (1959) [9] using experimental field data. The results showed that the CSU and Larsen and Toch equations have better accuracy than the other two equations. Torabi et al. (2015) experimentally investigated on the effect of erosion on the stability of the breakwaters' foundation. They modified and proposed a new method [10]. A numerical study of bridge scouring using one-dimensional and two-dimensional HEC-RAS and FLO2DH models was carried out by Yu and Yu (2008), in which both models presented acceptable results for scour depth estimation [11].

Ismail (2009) used a one-dimensional model to investigate scouring of Aswan Bridge pier for 270 and 350 cubic meters per day. The results showed that local scour in the piers was 13.66 and 14.47, respectively. He also studied the scouring of the El-Minia bridge piers for a discharge of 181 cubic meters per day with a 30-degree degradation angle. The results of this study showed that local scouring in all piers was 1.81 and 5.34 m [12]. Dahe and Kharode (2015) evaluated the scour depth of bridge piers using the HEC-RAS model. The results showed that the maximum scour depth is related to the square-shaped bridge pier, whereas the lowest value is related to the pier of a sharp-nosed pier. They also proved that the scour depth of the cylindrical base, the rounded nose, and the circular bridge pier group are the same [13].

Kardan et al. (2017) evaluated the bed shear stress distribution, several numbers of numerical simulations are conducted to predict the bed shear stress distribution around a Circular Cylinder pier using FLUENT [14]. Ghasemi et al. (2018) evaluated the Nanostructures to control local scour of bridge piers for steady and unsteady flows [15]. Zahabi et al. (2018) investigated the effects of different geometries of reservoirs on the flow pattern and deposition of sediments by using the finite volume method [16]. Moussa (2018) evaluated local scouring of the Aswan and L-Minia bridge piers with different geometric shapes using the HEC-RAS and CCHE2D models. It showed that the scouring of the bridge piers in Aswan is higher due to larger dimensions of the foundation (compared with the L-minia). It was also shown that the CCHE2D model offers more accurate results than the HEC-RAS model, especially in non-uniform transverse sections [17]. Sadeghfam et al. (2019) and Daneshfaraz et al. (2019) studied scouring of supercritical flow jets in upstream and downstream of screens and estimated scouring dimensions using artificial intelligence to combine multiple models [18,19].

To design an economical and reliable bridge foundation, the maximum depth of scour around the piers is essential. For this purpose, costly and time-intensive laboratory studies have been conducted. Although these methods are capable of answering the problem, it is not possible to apply it to all issues, due to calculation challenges and the lack of expert knowledge. In this research, according to existing limitations, an attempt was made to assess the empirical equations of the calculation of the scour depth at the site of the bridge foundations using the HEC-RAS model, a multi-purpose model, to solve the problem and compare results.

2. Methods and Material

2.1. HEC-RAS Software

The HEC-RAS software was created and developed by the US Army Engineering office and has been used by engineers around the world for many years. In 2006, the ability to simulate and transfer sediment was added. The HEC-RAS model is capable of fully modeling grids, channels, and branches or individual rivers. The capabilities of this software can be used to analyze steady and unsteady flows, sub-critical and supercritical flows, culvert design, and simulation of sediment transport, etc. (Brunner, 2010) [20] The software also calculates the scour depth of the bridge piers constructed on rivers using hydraulic flow data, shape and geometric characteristics of the bridge pier, as well as the material and shape of the substrate around the river. The default software model for estimating the local scour depth around the bridge piers is the CSU model defined as follows (Richardson and Davis, 1995) [21].

$$y_s = 2\, k_1\, k_2\, k_3\, k_4\, a^{0.65} y_1\, Fr_1^{0.43} \tag{1}$$

In this case, y_s is the maximum scour depth, a width or diameter of the pier, y_1, the flow depth in the pier upstream, k_1, the pier shape coefficient, k_2 the coefficient of the impact angle, k_3 the bed condition coefficient, k_4 the bed's coefficient of reinforcement by the sediment particles, Fr_1, the landing number The flow is at the base upstream. Another model used in the HEC-RAS software to estimate the scour depth of the bridge base is the Frohlich equation, which is defined as follows for calculating the screw base of the bridge:

$$y_s = 0.32\, \phi\, (a')^{0.62}\, y_1^{0.47}\, Fr_1^{0.22}\, D_{50}^{-0.09} + a \tag{2}$$

Also, the Frohlich equation is used to estimate the maximum scour depth at the side of the bridge (abutments), when the abutment length is less than 25 in depth, defined as follows:

$$y_s = 2.27\, k_1\, k_2\, (L')^{0.43}\, y_a^{0.57}\, Fr_1^{0.61} \tag{3}$$

In these relations, y_s, the maximum scour depth, ϕ, the coefficient of correction for the nose, y_1, the depth of flow at the upstream of the pier, D_{50} is the average diameter of the bed particles, a is the width or diameter of the base, a' width of the lateral bridge pier that imported to flow path, k_1, coefficient of bearing shape, k_2 influence coefficient of the impact angle of the flow with the abutment, L' length of the bearing, y_a average flow depth (hydrological depth), Fr_1, the Froude number in the floodplain.

2.2. Governing Flow Equations

Water surface profiles are computed from one cross section to the next by solving the energy equation to a body of water enclosed by two cross sections at locations 1 and 2 as:

$$h_2 + Z_2 + \frac{\alpha_2 V_2^3}{2g} = h_1 + Z_1 + \frac{\alpha_1 V_1^3}{2g} + h_e \tag{4}$$

where h is the water depth at cross sections, Z is the elevation of the river inverts, V is the average velocity, α is velocity weighting coefficient, g is the gravitational acceleration and h_e is the energy head loss. The energy head loss between two cross sections is comprised of friction losses and contraction and expansion losses as:

$$h_e = LS_f + C\left(\frac{a_2 V_2^{\,2}}{2g} - \frac{a_1 V_1^{\,2}}{2g}\right) \tag{5}$$

where L is the distance between cross section 1 and 2 along the direction of the flow, S_f is the friction slope between two cross sections, which is computed using Mannings equation as:

$$Q = \frac{1}{n} A R^{2/3} S_f^{1/2} \tag{6}$$

where Q is the flow discharge, n is the Manning roughness coefficient, A is the flow area, R is the hydraulic radius (area/wetted perimeter) and C is expansion or contraction loss coefficient. The program assumes that a contraction is occurring whenever the velocity head downstream is greater than the velocity head upstream. When the change in river cross section is small and the flow is subcritical, the contraction and expansion coefficient are equal to 0.1 and 0.3, respectively. In more abrupt change, such as these occurring at bridges, the used values are 0.3 and 0.5, respectively [22]. The HEC-RAS sediment model solves the sediment continuity equation, known as the Exner equation, as:

$$(1-\lambda_p)B\,\frac{\partial z}{\partial t} = -\frac{\partial Q_s}{\partial x} \tag{7}$$

where B is the channel width, z is the channel elevation, λ_p is the active layer porosity, t is the time, x is the distance along the flow direction and Q_s is the sediment discharge. Equation (7) states that the change of sediment volume in a control volume (aggradation or degradation) is equal to the difference between the inflowing an outflowing sediment loads. The sediment continuity equation is solved by computed a sediment transport capacity through the control volume associated with each cross section. If sediment transport capacity is greater than supply there is a sediment deficit and bed erosion. If supply exceeds transport capacity there is a sediment surplus and material deposition [20].

2.3. The Studied Region

The Simineh Bridge is located on the Simineh Rood River in Miandoab, an Iranian city. The Simineh Rood River basin, along with the river branches (Fig. 2), is the only bridge between Miandoab and the surrounding villages, as well as the bridge between the center of Urmia province and the cities of Mahabad, Naghadeh, Piranshahr, Sardasht and other southern cities of the Urmia province. This bridge has 7 rows of quadruple piers. In Fig. 3 and Table 1, an exponential representation of the bridge position and the bridge is presented.
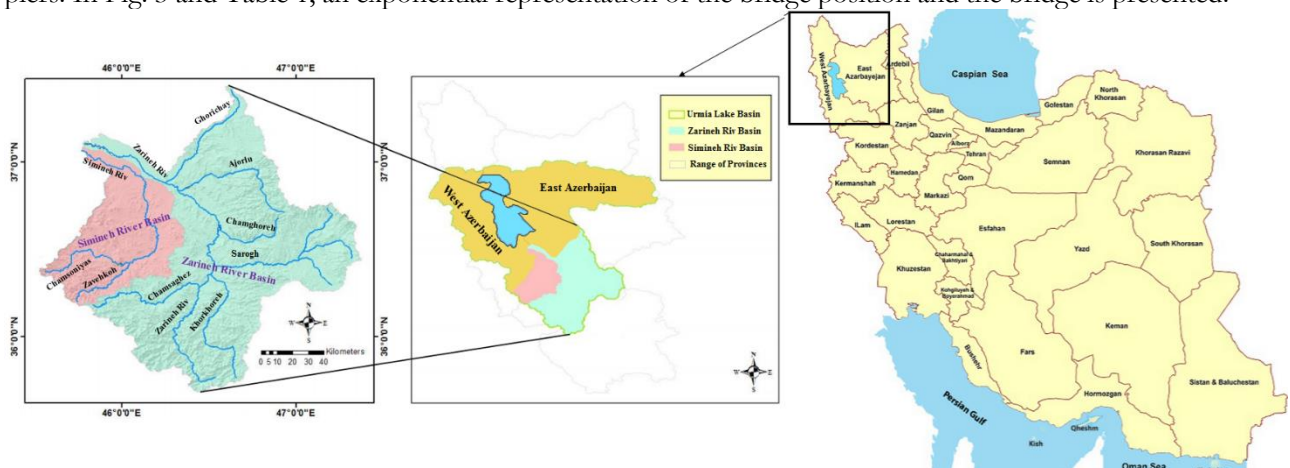

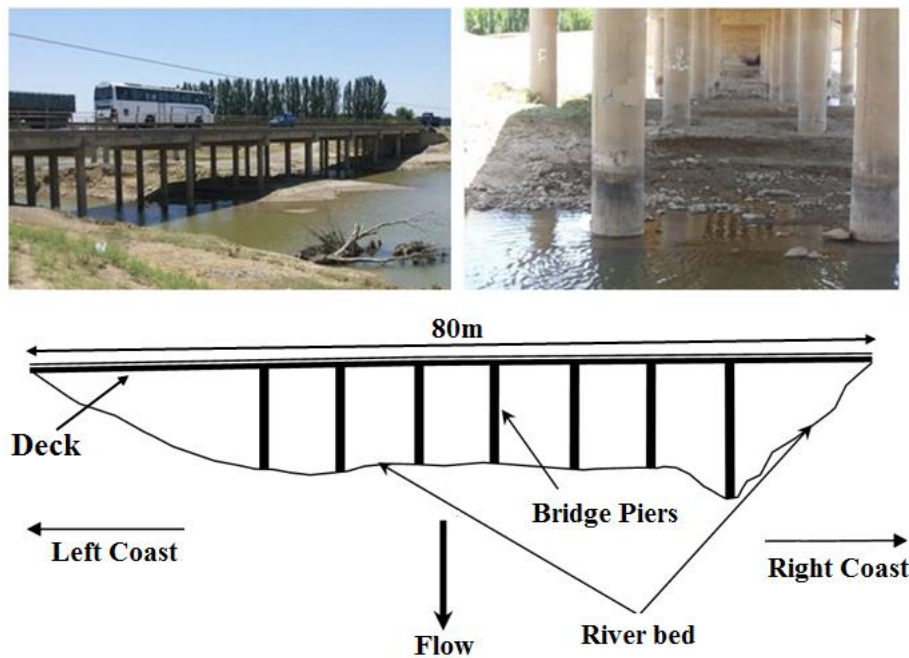Fig. 2.   River basin of Simineh Rood with its branches.

Fig. 3.   View the position of the bridge.

Table 1.   Geometry parameters of Simineh Rood bridge.

Number of Pier	28	Width of bridge	9 m
Kind of pier	cylindrical	Height of bridge	5.5 m
Section of Pier	circular	Height of piers	4 m
Length of Bridge	80 m	Diameter of pier	0.8 m

Upstream of the Simineh Rood Bridge, a second bridge is being constructed. In this study, sedimentation data of the under-construction bridge is being used. Accordingly, the mean diameter of sediment aggregate (D_{50}) where the bridge is located is measured at 7.75 mm and for sediments with D_{95} it is 12 millimeters. The river is in the plain area of the studied bridge, with a maximum 1% gradient. The roughness coefficient for this range was selected between 0.055 and 0.025. Using statistics from the Miandouab Bridge Hydrometer Station, which is located 20 meters below the bridge, discharge was calculated across different return periods. For this purpose, regular moments were used with three-parameter statistical distributions and SMSDA software. In this study, the best statistical distribution was a three-parameter normal log distribution, and the results of discharge simulations with different return periods are presented in Table 2.

Table 2.   Discharges with different return periods for the Simineh Rood River.

Return period(year)	2	5	10	15	20	25	35	40	50	75
Discharge(m³/s)	50.1	75.4	100.2	125.1	149.8	174.7	200	225.2	248.4	271.6
Return period(year)	85	100	150	200	250	300	350	400	500	1000
Discharge(m³/s)	363	426.5	497	544.4	587.2	599.2	742.4	874.1	1050	1125

2.4. Hydraulic Model

The first step in HEC-RAS software simulation to calculate the maximum scour depth of bridge piers is to select a hydraulic model from the range of the river where the bridge is located. This model should consist of several cross-sections at the bottom and several cross-sections upstream from the bridge. For this purpose, the site's aerial view was added to the software using Google Earth software. Then, lateral sections of 15.09 meters were created at the bottom and upstream from the bridge and were interpolated between them. Fig. 4 shows the cross-sections of the bridge piers of the Simineh Rood River.

Fig. 4.   Cross sections to determine the depth of scouring of bridge piers.

2.5. Bridge Geometry

The geometry of the Simineh Rood Bridge includes the deck of the bridge, the abutments, and the 7 rows of quadruple piers. In Fig. 5 and 6, sections and profiles along with the location of the bridge in the HEC-RAC software are shown to determine the scouring depth of the Simineh Rood Bridge piers.
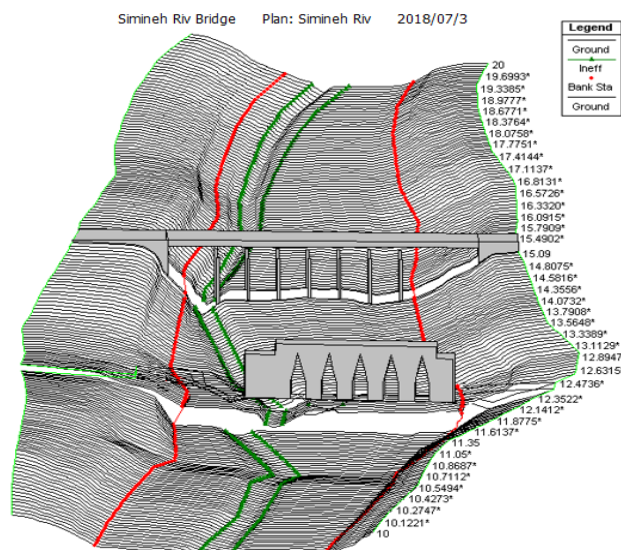

Fig. 5.   Cross sections and bridges to determine the depth of scouring of the Bridge bridge.
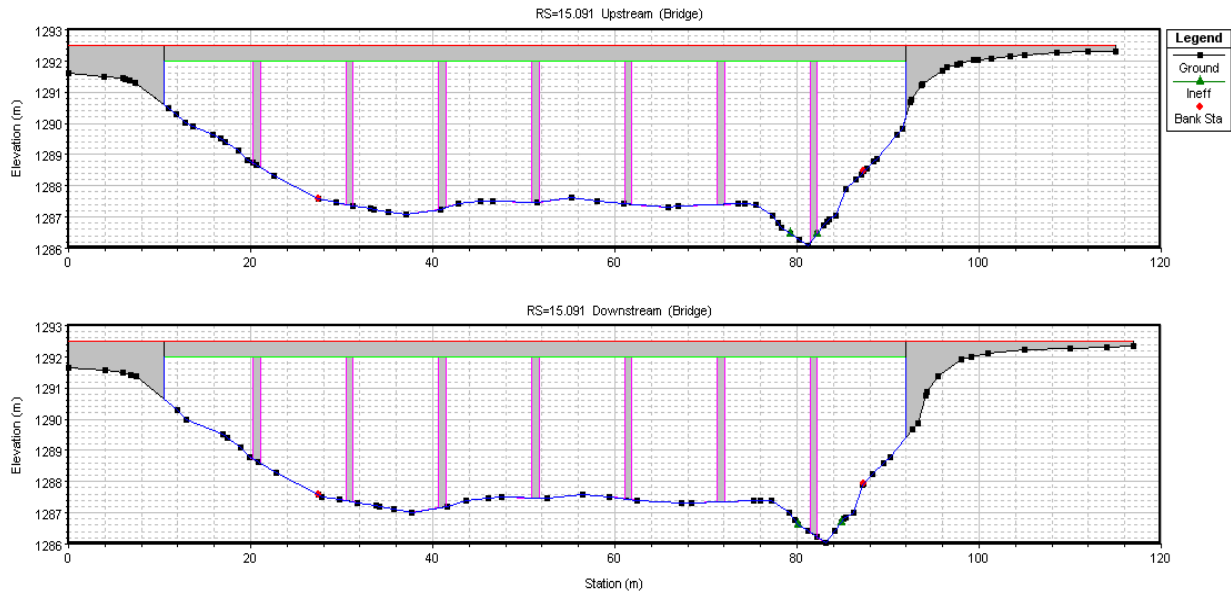
Fig. 6.   Transverse sectional profiles in the bridge of the Simineh Rood bridge.

The existence of boundary conditions is essential for carrying out computations. If the flow analysis is subcritical only the downstream boundary conditions will be considered; but if the flow analysis is supercritical, then defining the upstream boundary conditions is necessary. In calculating the mixed flow regime, boundary conditions must be introduced upstream and downstream for the model [23, 24, 25]. In the present study, due to the very low slope of the Simineh Rood River in the area of research and establishment of a subcritical regime, the lower boundary condition is the pier on the Stage-Discharge curve of the Miandoab bridge hydrometric station. In Fig. 7, the diagram of the Stage-Discharge is shown for this hydrometric station.
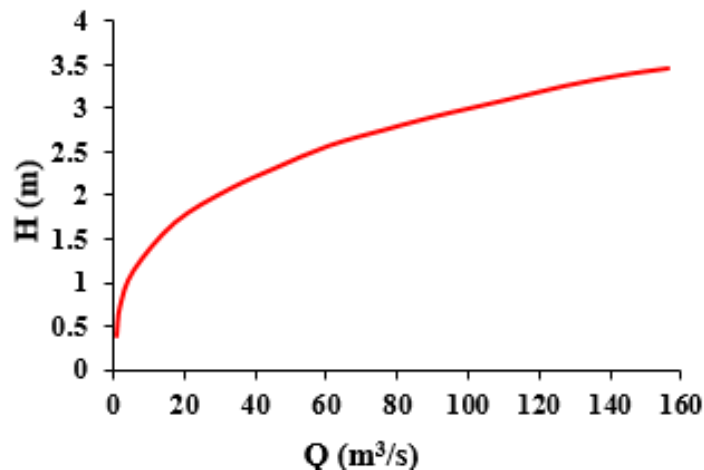

Fig. 7.   Flow diagram of stage-discharge at Milandoab hydrometric station.

2.6. The Empirical Equations for Determining Scour Depth

Simons and Şentürk (1992) look at the width and the length of the bridge, the depth of flow upstream, the flow velocity at the bottom, the size of the material forming the bed, the angle of the flow of the pier, the shape of the pier, the up and down in the river bed due to the accumulation of sediments, the accumulation of ice packs in the bridge span, and flow silt as issues that affect scour depth around bridge piers [26]. According to the above, a set of experimental equations has been presented by researchers to determine scour depth in accordance with Table 3.

Table 3.   Empirical equations provided by researchers to determine the scour depth.

Jain (1981) [27]	$\dfrac{d_s}{b}=1.41(\dfrac{y}{b})^{0.3}(\dfrac{V_c}{\sqrt{gy}})^{0.25}$	-	-
Johnson (1992) [28]	$\dfrac{d_s}{y}=2.02\left(\dfrac{b}{y}\right)^{0.98}Fr^{0.21}\sigma^{-0.24}$	-	$\sigma=\dfrac{D_{84}}{D_{50}}$ $Fr=V\,/(gy)^{0.5}$
Colorado State University (CSU)(1993)	$\dfrac{d_s}{y}=2K_1K_2K_3F_r^{\,0.43}(\dfrac{b}{y})^{0.65}$	-	-
Shen et al. (1969) [29]	$d_s=0.0002\,\mathrm{Re}^{0.619}$	-	-
Breusers et al. (1977) [30]	$\dfrac{d_s}{b}=f\,K_1K_2(2tgh(\dfrac{y}{b}))$	$\dfrac{V}{V_c}\ge 0.5$ $0.5<\dfrac{V}{V_c}\le 1$ $\dfrac{V}{V_c}>1$	$f=0$ $f=2(V/V_c)-1$ $f=1$
Baker (1980) [31]	$\dfrac{d_s}{b}=(N_s-1.3)tgh(\dfrac{y}{b})$	-	$N_s=\dfrac{V}{\sqrt{(G_s-1).gD_{50}}}$
Melville and Sutherland (1988) [8]	$\dfrac{d_s}{b}=K_dK_lK_yK_aK_s$	-	-
Laursen and Toch (1956) [9]	$\dfrac{d_S}{b}=1.35(\dfrac{y}{b})^{0.3}$	-	-
Sheppard and Miller (2006) [32]	$\dfrac{d_s}{b^*}=2.5f_1f_2\left\{1-1.75\left[\ln\left(\dfrac{V}{V_C}\right)\right]^2\right\}$ $\dfrac{d_s}{b^*}=f_1\left[2.2\left(\dfrac{\dfrac{V}{V_C}-1}{\dfrac{V_{lp}}{V_C}-1}\right)+2.5f_3\left(\dfrac{\dfrac{V_{lp}}{V_C}-\dfrac{V}{V_C}}{\dfrac{V_{lp}}{V_C}-1}\right)\right]$ $\dfrac{d_s}{b^*}=2.2f_1$	$0.47<\dfrac{V}{V_c}<1$ $1<\dfrac{V}{V_C}<\dfrac{V_{lp}}{V_C}$ $\dfrac{V}{V_C}>\dfrac{V_{lp}}{V_C}$	$f_1=\tanh\left[\left(\dfrac{y_1}{b^*}\right)^{0.4}\right]$ $f_3=\left[\dfrac{\left(\dfrac{b^*}{D_{50}}\right)}{0.4\left(\dfrac{b^*}{D_{50}}\right)^{1.2}+10.6\left(\dfrac{b^*}{D_{50}}\right)^{-0.13}}\right]$ $V_{lp1}=0.8\sqrt{g\,y}$ $b^*=K_sb$
Lee and Sturm (2009) [33]	$\dfrac{d_s}{b}=5\log(\dfrac{b}{D_{50}})-4$ $\dfrac{d_s}{b}=\dfrac{1.8}{(0.02b/D_{50}-0.2)+1}+1.3$	$6\le b/D_{50}\le 25$ $25<b/D_{50}$	-
Neil (1964) [34]	$\dfrac{d_s}{D}=1.35\left(\dfrac{b}{D}\right)^{0.70}$	-	-

where d_s is scouring depth, b is pier width, y is flow depth directly upstream of the pier, V is velocity, V_c is critical velocity, V_{lp} is live-bed peak scour velocity (velocity where the bed planes out), g is acceleration of gravity, D is circular pile diameter, D_{50} is the median sediment grain diameter, D_{84} is the sediment size for which 84% of the bed material is finer, σ is standard deviation of the sediment particle size distribution, Fr is Froude number, Re is Reynold's number, K_1 is correction factor for pier nose shape, K_2 correction factor for angle of attack flow; K_3 correction factor for bed condition, K_l flow-intensity factor, K_d sediment size factor, K_y flow-depth factor, K_a pier-alignment factor and K_s pier-shape factor.

3. Results and Discussion

3.1. Determination of the Depth of Scouring Using Empirical Equations

Johnson (1992), Melville and Satherland (1988) [8] and Sheppard and Miller (2006) equations

Figure 8 shows the scouring rate of the Simineh Rood bridge piers using the Johnson (1992), Melville and Satherland (1988) and Sheppard and Miller (2006) equations.
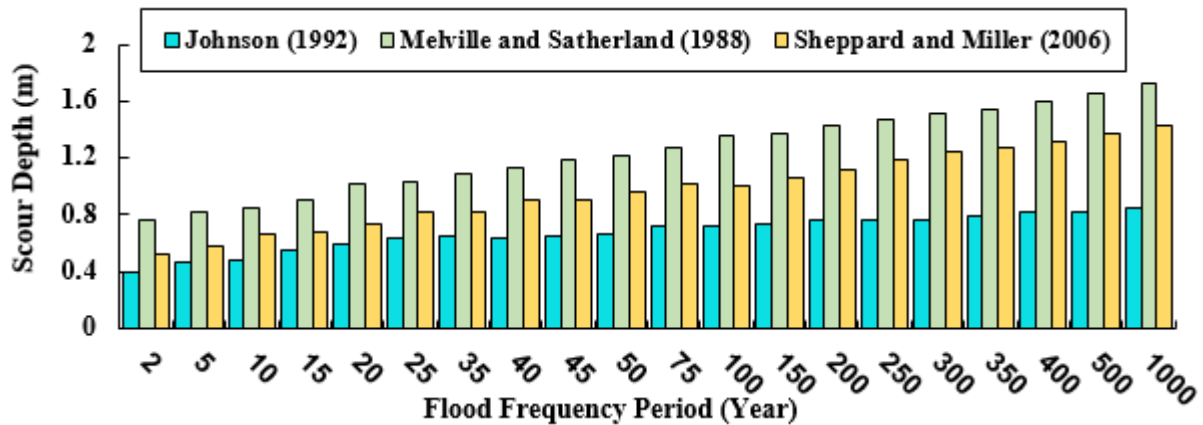

Fig. 8. The depth of Simineh Rood pier scouring using Johnson (1992), Melville and Satherland (1988) and Sheppard and Miller (2006)

According to Fig. 8, the scour depth increases, with an increasing return period. The highest growth is related to the empirical equation provided by Melville and Satherland (1988) and the lowest belongs to Johnson (1992). The differences between Melville and Satherland (1988) with Sheppard and Miller (2006) and Johnson (1992) equations in the results is considering consists of the most effective parameters on scouring depth. The nature of the flow, sediment and bed, the shape, width and tip of the bridge, the angle of attack flow with the bridge, the time and erosion equilibrium are among effective parameters that are well-considered in these relations.

Lee and Sturm (2009) and Neil (1964) equations

Figure 9 shows the scouring rate of the Simineh Rood Bridge using Lee and Sturm (2009) and Neil (1964) equations.
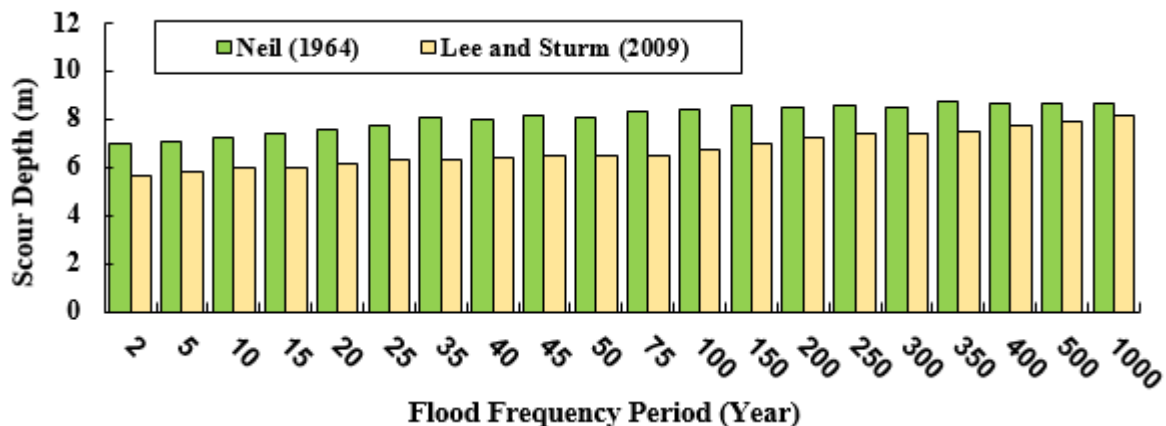

Fig. 9. Scour depth of the bridge structures of the Simineh Rood using the equations between Lee and Sturm (2009) and Neil (1964)

Neil (1964) and Lee and Sturm (2009) did not show a sensitivity of the scour depth calculation to an increase in discharge, and scour depth was determined by increasing the flow rate constant. In this regard, the difference between the scour depth in flood frequency period from 5 to 1000 years in Neil's equations is 123 percent, whereas the difference between Lee and Sturm (2009) is 178 percent.

Colorado State University (1993) and Shen et al. (1969) equations

Figure 10 shows the scouring rate of the Simineh Rood bridge piers using the relationships from Colorado State University (1993) and Shen et al. (1969).
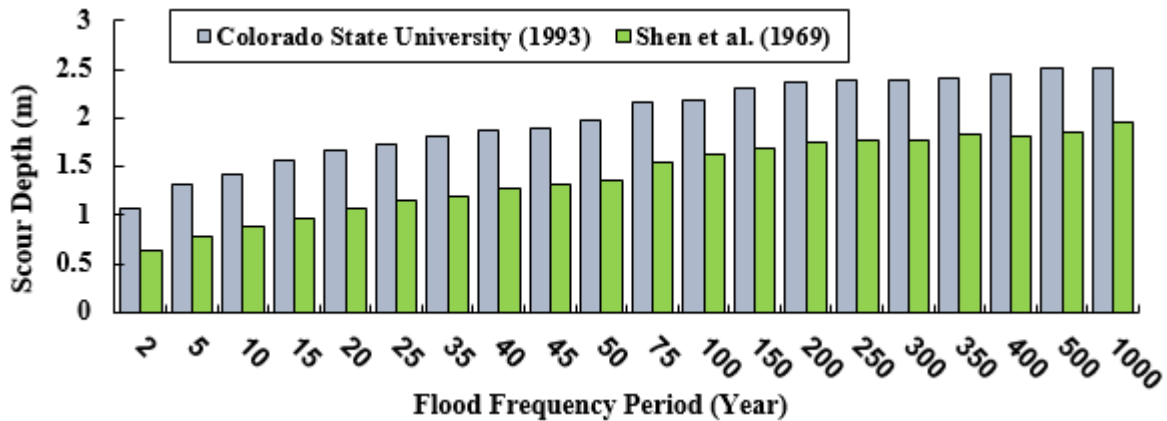

Fig. 10. The depth of Simineh Rood bridge piers scour using the equations from Colorado State University (1993) and Shen et al. (1969)

It is observed that the scour depth for the equation from Colorado State University (CSU) (1993) is greater than that of Shen et al. (1969). The reason for this is interference between the conditions of the bed and the shape of the bridge pier in calculating bridge pier scour depth, as well as considering the Fr number and the flow condition, and entering the average diameter of the sediment and bed sediment conditions. In contrast, Shen et al. (1969) describes the scouring depth as a function of the Reynolds number.

Laursen and Toch (1959) and Jain (1981) equations

Figure 11 shows the scour depth of the Simineh Rood bridge piers using the equations between Laursen and Toch (1959) and Jain (1981).
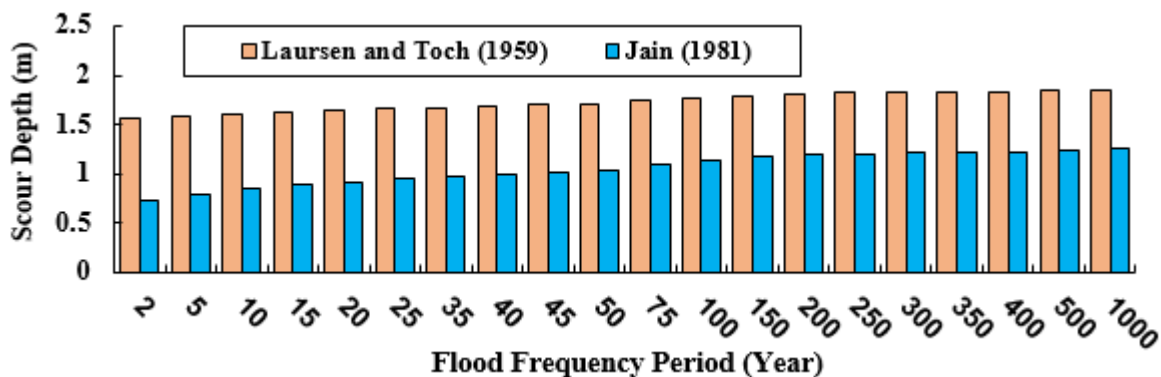

Fig. 11. Scour depth of the Simineh Rood bridge piers using the equations between Larsen and Toch (1959) and Jain (1981).

The scour depth differs more across different return periods for the Larsen and Toch (1959) equation due to the application of bridge pier width parameters and flow depth. The Jain (1981) equation also has connotations for critical velocity in addition to the above-mentioned parameters.

Breusers et al. (1977) and Baker (1980) equations

Figure 12 shows the scour depth at the Simineh Rood bridge piers using the equations between Breusers et al. (1977) and Baker (1980).
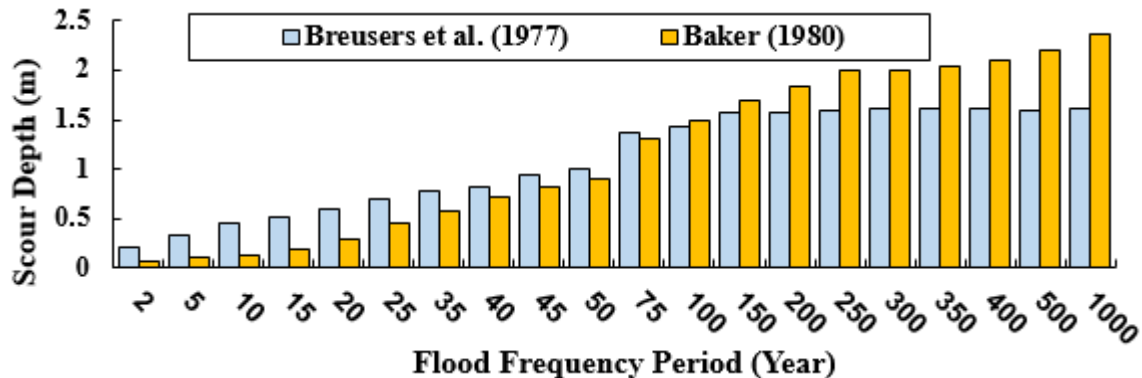

Fig. 12. Scour depth at Simineh Rood bridge piers using the equations between Breusers et al. (1988) and Baker (1980)

The results obtained from these equations were almost the same. For both, the scour depth increased for different return periods. In addition to the flow profile, the scour depth depends on sediment characteristics and bridge pier specifications.

3.2. Determination of Scour Depth Using HEC-RAS Software

Flow Profiles in the studied periods

Figure 13 shows the flow profiles for return periods of 100, 200, 500, and 1000 years. In these periods, due to rising discharges, a bed type that is suitable for each period was selected. For a return period of 100 and 200 years, a live-bed with small pellets of 2 to 10 mm in diameter was selected, whereas for a return period of 500 and 1000 years old, the beds chosen were between 10 to 20 mm and more than 30 mm, respectively, and scour depth calculations were done based on the type of bedding. In these periods, scouring starts from the right abutment, and as discharge increases, the amount increases substantially. The current profiles during the 500-year return period show that, except for a small part of the river, other areas are entirely underwater. For a return period of 1000-years, the discharge increased dramatically and crossed the deck of both Simineh Rood bridges. In the two return periods (500 and 1000 years), the bridge abutments on both the right and left abutments are heavily influenced by flow discharge.

Fig. 13. The flow profiles in the studied range.
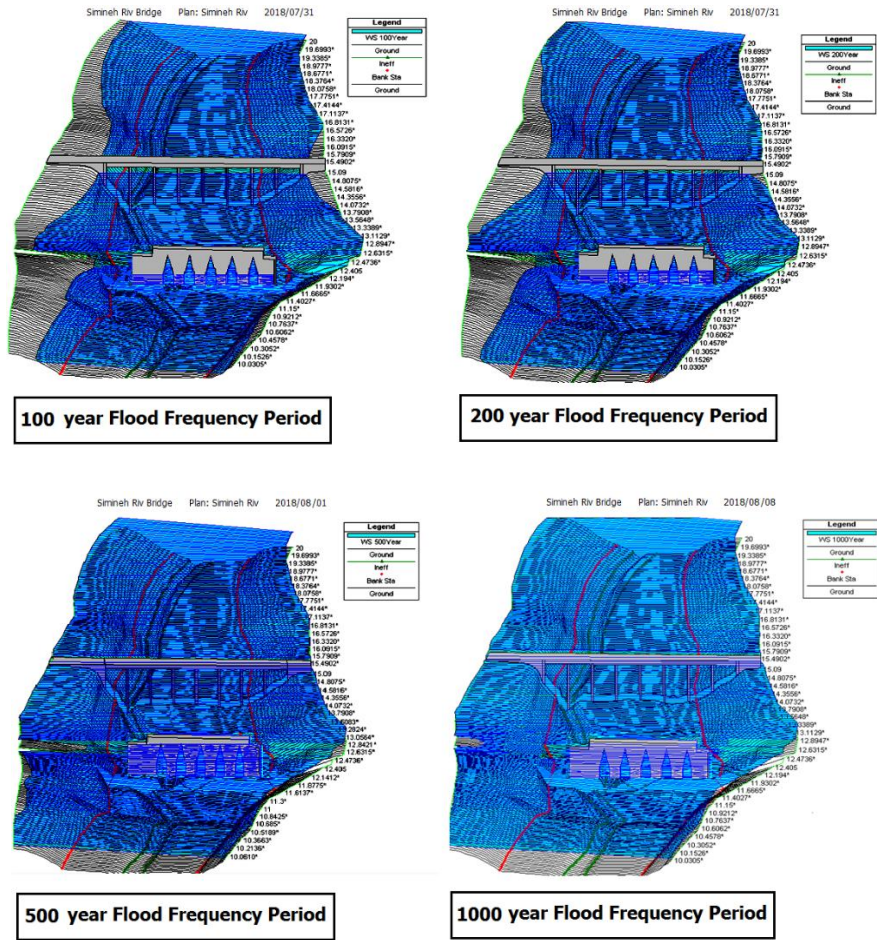
The CSU and Frohlich methods for calculating scour depth

Figures 14 and 15 show the variation in scour depth relative to the pier number of the bridges for discharge with a return period of 75, 100, 150, 200, 300, and 400 years for the CSU and Frohelich methods in the HEC-RAS software.
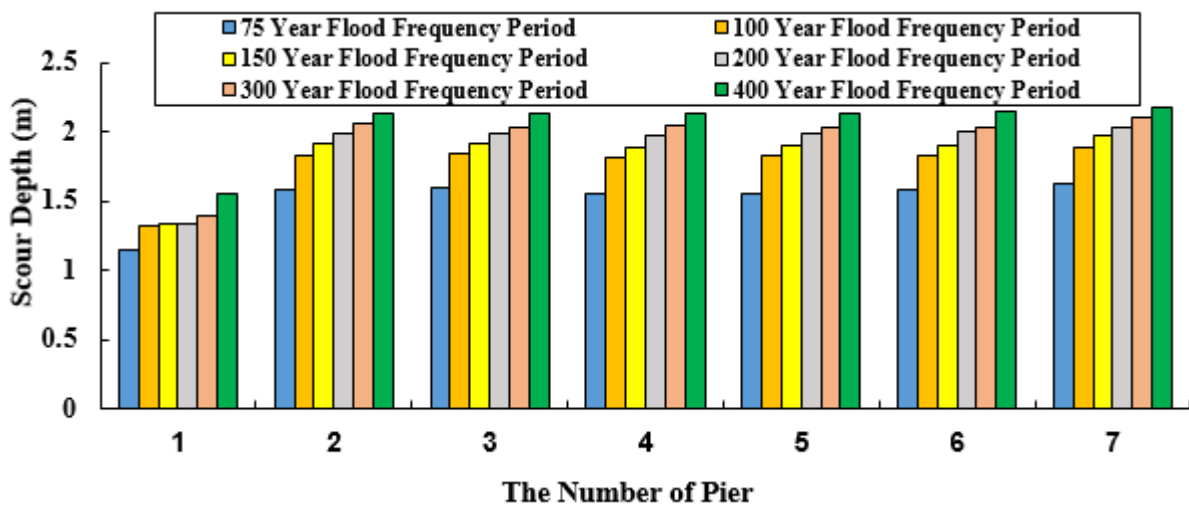

Fig. 14. Scour depth of the Simineh Rood Bridge with CSU method at HEC-RAS software.
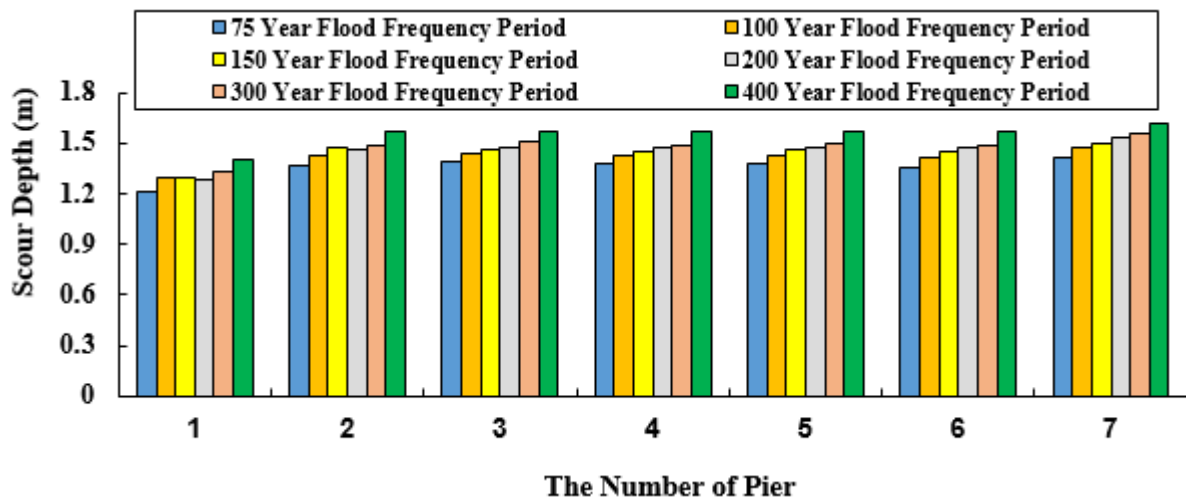
Fig. 15. Scour depth of the bridge with the HEC-RAS Frohlich method.

It can be seen that for bridges, the scour depth varies with different return periods, so that the scour depth for both of the above-mentioned relations in discharge with the 400-year return period for the base of the 7th bridge is the highest, and for the first base is the lowest. Also, during the return periods, the sensitivity of the CSU method to increase the rate of discharge in the calculation of scour depth is more than the Frohelich method. Figure 16 and 17 show the longitudinal profile of scouring caused by a 400-year-old discharge in the HEC-RAS software for both the CSU and Frohlich methods.
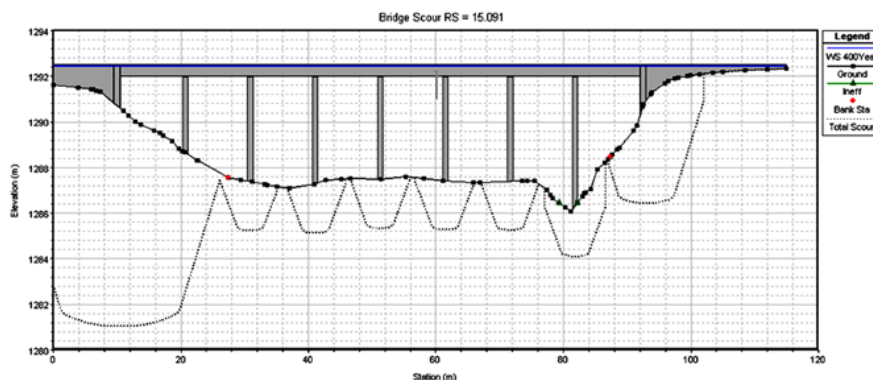
Fig. 16. Scouring caused by a 400 year-old discharge, HEC-RAS CSU method.
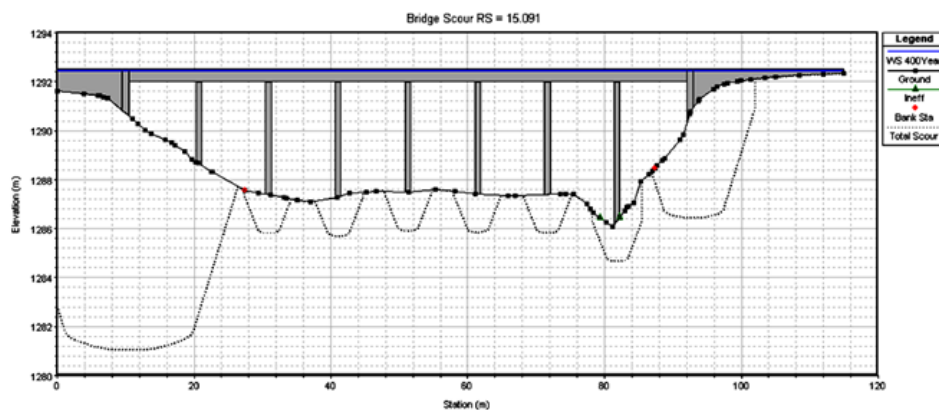
Fig. 17. Scouring caused by a 400 year-old discharge, HEC-RAS Froliche method.

It is observed that the left abutment is strongly influenced by scouring. The reason for this is the curvature of the river flow in this abutment before it collides with the bridge, increasing the velocity in this area relative to the right rail bridge. Figure 18 and 19 show the longitudinal profile of scouring caused by 1000 year discharge for both the CSU and Frohlich methods.
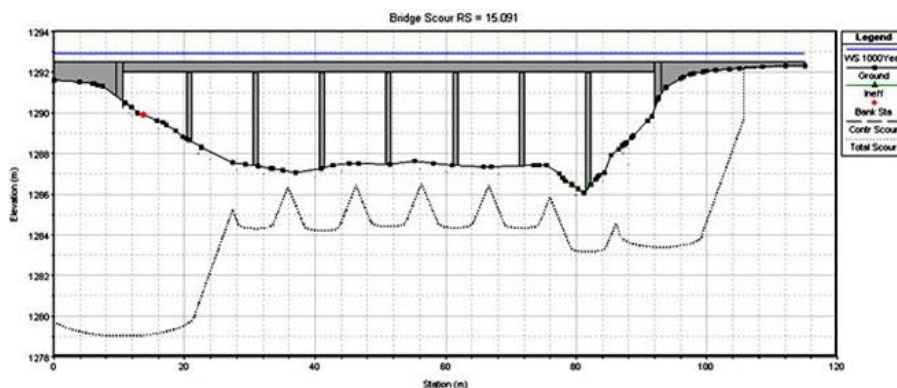
Fig. 18. Scouring caused by 1000 years-old discharge, HEC-RAS CSU method.
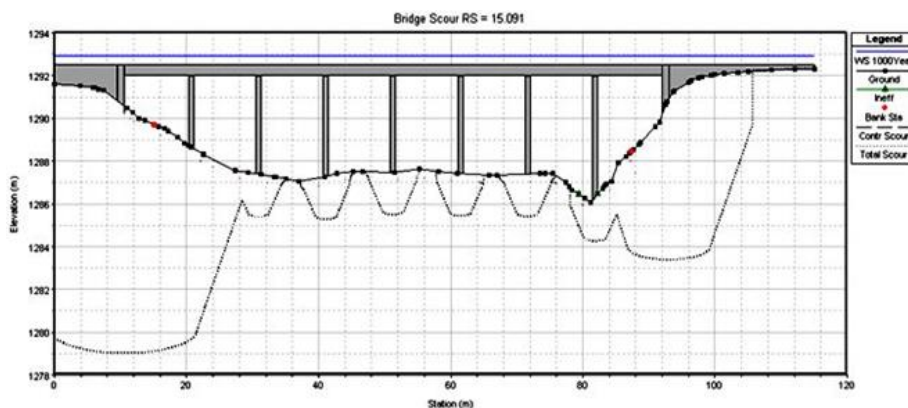
Fig. 19. Scouring caused by 1000 years old discharge, HEC-RAS Froliche method.

During the return period due to the 1000-year-old discharge, which is considered as the peak courier of present research, bridge piers on both the right and left are strongly affected by the flow rate and their scouring rates, respectively. The left and right abutments are 11.19 and 6.32 meters. This height, however, cannot be measured from the bottom of the river. But, it is taken from the abutments, which are about 3 meters above the floor. In Fig. 20 and 21, the highest scour depth is related to the seventh pier, and its value by the CSU and Frohhelich method for discharge with a return period of 500 years is 2.32 and 2.8 meters, respectively, and for discharge with a return period of 100 years, it is 2.86 and 2.45 meters, respectively.

Fig. 20. Scour depth of the Simineh Rood bridge piers with HEC-RAS CSU method.

Fig. 21. Scour depth of Simineh Rood bridge piers with the HEC-RAS Frohlich method.

Table 4 Summarize the scour depth of the Simineh Rood Bridge around the abutments and bridge piers from the HEC-RAS software.

Table 4.   Scour depth of the HEC-RAS.

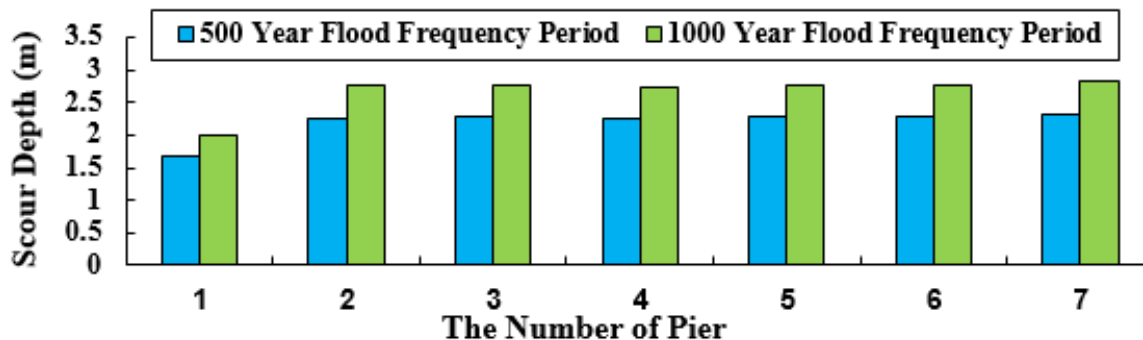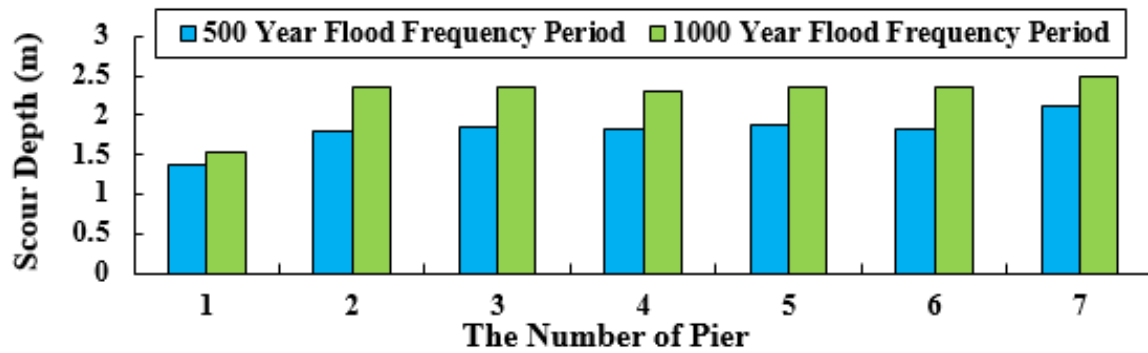| Pier	Scour Depth - CSU Method(m)												
	5	15	25	40	50	75	100	150	200	300	400	500	1000
1	0.46	0.68	0.82	1.01	1.07	1.21	1.39	1.52	1.63	1.71	1.86	1.98	2.14
2	0.68	0.84	1.02	1.29	1.40	1.49	1.38	1.92	1.99	2.07	2.16	2.27	2.66
3	0.61	0.76	0.89	1.21	1.34	1.41	1.72	1.80	1.85	2.00	2.07	2.15	2.58
4	0.60	0.72	0.86	1.20	1.38	1.40	1.71	1.79	1.85	2.00	2.05	2.12	2.54
5	0.59	0.73	0.85	1.22	1.32	1.40	1.70	1.80	1.86	2.01	2.06	2.14	2.55
6	0.67	0.81	1.07	1.30	1.41	1.54	1.83	1.90	2.00	2.09	2.19	2.29	2.64
7	0.73	0.97	1.19	1.38	1.50	1.62	1.89	1.97	2.09	2.22	2.34	2.43	2.84
The right abutment	–	–	–	–	0.58	0.92	1.36	1.79	2.48	4.24	5.11	5..87	6.32
The left abutment	–	–	–	0.64	0.97	1.24	1.86	2.14	3.51	5.77	7.61	10.54	11.19

Pier	Scour Depth - Frohlich Method(m)												
	5	15	25	40	50	75	100	150	200	300	400	500	1000
1	0.35	0.50	0.70	0.86	0.99	1.14	1.29	1.41	1.50	1.59	1.72	1.79	1.98
2	0.51	0.71	0.88	1.01	1.16	1.35	1.46	1.52	1.56	1.68	1.81	1.86	2.45
3	0.49	0.62	0.77	0.92	1.09	1.28	1.35	1.44	1.51	1.61	1.76	1.71	2.36
4	0.48	0.63	0.77	0.91	1.10	1.27	1.33	1.45	1.52	1.60	1.76	1.71	2.30
5	0.48	0.62	0.76	0.92	1.11	1.26	1.34	1.46	1.51	1.61	1.75	1.72	2.31
6	0.55	0.73	0.90	1.09	1.23	1.34	1.43	1.53	1.56	1.63	1.78	1.88	2.32
7	0.62	0.82	1.01	1.18	1.37	1.46	1.57	1.70	1.78	1.86	1.98	2.09	2.51
The right abutment	-	-	-	-	0.42	0.77	1.14	1.40	2.19	3.75	4.57	5.31	5.98
The left abutment	-	-	-	0.47	0.72	1.02	1.49	1.76	3.08	5.37	7.29	9.89	10.60

3.3. Comparison of the Results of Empirical Equations and HEC-RAS Software

Most empirical equations used for all bridge piers and abutment assemblies yield the same number as scour depth. However, software for analyzing steady and unsteady flow can also analyze flow velocity transversely. For each bridge base and abutment, in proportion to speed and flow properties, a suitable number for scour depth is generated. To compare the numerical results of CSU and Frohlich, the ratio of the fourth pier to the middle ground is expressed as a percentage of the results of the empirical equations in Tables 5 and 6.

Table 5. Scour depth ratio of CSU-HEC-RAS software on scour depth, obtained from empirical equations as a percentage (fourth grade).

Return Period (Year)	Empirical Equations										
	Sheppard and Miller (2006)	Melville and Satherland (1988)	Johnson (1992)	Colorado State University (1993)	Shen et al. (1969)	Jain (1981)	Breusers et al. (1977)	Baker (1980)	Laursen and Toch (1959)	Neil (1964)	Lee and Sturm (2009)
5	166	111	218	79	137	118	1720	471	71	13	51
10	171	133	299	84	133	137	1289	273	92	16	59
15	182	136	230	84	130	142	1125	280	99	17	62
25	177	138	235	84	125	147	329	204	109	18	65
40	174	137	241	85	123	153	222	191	117	20	69
50	175	135	244	86	122	155	198	176	120	20	71
100	194	143	268	88	120	167	126	135	138	23	71
150	193	145	279	98	120	171	117	131	143	24	73
200	190	145	273	90	120	174	112	131	147	25	76
250	179	144	277	90	120	175	107	134	149	25	78
300	173	140	279	90	120	176	106	134	150	25	82
350	175	142	290	94	124	182	107	141	156	27	88
400	168	135	272	91	121	179	103	139	154	26	93
500	172	139	286	95	125	187	103	146	162	28	98
1000	206	162	338	111	145	221	115	176	193	33	120

Table 6. Scour depth ratio of the HEC-RAS Frohlich equation on the scour depth, obtained from the empirical equations as a percentage (fourth grade).

Return Period (Year)	Empirical Equations										
	Sheppard and Miller (2006)	Melville and Satherland (1988)	Johnson (1992)	Colorado State University (1993)	Shen et al. (1969)	Jain (1981)	Breusers et al. (1977)	Baker (1980)	Laursen and Toch (1959)	Neil (1964)	Lee and Sturm (2009)
5	266	151	297	107	186	161	2340	640	96	17	38
10	183	142	245	89	142	146	1378	292	99	17	39
15	184	137	232	85	131	143	1134	282	100	17	39
25	165	128	218	78	116	137	306	190	101	17	45
40	153	120	211	75	108	134	194	167	103	17	46
50	150	116	209	73	105	133	169	151	103	17	46
100	147	109	203	67	91	136	95	102	104	17	46
150	142	107	205	65	88	126	86	96	105	18	45
200	136	104	195	64	86	124	80	94	105	18	47
250	127	102	196	64	85	124	76	95	106	18	47
300	122	98	196	64	84	124	75	65	106	18	48
350	121	98	201	65	85	126	74	97	108	18	48
400	120	97	194	65	86	128	73	99	110	19	48
500	120	96	199	66	87	130	72	102	112	19	48
1000	120	94	196	65	84	128	67	102	112	19	48

According to Tables 4 and 5, it is observed that the lowest ratio is related to Neil's (1964) equation for all return periods. On the other hand, the highest ratio for the 5 to 25 year return period is related to the Breusers et al. (1977) equations, and for other return periods (40 to 1000 years), it is related to Johnson's (1992) equation.

4. Conclusion

The present study consists of three sections of scour depth estimates based on the bridge of the bridge between HEC-RAS, empirical equations and their comparison. These results are summarized:
1- For the scour depth calculation, the Nile (1964) and Lee and Sturm (2009) equations are not sensitive to increases in flow discharge. The scour depth is almost constant with increasing discharge.
2- In the Laursen and Toch (1959) equation, there is not much difference in the depth of scouring for the bridge piers in high and low flow discharges.
3- Breusers et al. (1977) and Baker (1980) equations are correlations that are used to calculate the scour depth, in addition to the flow profile, sediment profile, and bridge pier specification. The results obtained in these relationships are also somewhat close.
4- Important characteristics of the Colorado State University (1993) equation are the involvement of the bed conditions and the shape of the bridge pier to calculate the depth of scouring for the bridge piers. Therefore, the results obtained from this equation appear to be reasonable.

5- To apply effective parameters in scour depth, the equations of Johnson (1992), Melville and Satherland (1988), as well as Sheppard and Miller (2006), are the most commonly used parameters. By increasing the return period, the most significant increase in scour depth is related to Melville and Satherland, and the least belongs to Johnson.

6- Calculation results for the scour depth with different return periods showed that for both CSU and Frohlich pier number 7 had the maximum scour and pier number 1 had the minimum.

7- The Frohlich method is not as sensitive as the CSU method in calculating scour depth due to an increase in flow discharge.

8- For discharges with a return period of up to 40 years, no scouring is observed on bridge abutments. While for discharges with a return period of 50 to 1000 years, the scouring rate of the ridge is greatly increased.

9- In discharges with a return period of 500 years, the bridge abutments on both the right and left abutments were strongly affected by the flow discharge, and the scouring rates for the left and right were 5.87 and 10.74, respectively.

10- In discharges with a return period of 1000 years, the bridge abutments on both the right and left are strongly affected by the flow rate, and the scouring rates for the left and right are 11.19 and 6.32 meters respectively.

11- By comparing numerical results and empirical equations, it was determined that the lowest scour depth ratio is related to the Neil (1964) equation for all return periods. This is while the highest ratio for the return period of 5 to 25 years is related to the Breusers et al. (1977) equations, and for other return periods (40 to 1000 years), it is related to the Johnson (1992) equation.

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