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ABSTRACT

The probability of pressurized flow conditions occurring in existing bridges is forecast to increase due to possible changes in extreme precipitation, storm surges and flooding predicted under climate change scenarios. The presence of a pressure flow is generally associated with scouring processes in proximity to the bridge. Scouring can also occur around bridge piers, possibly causing infrastructure failure. While there is a vast literature on bridge pier scour and pressure flow scour, only a few studies have investigated their combined effect. This study will provide a new overview of the main features of bridge pier scour under pressurized flow conditions, based on laboratory experiences. Special focus is placed on the analysis of the flow features under pressure and free surface conditions and to the temporal evolution of the scour. A comparison with existing literature data is also conducted. The results highlight the nonlinear nature of scour processes and the need to consider pressurized flow conditions during structural design, as the interaction between pressure flow and the bridge pier strongly influences scour features and leads to scour depths much greater than the sum of the individual scours created only by pressure flow or pier presence.

Keywords: Bridge decks, Piers, Pressure flow, Scour.

1 Introduction

Road bridges are essential for national transportation systems. Their disruption has severe consequences and can cost many lives (e.g. Deng & Cai, 2010; Gaudio et al., 2012; Melville & Coleman, 2000; Wardhana & Hadipriono, 2003). Bridge scour is one of the most common causes of bridge failure (e.g. Gaudio et al., 2012); Melville & Coleman, 2000; Muzzammil & Siddiqui, 2009; Pagliara & Carnacina, 2010; Wright et al., 2012; Zarrati et al., 2004). This kind of bridge failure has occurred in several locations and under various hydrological conditions, highlighting that bridge scour is a relatively common issue. For these reasons, numerous guidelines and references exist (e.g. FHWA, 2001, 2009; Ryall, 2000; Zhao, 2012; and many others).

New challenges have been arising for bridge design due to climate change and anthropization of catchments (e.g. Gill et al., 2007; NRC, 2008; Wright et al., 2012). Recent studies on climate change have warned about the risks connected to an increase in the magnitude of riverine and coastal flooding due to increased precipitation events, storm surges, sea level rise, and hurricane activity (e.g. Goldenberg et al., 2001; NRC, 2008; Saunders & Lea, 2005; Solomon, 2007; Lyddon et al., 2018a, b,). For example, for the African continent, Chinowsky et al. (2010) estimated that the potential impact of climate change for road infrastructure alone was USD 180 billion.

Several issues are connected to the fact that many existing bridges have been designed based on precipitation and flow rates relative to extreme events from past records, without considering the signature of climate change and climate non-stationarity. Therefore, in some instances their designed discharge capacity is no longer adequate for the current climate (Madsen et al., 2014). Large uncertainties exist in regard to the occurrence of extreme events, and the application of safety factor standards might not be enough to avoid the risk of submerged decks (e.g. Wright et al., 2012). According to Wright et al. (2012), more than 500,000 bridges in the United States are currently deficient and vulnerable to climate change, and the total cost for bridge adaptation to climate change has been estimated as ranging from USD 140 billion to 250 billion.

The most common cause of bridge failure is the scouring of bed material around bridge foundations during floods. According to Cook et al. (2015), hydraulic damage occurred in 52% of bridge failures, with the primary cause being scour. The accumulation of debris and drifts around a bridge pier can then substantially modify and increase local scour patterns (Pagliara & Carnacina, 2013). Normal flow conditions can cause continuous scour at the bridge, but under normal flow the scouring process is frequently slow enough to allow countermeasures via regular maintenance works. Floods have the potential to cause the large-scale removal of material over a short period, with no time for the application of possible remedies and countermeasures; this could lead to structural instability and failure (e.g. Anderson, 2018; Pagliara & Carnacina, 2013). For instance, in the US state of Georgia, in the 1994 flooding due to Storm Alberto, 500 bridges were damaged with 31 having to be replaced after experiencing 4.5–6 m of scour. The total damage, including replacements and repairs to the Georgia Department of Transportation highway system, was approximately $130 million (Arneson et al., 2012). During major floods, the flow regime can switch to pressure flow if the downstream edge of a bridge deck is partially or totally submerged (e.g. Kumcu, 2016). A pressure flow (also defined as vertical contraction or orifice flow) is characterized by a decrease in water depth at the bridge deck. This in turn accelerates the velocity field with respect to the case without a deck, increases turbulence intensity and shear stress, and can cause scour (Umbrell et al., 1998). In comparison to open channel flow, pressure flow significantly increases the erosion potential since scouring is one of the ways energy is dissipated in flood conditions (Guo et al., 2009; Kumcu, 2016).

Given the likelihood of increased flooding due to future climate conditions, and the shortage of publications on the combined effect of pier presence and pressurized flow, more studies are needed to understand the risks posed by pressurized flow. The goal of this paper is to investigate bridge pier scour and nonlinear effects at the bridge pier under pressure flow conditions. Specifically, this study will focus on the maximum scour depth, the temporal scour evolution factor (which represents the rapidity of scour development), and the ratio between scour depths under free and pressurized flow conditions. A comparison with existing literature data and literature formulations is provided throughout the text.

2 Literature Review

Several studies have dealt with the assessment of the maximum scour length, and depth under pressure flow and for a vertical flow contraction, but only a few studies have also considered the presence of piers or abutments. The scour at bridge piers is the local lowering and erosion of the bed elevation around the pier itself (for more details, see Chiew & Melville, 1987; Debnath et al., 2012; Khosronejad et al., 2012; Kim et al., 2014). Based on a set of clear-water experiences under pressure flow conditions, Abed (1991) suggested that the maximum pier scour hole depth could reach from 2.3 to 10 times the scour corresponding to free surface conditions (Arneson & Abt, 1999). Umbrell et al. (1998) proposed an experimental relationship to predict the maximum scour depth, z_{max}, underneath a bridge. The parameter z_{max} was defined as the difference between the original bottom elevation and the deepest point observed in the groove. The experimental relationship was based on laboratory analysis dealing with both orifice and weir flows. According to the experimental results, and on the basis of a theoretical approach which assumed that the velocity at scour equilibrium under the contraction would attain the critical flow velocity, z_{max} was estimated as:

$$\frac{z_{\max}+h_b}{h_0} = \left\{1.102\left[\left(1-\frac{w_e}{h_0}\right)\frac{U}{U_c}\right]^{0.603}\right\} \tag{1}$$

where h_0 is the approaching water depth upstream of the bridge crossing, h_b is the distance between the original bottom and the bridge's lower deck edge, w_e is the water elevation over the bridge's upper deck edge, U is the mean upstream flow velocity, and U_c is the critical flow velocity at which the particles move (subscript "c"), as defined in Richardson and Davis (2001). In Umbrell et al. (1998), the regression analysis of the data showed a coefficient of determination $R^2 = 0.81$ with uniform scattering and no detectable systematic errors. Based on Arneson and Abt's (1999) orifice flow data, Richardson and Davis (2001) proposed a different equation for the maximum scour:

$$\frac{z_{\max}}{h_0} = -5.08 + 1.27\left(\frac{h_0}{h_b}\right) + 4.44\left(\frac{h_0}{h_b}\right)^{-1} + 0.19\left(\frac{U_b}{U_c}\right) \tag{2}$$

where U_b is the averaged flow velocity through the bridge opening before the scour formation.

The authors reanalysed the data using a multi-linear regression analysis suggesting an adjusted R^2 = 0.89. Of the independent variables, $z_{max} + h_b/h_0$ is the most significant with a beta coefficient of 0.87. The flow intensity U_b/U_c was the independent variable with the least significance, with a beta coefficient of 0.13 (Arneson & Abt, 1999). More recently, Lyn (2008) proposed a critical statistical re-examination of the Richardson and Davis (2001), and Umbrell et al. (1998) equations:

$$\frac{z_{\max}}{h_0} = \min\left[0.105\left(\frac{U_b}{U_c}\right)^{2.95}; 0.5\right] \tag{3}$$

By modifying the term U_b/U_c to account for the presence of piers, Lyn (2008) also suggested that Equation 3 could be used to predict the maximum scour for pier pressure flow conditions. The linear regression yielding to Equation 3 displayed an R^2 coefficient of 0.87 with a mean square error of 0.053 in log10 scale (Lyn, 2008).

Another important process connected to the pressurized flow is the generation of buoyant forces, which under certain circumstances might be sufficient to lift the bridge deck (Chen et al., 2016). Examples include the bridge across Biloxi Bay, Miss., which during Hurricane Katrina was subjected to 609 kN due to buoyancy, a value close to the deck weight (e.g. Chen et al., 2009).

The effect of the girder underneath the bridge deck on pressure flow scour was studied by Guo et al. (2009). Assuming all other variables remained constant, they observed that the scour hole increased with the structure thickness. Only one flow depth (0.25 m) at the critical flow velocity was tested, which in part limits the applicability of their results to more general conditions. Guo et al. (2009) also suggested the necessity for further studies on the effect of pier presence under pressure flow conditions on the maximum scour hole, to better clarify non-linear interactions between bridge structures.

In a more recent work, Kumcu (2016) presented a new analysis of temporal pressure bridge scour evolution. The model proposed was based on clear-water scour conditions for steady and unsteady flows. After reanalysing pressure scour data from Arneson (1997), Umbrell et al. (1998), and the new dataset, Kumcu proposed a new set of regression equations to evaluate the maximum scour:

$$\frac{z_{max}+h_b}{h_0} = 0.65 + 0.5\frac{U_b}{U_c} \text{ for } 0.5 \le \frac{U_b}{U_c} < 1 \tag{4a}$$

$$\frac{z_{\max}+\mathrm{h_b}}{h_0} = 1.025 + 0.125\frac{\mathrm{U_b}}{\mathrm{U_c}}\ for\ 1 \le \frac{\mathrm{U_b}}{\mathrm{U_c}} < 1.8 \tag{4b}$$

In the present study, maximum flow intensities were varied from clear-water to live bed conditions; that is, $0.5 < U/\underline{U}_c < 1.8$ (see also Table 1). Table 1 summarizes the conditions and experimental validity of the various previous experiences compared to this work. It is clear that a broad range of applicability of the various equations emerges from past studies, with flow intensities that range from clear-water scour to live bed scour with maximum tested $U/\underline{U}_c < 4$, and various degrees of deck submergence, up to 50%

of the conveyance section. The duration of tests ranged from a few hours (3.5 in the case of Umbrell et al. (1998)), up to 70 hours in the present work.

3 Methods

3.1 Experimental apparatus, flume characteristics and tests execution

All tests were carried out under controlled laboratory conditions. The experimental apparatus consisted of a glass-walled tilting recirculating flume of 0.61 m width and 7.6 m length. A Perspex cylindrical pier of 0.03 m diameter was embedded inside the sediments at the centre of the flume, and a deck was placed over it (Figure 1). Tests were executed as in Pagliara and Carnacina (2010) and Pagliara and Carnacina (2011a), using the same experimental apparatus.

Figure 1 presents a diagram sketch (planar view A, three-dimensional view B, and cross sections C, D) of the experimental setup; it also summarizes the main variables used in the tests. In the figure, x, y, and z are longitudinal, transverse and vertical coordinates measured from the centre of the pier and from the original bottom level; $z_{\max}$ is the maximum scour hole measured at the bridge pier; B is the channel width; D is the pier diameter; and q is the water discharge per unit of width.

The approaching flow depth h_0 is the water elevation measured by averaging 4 water depths at the transverse section located 10 pier diameters upstream of the bridge deck (Hahn & Lyn, 2010). Tests were conducted with different discharge values, different ratios between the water depth underneath the bridge deck (h_b) and the upstream water depth (h_0), different large woody debris accumulations, and different deck sizes and shapes. Specifically, we executed tests using flat decks with two different widths (Figure 1c, $l_{dk} = 6D$; $l_{dk} = 3D$), and with girders along the entire length or only at the ends. Further, we tested different large woody debris accumulations (Figure 1d). Large woody debris accumulations at the bridge crossing exert an important forcing acting over the bridge deck, and can increase the scour depth at the pier base and the likelihood of bridge failure (Pagliara & Carnacina, 2011b). Reference tests without the pier (only the deck) were also executed. The table in the supplementary material details the experimental conditions and results.

An initial set of flow velocity measurements were collected, first using a fixed bottom and then using a mobile bed. The temporal scour evolution at the bridge pier was recorded at regular intervals of time; that is, t = 1, 2, 4, 8, 15, 30, 60 minutes and every hour thereafter. Long tests lasting up to 70 hours were conducted together with shorter experiments normally lasting up to 6 hours, with the intention to provide specific details on temporal scour evolution rather than just in terms of equilibrium. The long tests were used to benchmark any extrapolation assumption of shorter duration when assuming a logarithmic temporal evolution of scour.

The critical flow velocity U_c was calculated according to Wu and Wang (1999). Critical flow velocity and clear-water conditions referred to the undisturbed section "0", upstream of the deck.

Sand of median diameter d_{50} equal to 1 mm, geometric standard deviation of the grain size distribution curve $\sigma_s = (d_{84}/d_{16})^{0.5}$ equal to 1.2, relative specific sediment density $\Delta = (\rho_s/\rho - 1)$ equal to 1.44, and dry and wet sediment angles of repose equal to φ = 31° and φ' = 36° respectively was used for all the experiments. A special series of tests included large woody debris accumulations (Figure 1d). According to Schmocker and Hager (2010), the flow can be greatly affected both by large woody debris and bridge characteristics (Figure 1d, Table 2).

Hereafter, p_d, l_d, and w_d are the large woody debris thickness, width, and length respectively (subscript "*d*"); h_b/h_0 is the deck ratio associated with the accumulation; and $\Delta A = [(w_{dk}-D)(h_0-h_b)+(w_d-D)p_d]/(Bh_0)\cdot 100$ is the additional blockage operated by both the large woody debris and the deck, and normalized to the total flow area.

3.2 Dimensional analysis

To analyse the scour-hole features, a non-dimensional analysis and Buckingham Π theorem were used. The following variables were identified as affecting the maximum scour, $z_{\max}$:

$$z_{\max} = \Psi\ [\text{flow } (v, \rho, q, h_0, g, B), \text{ sediment } (d_{50}, \sigma_s, \rho_s), \text{ pier } (D, Sh_p, Al_p), \text{ time } (t), \text{ deck } (h_b, l_{dk}, Sh_{dk}, Al_{dk})] \qquad (5)$$

where v = kinematic viscosity, g = gravitational acceleration, Sh_p = pier shape factor (subscript "*p*"), Al_p = pier alignment factor, Sh_{dk}, = deck shape factor, Al_{dk} = deck alignment factor, and Ψ is a functional symbol.

By applying the Π–theorem and considering that for our experimental conditions, we tested a single cylindrical pier and a deck with zero skewness angle, Equation 5 can be rewritten as:

$$z_{\max} / D = \Pi\ (D/d_{50}; \rho_s/\rho; U/(g\ \Delta\ d_{50})^{0.5}; UD/v; Ut/D; h_b/h_0; h_0/D; B/h_0; l_{dk}/D, Sh_{dk}) \qquad (6)$$

where $z_{\max}/D$ is the non-dimensional maximum scour, Π is the functional symbol, and the terms in parentheses are the non-dimensional parameters that can be used to describe the maximum scour. Given Equation 6, the following assumptions can be made, which further simplify the problem:

a) when the flow is fully turbulent (Reynolds number R = $UD/v > 2000$), the effect of the Reynolds number can be neglected (Franzetti et al., 1994);

b) for h_0/D = submergence ratio > 1.4, the effect of bow vortex on the scour without a bridge deck can be neglected (Melville & Chiew, 1999; Melville & Coleman, 2000) and the maximum scour hole depth $z_{\max}$ can be normalized by the pier diameter D (Breusers & Raudkivi, 1991; Ettema, 1980; Melville & Sutherland, 1988);

c) according to Oliveto and Hager (2002), in the case of mineral or plastic sediment, ρ_s/ρ only slightly affects the scour process and its temporal evolution;

d) we only used one sediment fraction, and therefore one d_{50}, ρ_s value;

e) sediment cohesion and bed forms are not present for $d_{50} > 0.9$ mm (Melville & Chiew, 1999; Oliveto & Hager, 2005; Raudkivi & Ettema, 1983);

f) sediments can be considered uniform for $\sigma_s < 1.4$ (Dey & Debnath, 2001; Dey & Raikar, 2005), and the armoring effect is negligible if $\sigma_s < 1.3$ (Raudkivi & Ettema, 1985);

g) for an aspect ratio $B/h_0 > 3$, the side wall effect on sediment transport is reduced to less than 20% (van Rijn, 1981), and can thus be preliminarily neglected for the conditions tested in this study; and

h) the densimetric Froude number $\mathsf{F}_{d50} = U/(g\ \Delta\ \mathrm{d}_{50})^{0.5}$ can be reduced to the flow intensity U/U_c, which is a parameter more frequently used when dealing with bridge scour equations.

Thus, the functional relationship in Equation 6 can be simplified as:

$$z_{\max}/D = \mathrm{f}\ (U/U_c;\ T^*;\ h_b/h_0;\ l_{dk}/D;\ Sh_{dk}) \qquad (7)$$

$$T^* = Uth_0/[Dh_0 + (B - D)(h_0 - h_b)] \qquad (7b)$$

where f = functional symbol and T^* is the non-dimensional time which also accounts for the flow area blocked by the bridge deck (see Pagliara & Carnacina, 2010).

4 Results

4.1 Observations of the scour features and of the flow field

Under the present set of experimental conditions, the scour initially forms in the central part of the channel area, and then progresses laterally. Under pressurized flow conditions (deck and no pier), the scour is mostly two-dimensional, while in the presence of a pier the scour assumes three-dimensional features, and the scour depth is much deeper around the pier (Figure 2). Figure 2a presents results for $U/U_c = 0.5$, and Figure 2b presents results for $U/U_c = 1$. The figure shows how the reciprocal interaction between deck and pier strongly influences scour features. Under pressure flow conditions, the scour around the bridge pier is significantly deeper and wider than for free flow. Specifically, for $U/U_c = 0.5$, no scour is observed under free flow conditions (EB36). The scour hole under pressure flow conditions, and in the absence of a pier, is mostly two-dimensional, although a slight increase in the scour is observed toward the corners, which is most likely connected to differences between bottom and wall roughness. Pier pressure scour displays three-dimensional features, and the maximum scour depth, $z_{\max}$, is much deeper than the sum of the maximum scour recorded for the two previous configurations (test EB55). This suggests that the combined presence of a pressure flow field and a bridge pier has a non-linear effect on the scour features. Similarly, for $U/U_c = 1$, the scour depth under pressure flow conditions and in the presence of a pier is significantly larger than the sum of the scours corresponding to the sole presence of a pier under free flow, and to the pressurized flow when considered separately.

4.2 Scour temporal evolution

The evolution in time of the maximum scour depth can be studied by following an approach similar to that of Oliveto and Hager (2002, 2005) and Link et al. (2008) for reference test bridge pier scour, Pagliara and Carnacina (2010) for pier scour under large woody debris accumulation, and Hahn and Lyn (2010) for pressure flow conditions without a pier, where the temporal z_{max} evolution can be approximated by a logarithmic law. In particular, Pagliara and Carnacina (2010) proposed the following:

$$(z_{max}/D)_{T^*} = \xi \cdot \ln(T^*/10) \tag{8}$$

in which ξ = temporal scour evolution factor, the validity of which has also been confirmed in the presence of pressure flow scour.

Figure 3a shows the temporal evolution of the non-dimensional scour z_{max}/D as a function of T^* as measured from laboratory experiments (scattered points), and as predicted by Equation 8 (continuous lines). In the figure, all tests are characterized by a bridge opening of h_b/h_0 = 0.96, U/U_c = 1, h/D = 5.66; values for l_{dk}/D are Sh_{dk} are reported in the supplementary material.

Under free flow conditions (no deck), the pier scour is significantly smaller. The scour significantly increases with a larger deck (EB3 to EB5), especially in the presence of large woody debris accumulations (EB3 to EB48). In contrast, for the given conditions, the presence of girders does not significantly affect the scour (EB3 to EB44). As for the temporal evolution of the scour, Figure 3b presents a comparison between the data of our laboratory experiments and literature data observed by Hahn and Lyn (2010) for pressure scour. In this panel, the horizontal and vertical axes follow Hahn and Lyn (2010): z_{max} has been normalized by $h_0 - h_b$ rather than D to facilitate the comparison with the ordinal data, and t has been normalized by T_{HL}; that is, the time scaling factor proportional to the sediment transport rate (eq, 1 in Hahn and Lyn (2010)). Hahn and Lyn's tests have been carried out for h_b/h_0 = 0.8, a rectangular deck of l_{dk}/h_0 = 1.5, with two different flow intensities (U/U_c = 0.64, U/U_c = 0.73) and no pier. A direct comparison with the test EB29 (reference test, with pier and without deck, for U/U_c = 0.75) revealed a different temporal behaviour compared to Hahn and Lyn's data. In the absence of a deck, and for t/T_{HL} < 1, our data suggest a slower development, although no information on the scour formation at an earlier stage is present in Hahn and Lyn (2010). Despite these differences, the scour depth remains of the same order of magnitude when compared to the reference test. In the presence of a deck at h_b/h_0 = 0.8 and a pier (EB30), the scour is more than three times deeper than that without the deck and is significantly higher than that observed in Hahn and Lyn's experiments. A simple linear superposition of the two scours, a simple design methodology that could be adopted in the absence of any other available dataset, would have produced a significant underestimation of the total scour, with a value significantly lower than the scour with pier and deck. This highlights the strong non-linear interaction created by the combination of vertical contraction and the presence of pier scour.

A more detailed analysis of the scour evolution factor, ξ (Equation 8), offers the opportunity to investigate further the scour evolution in time and those parameters which are more influential.

Figure 4a shows the dependence of ξ on h_b/h_0, for tests carried out with flat decks (Sh_{dk} = R), $0.5 < U/U_c < 1$, $3 < l_{dk}/D < 6$, and $2.64 < h_0/D < 5.71$.

The variable ξ linearly decreases with h_b/h_0. In fact, the larger the vertical contraction operated by the deck, the smaller the area where discharge can flow (the smaller is h_b/h_0), the higher the velocity under the deck, and the faster the temporal scour evolution observed at the base of the pier. Similarly to observations in free surface conditions, increasing flow intensity U/U_c also causes higher erosion rates in correspondence with the groove, due to the larger downflow intensities occurring at the base of the pier.

The scour evolution factor, ξ, is also greatly affected by the width of the deck l_{dk}/D. This is because a boundary layer develops underneath the deck, which further contributes to increasing the flow velocity in proximity to the pier section; the development of this is linked to the deck's length. For $l_{dk}/D = 6$, ξ always shows larger values compared to $l_{dk}/D = 3$.

By using a multiple regression analysis, ξ can be evaluated as follows:

$$\xi = b_1\left(\frac{h_b}{h_0}\right) + b_2\left(\frac{U}{U_c}\right)^2 + b_3\left(\frac{U}{U_c}\right) + b_4\left(\frac{l_{dk}}{D}\right) \qquad (9)$$

where b_1 = –0.75 (–0.88; –0,61); b_2 = –0.595 (–0.74; –0,45); b_3 = 1.5 (1.23; 1.75); and b_4 = 0.012 (0.006; 0.018) are b_{i-th} regression coefficients. The values in brackets relate to the 95% confidence intervals of the estimated b_{i-th}, which can be used for safety evaluations.

Figure 4b shows the relationship between the measured and calculated values of ξ (Equation 9). The scatter points refer to the laboratory experiments of panel (a), as well as to existing literature data (Abed, 1991). Errors are generally less than 25% except for one test carried out at $U/U_c = 0.5$, which might have been affected by some bias during its execution and a larger relative error occurring when measuring small scour hole depths. Abed's (1991) data show a good agreement with Equation 9, although a general over-prediction of 20% can be assessed. In particular, three of the pier scours under pressure flow conditions, and carried out for h_0/D = 1.6, are rather over-predicted. However, it is important to observe that Abed's tests were carried out for $l_{dk}/D = 12$ and different deck and pier configurations. Specifically, Abed's tests were carried out using a round-nose pier of 12D length. In those tests, the surface roller was located immediately at the upstream edge of the pier, while in the present tests the surface roller is located far from the pier and a second smaller and weaker roller forms underneath the deck at the downstream edge of the pier.

4.3 Differences in scour between free surface and pressure flow conditions

To assess the effect of pressure flow conditions over the pier scour hole, the deck factor K_{dk} has been defined as the ratio between the temporal scour coefficient ξ under pressure flow conditions and that of the reference test ξ_{pt} (no deck) under free surface flow. Two sets of tests were thus conducted, with and without the deck but with otherwise identical hydraulic conditions. The deck factor is defined as follows:

$$K_{dk} = (z_{max}/D)_{T^*} / (z_{max-pt}/D)_{T^*} = [\xi \cdot \ln(T^*/10)]/[\xi_{pt} \cdot \ln(T^*/10)] = \xi / \xi_{pt} \qquad (10)$$

Figure 5a shows the deck factor, K_{dk}, as a function of h_b/h_0. In the presence of a pressure flow field, the temporal scour evolution factors are around 2.52 times those for the reference test for $h_b/h_0 = 0.75$, and $l_{dk}/D = 6$; this indicates a larger erosion potential observed at the base of the pier, which in turn could greatly increase the likelihood of pier failure. The variable K_{dk} reduces as h_b/h_0 increases, owing to the effect of smaller deck submergences on the pressure flow. Generally speaking, as conditions approach free surface flow, the deck factor tends to 1. Accordingly, longer decks ($l_{dk}/D = 6$) show larger K_{dk} compared to decks with $l_{dk}/D = 3$, whilst U/U_c only slightly affects K_{dk}. With the help of a multi-regression analysis, the equation that best describes the deck factor has been found to be the following:

$$K_{dk} = \left(\frac{h_b}{h_0} - 1\right)\left(-7.33 + 4.40\frac{U}{U_c} - 0.68\frac{l_{dk}}{D}\right) + 1 \qquad (11)$$

For the case without a deck, the functional relationship reduces back to the case of free surface pier scour, where for $h_b/h_0 = 1$, $K_{dk} = 1$.

Calculated, $K_{dk\mathrm{calc}}$, and measured, $K_{dk\mathrm{meas}}$ values (Figure 5b) show good agreement, with error bars less than 25% and $R^2 = 0.8$. Tests with girders are also shown in the figure. The agreement between Equation 11 and K_{dk} observed for a deck with girders indicates that, in the tested conditions, the presence of the girders only slightly affects the scour process. Conversely, when rectangular large woody debris accumulations are present, the factor K_{dk} can be up to 2.5 times larger than in the absence of accumulations (section 4.4).

4.4 Effect of large woody debris accumulation on pier pressure scour

Pressure flow conditions increase the scour hole at the base of the pier. This increase is proportional to the upstream wetted area blocked by the deck and thus to the ratio h_b/h_0. The presence of large woody debris accumulations further increases the blockage at the pier section (De Cicco et al., 2018).

To account for the reduction in cross section due to the presence of floating objects, a different parameter, referred to as blockage ratio ΔA, can be introduced in the case of pressure flow and debris accumulations (Martín-Vide & Prió, 2005; Pagliara & Carnacina, 2010). The blockage ratio $\Delta A = [(w_{dk}–D)(h_0–h_b) + (w_d–D)p_d]/(Bh_0)\cdot 100$ is the blockage operated by both the debris and the deck, normalized to the total flow area. Figure 6 shows the scour evolution factor as a function of the blockage ratio ΔA (Figure 6a). The scour evolution factor linearly increases with ΔA and, similarly to what has been done for the other parameters, the following equation is proposed to describe the relationship between the different variables:

$$\xi = 0.0085\Delta A - 0.61\left(\frac{U}{U_c}\right)^2 + 1.543\left(\frac{U}{U_c}\right) + 0.012\left(\frac{l_{dk}}{D}\right) - 0.7822 \qquad (12)$$

Calculated and measured values of ξ are plotted in Figure 6b. Cases with only debris accumulations show the largest differences with respect to the values calculated with Equation 12. In fact, the actions of debris accumulations differ from those of the deck because debris only covers part of the total flow width and debris alone cannot lead to pressure flow conditions. Table 2 illustrates the different levels of large woody debris accumulations used in the tests.

5 Discussion

Bridge pier scour has been extensively analysed and discussed in several studies (e.g. Brandimarte, 2012; Qi et al., 2016). However, despite the important economic and social impacts of bridge failures, the problem is far from being solved, due to the complexity of the processes at play. Furthermore, as illustrated in the previous literature review, there is a paucity of studies focussed on the scour at bridge piers under pressure conditions.

To provide further insights into relationships that can possibly be used to predict the maximum scour depth at the bridge pier under pressure conditions, the non-dimensional scour depth as measured in all the tests is compared against the non-dimensional scour depth calculated from different equations; that is, Equations 1, 2, 3 and 4 and a normalized form of Equation 8. Equations 1, 2 and 4 refer to the scour under pressure conditions and without a pier; Equation 3 refers to the pier scour, and has been proposed as suitable for both pressure and free surface flow (Arneson, 1997; Hahn & Lyn, 2010; Umbrell et al., 1998).

The range of variables tested by Arneson (1997) and Umbrell et al. (1998) have been summarized by Hahn and Lyn (2010); Lyn's (2008) elaborations were based on the datasets of Arneson (1997) and Umbrell et al. (1998). Umbrell et al.'s (1998) equations were based on short test durations (3.5 hours), while Arneson's (1997) test duration was not reported. Hence, to compare the performance of their equation with the present datasets, measured final scour hole depths have been normalized by h_0, and Equation 8 has been rewritten as:

$$(z_{\max}/h_0)_{T^*} = \xi\cdot\ln(T^*/10)/(h_0/D) \qquad (13)$$

Equation 13 shows an overall good agreement with our experimental data (Figure 7). However, when comparing the present dataset with previous literature equations, a general scatter is observed. Equation 3, which is the only one referring to the pier scour, largely underestimates the data. Scours calculated from Equation 1 also underestimate $(z_{\mathrm{max}}/h_0)_{\mathrm{meas}}$, with a larger scatter than that of Equation 3. Finally Equation 2 both underestimates and overestimates $(z_{\mathrm{max}}/h_0)_{\mathrm{meas}}$. It is worth observing that overestimated values generally refer to $h_b/h_0 > 0.94$; these are values outside the range tested by Arneson (1997) (see Lyn, 2008).

The underestimation of the scour can be further highlighted by considering the maximum scour as a function of flow intensity as predicted by one of the equations (e.g. Equation 3, Figure 7b). Equation 3 is based on the definition of contracted velocity U_b/U_c. Lyn (2008) suggests that the contracted velocity can indirectly account for both pressure flow and pier presence. However, the underestimation of the scour illustrated in the present dataset highlights the possible importance of the non-linear interactions between pier presence and pressure flow. In fact, the pressure flow pier scour does not linearly depend on the contracted velocity; rather, it is connected to a scour evolution close to the bridge pier scour mechanics, which is not fully considered in the definition of contracted velocity. Additionally, Kumcu (2016) observed how the duration of the test in Arneson's and Umbrell's dataset could have possibly biased Equation 3. Specifically, the duration of the tests was deemed not sufficient to attain the maximum scour; this could potentially lead to a dangerous underestimate of design scour conditions. This is further highlighted when comparing the data of Figure 7a, which generally show a deeper calculated scour than that of Equation 3. The non-linear effect is again highlighted when comparing Equation 4 with the present dataset, especially for larger values of z_{max}/h_0.

6 Conclusions

Bridge pier scour is a remarkably complex phenomenon; pressurized flow conditions further enhance the complexity of the problem. Understanding the full scour potential is extremely important for increasing bridge resilience and reducing the risks to life and economic losses. In this context, a series of laboratory experiments were used to quantify bridge pier scour features with different deck configurations and discharge values, and with and without debris accumulations. The results show that pressure flow conditions accelerate the scour hole's temporal evolution compared to reference tests under free surface conditions. Large woody debris accumulations further increase the scour. The temporal scour evolution is affected by the following: the ratio between the water depth underneath the bridge and the free surface water level, the flow velocity, the width of the deck, and the blockage ratio accounting for large woody debris accumulation. Based on a non-dimensional analysis and data elaborations, we developed simple equations aimed at quantifying the maximum scour depth and the temporal evolution of the scour as functions of the above-mentioned variables. We also provided a comparison with existing literature data. The results highlight the need to consider the occurrence of pressurized flow conditions during structure design, as the interaction between pressure flow and bridge piers strongly influences scour features and leads to a bridge scour much larger than the sum of the scours created only by the pressure flow or the pier. Such a scenario is especially important during extreme flood conditions, when maximum scour processes are expected to occur within a small time frame. Our results suggest that practitioners should consider pressurized flow conditions as a likely scenario during flooding, as flood conditions are potentially more deleterious in terms of risk to infrastructures.

Data availability

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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Variable		Umbrell et al., 1998	Richardosn and Davis, 2001; Lyn, 2008	Kumcu, 2016	Present study
Pier	[-]	no	yes	no	yes
h_b/h_0	[-]	0.5 ÷ 0.95	0.3÷1	0.19÷065	0.46÷0.99
U/U_c	[-]	0.42 ÷ 1.06	0.3÷4	0.56÷1.8	0.5 ÷ 1
d_{50}	[m]	0.0003 ÷ 0.0024	0.0006 ÷ 0.0033	0.9	0.001
h_0	[-]	0.305	NA	0.115÷0.131	0.07 ÷ 0.176
h_b	[-]	0.153 ÷ 0.305	0.17÷0.43	0.05÷0.075	0.038 ÷ 0.165
z_{max}	[-]	0.004 ÷ 0.13	NA	NA	0.099
t_{max}	[h]	3.5	NA	32	6 (70 max)
B	[m]	1.8	0.914	1.5	0.6
z_{max}/h_0	[-]	0.013 ÷ 0.43	-0.1÷0.6		0 ÷ 1.23
Notes	[-]		After Anerson and Abt 1999 data	l_{dk} = 0.30 m	9<l_{dk}<0.12 m

Table 2. Large woody debris accumulation: flow shallowness, bridge openings, large woody debris dimensions and blockage ratios

Accumulation	h_0/D	h_b/h_0	p_d/D	w_d/B	l_d/D	ΔA¶
[-]	[-]	[-]	[-]	[-]	[-]	[-]
D1	2.66	‡ >1 †	2	‡ 0.5	3	4.35 ‡
	5.66 §		4 §			30.5 §
D2	5.66	0.95	2	0.35	3	15.23
D3	5.66	0.75	2	0.32	3	33.31
D4	2.66	0.95	1	0.33	3	15.23
D5	2.66	0.65	0.5	0.41	3	40.02

† free surface flow conditions, ‡ minimum and § maximum range of tested values,

¶ nominal values

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Fig 1. Diagram sketch of the experimental apparatus and notation: (a) top view; (b) three-dimensional view of the cross section in panel (a), scour hole indicated on the left side. c) side view of the cross section in panel (a) for two different deck widths, l_{dk}=6D and l_{dk}=3D. Girders are indicated as well, note that p_{dk} is defined from the girders edges when present; (d) side view and transverse view of tests with large woody debris accumulation underneath the deck.

Fig. 2 Transverse scour sections for (a) U/U_c =0.5 and (b) U/U_c=1

Fig. 3. (a) Non-dimensional maximum scour as a function of the non-dimensional time. Scatter points are experimental data, continuous lines represents equation 8. (b) Comparison with Hahn and Lyn (2010) data.

Fig.4 (a) dependence of the scour evolution factor ξ from h_b/h_0 at different U/U_c and l_{dk}/D for flat shaped decks. (b) calculated versus measured ξ, R^2=0.93 (symbols are as in (a)).

Fig. 5 (a) dependence of K_{dk} from h_b/h_0 for Sh_{dk}=R at different U/U_c and l_{dk}/D, and (b) calculated K_{dk} (Eq.10) agreement versus measured K_{dk}, R^2=0.8.

Fig. 6 (a) dependence of the scour evolution factor ξ from ΔA at different U/U_c and l_{dk}/D for flat shaped decks (R); (b) calculated ξ (Equation 12) versus measured ξ.

Fig. 7 (a) comparison between measured values of ($z_{\max}/h_0$), and values calculated from present study equation (Eq.13), Umbrell et al. (1998) (Equation 1), L. A. Arneson (1997) (Equation 2), and Lyn (2008) (Equation 3), Kumcu (2016) (Equation 4) (b) pier scour in pressure flow conditions and Lyn (2008) equation (Equation 3).