

Evaluation of local scour depth around complex bridge piers

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ABSTRACT: The main objective of this study is to evaluate and compare three available methods for the prediction of the local scour depth around complex bridge piers using new experimental data. The complex pier model used in the study was embedded in uniform diameter (0.86 mm) sand. A total of 9 tests were carried out in a 40 m long and 2 m wide rectangular sand-bed tilting flume. Tests lasted between 15 and 25 days, for a variety of pile cap positions relative to the initial bed under steady clear-water flow conditions. The comparison of predictions with measurements, including some selected from the literature, allows stating that the HEC-18, Auckland and Florida methods potentially lead to both overestimations and underestimations, depending on the relative position of the pile cap. It became obvious from the study that differences may be influenced by the test stop criterion adopted in the experiments.

1 INTRODUCTION

Local scour around bridge foundations can lead to the partial failure or the collapse of bridge piers and decks. In the last six decades, the local scour at single piers (Fig. 1a) has been extensively studied. The studies focused on the understanding and characterization of (i) the phenomena involved in the scour process (e.g. Dargahi 1989, Ahmed & Rajaratnam 1998, Graf and Isiaro 2002, Dey & Raikar 2007), (ii) the temporal evolution of maximum scour depth (e.g. Franzetti et. al 1982, Unger & Hager 2007, Lança et. al 2010), (iii) the prediction of the maximum scour depth (e.g. Raudkivi & Ettema 1983, Melville & Sutherland 1988, Melville & Coleman 2000, Richardson & Davis 2001) or (iv) the mathematical modelling of the scour process (e.g. Kirkil et. al 2008). In contrast, studies on local scour at non-uniform cross section piers (Fig. 1b-e) are more recent. In the last three decades they cover scour at pile groups (e.g. Hannah 1978, Jones 1989, Zhao & Sheppard 1999) and columns founded on caissons or slabs (e.g. Jones et. al 1992, Umeda et. al 2010).

Physical and economic considerations are increasingly leading, to the design of bridge piers composed of a pier column founded on a pile cap, supported by an array of piles (Fig. 1e). Studies on scour at complex piers (Fig. 1e) were mostly carried out during the last decade (e.g. Salim & Jones 1998,

Richardson & Davis 2001, Coleman 2005, Sheppard & Renna 2010).

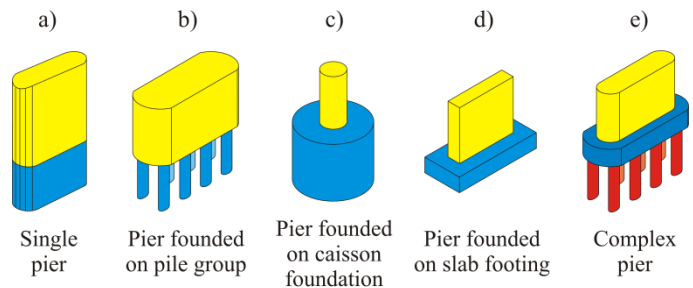


Figure 1. Bridge pier foundation

The use of complex piers is now common. Recent examples in Portugal are those of the Vasco-da-Gama or Lezirias bridges, over the Tagus River. According to Coleman (2005), for these piers, the pile cap is typically founded on the riverbed at construction. Over the life of such a pier, combinations of limitations of design features, long-term river bed degradation or aggradation and contraction scour can result in changed pile cap elevations relative to the riverbed, with varying combinations of the three structural elements being exposed to the flow and varying depths of resulting local scour below the surrounding bed levels, as shown in Figure 2.

The scour depth predictors are usually based on experimental studies. In the case of complex piers, three of the most widely used methods are HEC-18 (Richardson & Davis, 2001), Auckland (Coleman

2005, Melville et. al 2006) and Florida (Sheppard & Renna, 2010).

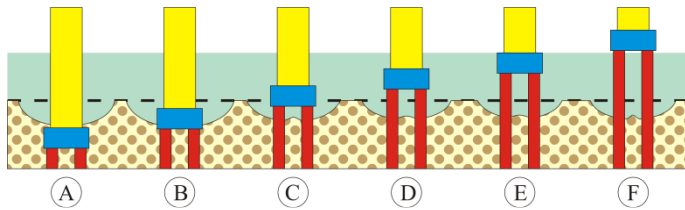


Figure 2. Scheme of the pile cap elevations

The main objective of this work is to study experimentally local scour around complex piers under clear-water flow condition by obtaining a new set of data to evaluate and compare the currently available predictors of maximum scour depth at complex piers. The most important feature of the experiments carried out in this work is their long duration.

2 PREDICTORS OF SCOUR DEPTH AT COMPLEX PIERS

2.1 HEC-18 method

The HEC-18 design method for complex piers was suggested by Richardson & Davies (2001). A superposition approach, comprising the conceptual separation of the pier components and the determination of the scour depths for individual components is adopted. The scour depth is calculated by adding the scour depth produced by each component of the complex pier exposed to the flow. The method suffers from some limitations in the cases where the pile cap is partially or completely buried.

2.2 Auckland method

Coleman (2005) and Melville et al. (2006) suggested a local scour predictor that combines expressions for scouring respectively at uniform piers, piers founded on a caisson or slab footing and pile group with a surface debris raft. The predictor considers five cases of the initial pile cap elevation relative to the surrounding bed level. For Case I (Fig. 2, position A), the top of the pile cap is buried below the bottom of the scour hole, and local scour is affected only by the column. For Case II (Fig. 2, position B), the top of the pile cap is exposed to the flow in the scour hole and local scour is typically reduced from that of the column alone, due to interception of the down-flow by the pile cap. For case III (Fig. 2, positions C and D), the three elements are exposed to the flow and the scour depth tends to be maximal. For case IV (Fig. 2, position E), the top of the pile cap is in line with the water surface, which is similar to a pile group with a floating debris raft of the size of the pile cap. For case V (Fig. 2, position F), the scour hole is only influenced by the pile group.

2.3 Florida method

According to Sheppard & Renna (2010), the effect of each pier component can be evaluated as the scour depth at one equivalent single cylindrical pier. This, in turn, depends on pier shape, size, location and alignment relative to the flow direction as well as on flow characteristics and sediment properties. The total diameter of the complex pier can be approximated by the sum of the equivalent diameters of the complex pier components. The mathematical expressions of these equivalent diameters were established from data of experiments carried out at Florida. The procedure of equilibrium scour depth prediction considers three cases: case 1, for the pile cap above the bed; case 2, for partly buried pile caps; case 3, for completely buried pile caps. These cases are defined by referring to initial bed level.

3 EXPERIMENTAL SETUP

Experiments were carried out in a 40.0 m long glass-sided rectangular tilting flume having a cross section of 2.0 m wide and 1.0 m deep. The flume can be tilted. In this study the slope was fixed at 0.2%. A concrete false bottom was installed in order to obtain two sand recess boxes. Two 5.0 m long and 0.4 m deep sand-filled recess boxes were left in the false bottom. Each of the sand boxes was preceded by a 2.0 m long accelerating ramp and a 7.0 m long fixed-bed reach and followed by a 3.0 m long fixed-bed reach (Fig. 3). A stilling basin was designed at the end of the first 3.0 m long fixed-bed reach, so as to store the sand removed by the flow from the upstream sand recess box. The accelerating ramp tends to re-generate a uniform flow distribution, this way “disconnecting” scour phenomena and associated flow fields in both boxes. A 0.3 m thick and 0.2 m long fine gravel mattress was embedded at the upstream end of the sand recess boxes, levelled with the adjacent concrete bed, in order to prevent scouring at the transition with this fixed bed. In this study, only tests carried out in the downstream recess box are considered.

The experimental setup includes a closed hydraulic circuit (Fig. 3) where the discharge can be varied from 0.0 to $0.25 \text{ m}^3 \text{ s}^{-1}$. The flow discharge is measured by mean of an electromagnetic flowmeter, positioned on the feeder pipe. At the inlet of the flume, a metallic grid was installed in order to guarantee the transversal flow distribution to be uniform. At the downstream end of the flume, a sluice gate allows the regulation of the water level inside the flume (Fig. 3).

The bed recesses were filled with uniform quartz sand of median size, $d_{50} = 0.86 \text{ mm}$, and geometrical standard deviation, $\sigma_g = 1.34$.

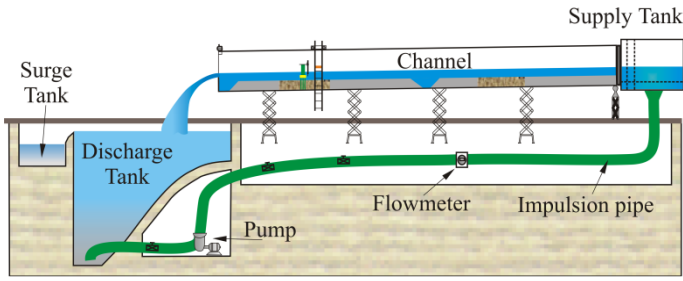


Figure 3. Schema of the hydraulic circuit

The complex pier was simulated by a model built with a concrete column and pile cap and aluminium piles, placed at ≈ 2.0 m from the upstream border of the bed recess. This model is formed by a round-nose rectangular column of 0.11 m width, D_c , and 0.43 m length, L_c . A similar shape round-nose rectangular pile cap of 0.20 m width, D_{pc} , 0.52 m length, L_{pc} , and 0.09 m thickness, T , was placed on top of eight piles (two rows of 4 piles), spaced by 2.5 times the pile diameter ($S_p = 2.5D_p$) and pile diameter of 0.050 m (Fig. 4). The pile cap top elevation, H_c , is measured from the initial bed level and it is positive when the top of the pile cap is above the initial bed level, as shown in Figure 4.

With the purpose of evaluating scouring effects around complex piers, nine different initial elevations of the pile cap relative to surrounding bed level were tested. These elevations correspond to four typical situations in which the complex pier can be after construction, degradation and contraction scour occurs. These situations are (Fig. 5): (1) pile cap out the water (test 1); (2) pile cap under water above the bed, (tests 2 and 3); (3) pile cap partially buried in the bed (tests 4, 5 and 6) and (4) pile cap completely buried in the bed (tests 7, 8 and 9).

In order to achieve clear-water conditions, the flow depth, h , was fixed as 0.20 m and the flow discharge was fixed as $0.134 \text{ m}^3\text{s}^{-1}$, to which corresponds an approach velocity, U , of 0.335 ms^{-1} . The critical velocity was obtained from Sheppard & Renna (2010) equation, as being $U_c = 0.362 \text{ ms}^{-1}$. The resulting flow intensity ratio was $U/U_c \approx 0.93$.

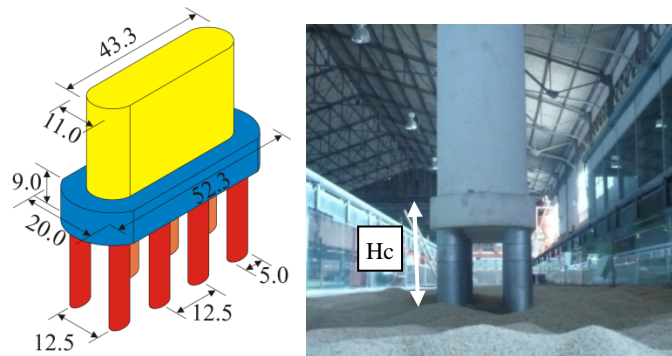


Figure 4. Dimensions and detail of the complex pier model (units in centimeters)

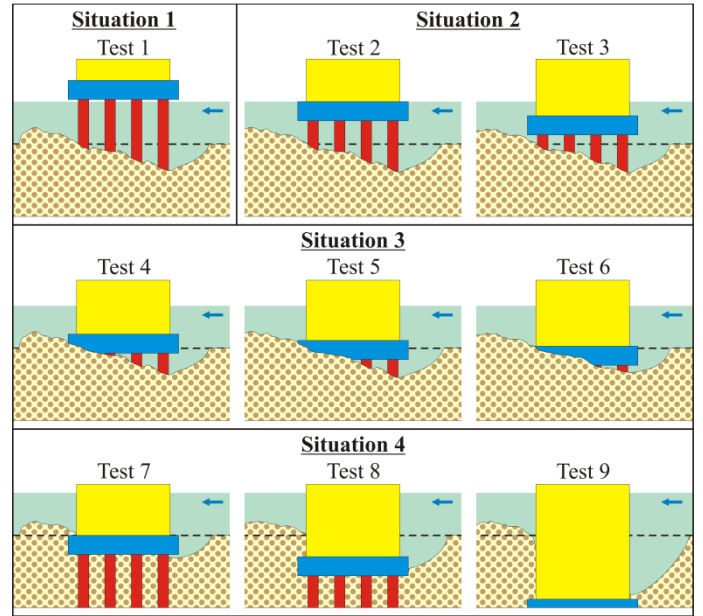


Figure 5. Tests pile cap type situations and studied positions

The contraction ratio was $B/D_{pc} \geq 10.0$ (B being the flume width) and B/h was taken into account in order to guarantee that the flume behaves as wide ($B/h \geq 5.0$). These values indicate that both wall effect and contraction effect are negligible. The relative sediment sizes, D_c/d_{50} , D_{pc}/d_{50} and D_p/d_{50} , were 127.9, 232.6 and 58.1 respectively. According to the literature (e.g. Sheppard et. al, 2004), the scour depth can be expected to be maximal, particularly when the piles are exposed to the flow.

4 EXPERIMENTAL PROCEDURE

Prior to each test, after the complex pier model was installed, the sand bed was compacted and levelled with the adjacent concrete level bed. The sand zone around the complex pier was covered with thin metallic plates to avoid uncontrolled scour at the beginning of the experiment. The flume was slowly filled with water to allow air trapped in the sediment to escape. The sand bed was levelled and the sand zone around the complex pier was covered with thin metallic plates again. The flume was gradually filled, imposing high water depth and low flow velocity. The flow depth was regulated by adjusting the downstream sluice gate. Once the flow depth and the discharge were established, the metallic plates were removed and the experiment started. Scour was immediately initiated and the depth of scour hole was measured, to the accuracy of ± 0.1 mm, with an adapted point gauge, every ≈ 10 minutes during the first hour. Afterwards, the intervals between measurements increased and, after the first day, two or three measurements were carried out per day. The tests were stopped according to criteria defined in the section 5.3.

In the present study, depending on the test, the pile cap was placed above the bed and partially buried or fully buried in the bed. In each situation, one, two or three complex pier elements were in contact with the bed. For this reason, it was necessary to choose three measuring points, as shown in Figure 6: point 1 and point 2 were located, respectively, in front of the column and in front of the pile cap, to carry out measurements when the talweg of the scour hole occurred in front of these two elements; point 3 was located in front of one of the upstream piles, which implied drilling one hole through the pile cap to operate the point gauge (Fig. 6).

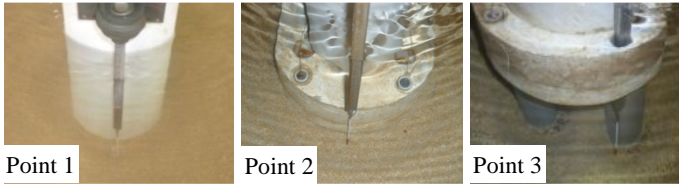


Figure 6. Scour depth measuring points

5 RESULTS AND DISCUSSIONS

5.1 Scour depth evolution

At single piers under clear-water flow conditions, the scour depth evolution follows a logarithmic trend and three phases of the scouring process may be identified: initial phase, principal phase and equilibrium phase (e.g. Ettema 1980; Couto & Cardoso 2001). At complex piers those phases of the scouring process can display different trends, depending on the initial elevation of the pile cap.

When the pile cap is out of the water (situation 1), the time evolution of the maximum scour depth is similar to that of the single pier case, following a unique curve, as shown in Figure 7. The scour process begins in front of the upstream piles, at individual holes, until they merge into one single scour hole; the maximum depth locates itself in front of the upstream piles of the group. In this case, measurements of the maximum scour depth were carried out at point 3 (Fig. 6).

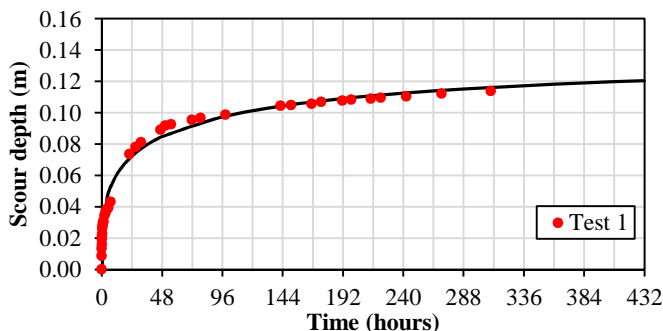


Figure 7. Scour depth evolution for situation 1

When the pile cap is placed in the water column above the bed (situation 2), the scour depth evolution is analogous to that of situation 1, displaying a unique trend as shown in Figure 8, since the maximum scour depth occurs immediately upstream of the piles facing the flow. In this case, measurements of the scour depth were also carried out at point 3 (Fig. 6).

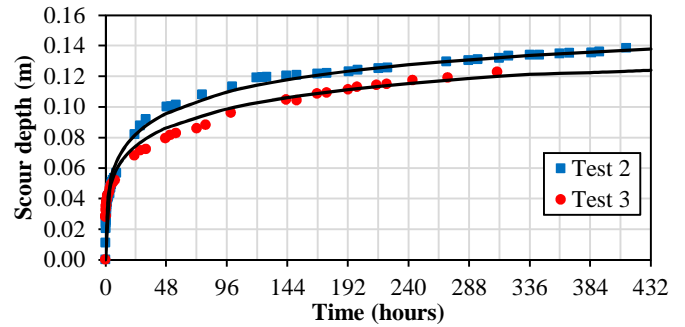


Figure 8. Scour depth evolution for situation 2

In the third situation, corresponding to the case where the pile cap is partially buried in the bed, the scour depth evolution does not follow a unique trend, as identified in the situations 1 and 2; in this case, the scour depth evolution presents different curvature changes that depend on the position of the pile cap, as shown in Figure 9. In the three tests covering this situation, the scour process began in front of the pile cap and measurements were made at point 2.

In test 4 (see Fig. 5), after 240 hours, the maximum scour depth location changed to the front of the underneath upstream piles and the measuring point changed to point 3 (Fig. 6). Test 5 and test 6 lasted 576 hours and 672 hours, respectively. Although these tests were run for such a long time, the scour hole did not reach the piles. The scour process began very slowly in test 6 ($H_c = 0.01\text{m}$) due to the small height of the pile cap that remained above the bed level, leading to a reduced downflow scouring contribution for a long time. In spite of the extremely long duration of test 6, it seems obvious that equilibrium was far from reached.

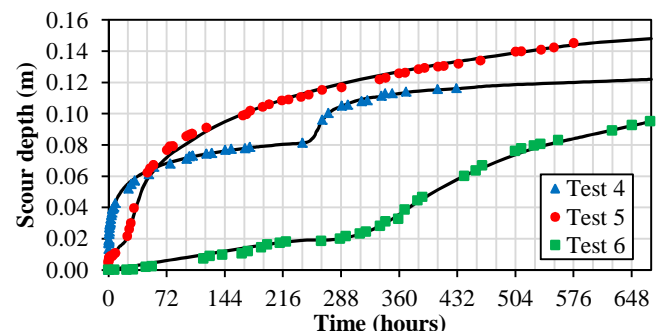


Figure 9. Scour depth evolution for situation 3

In situation 4 (tests 7, 8 and 9), characterized by the fact that the pile cap is completely buried in the bed, the scour depth records also evolve differently in time, as show in Figure 10.

In test 7, that was initiated when the top of the pile cap was levelled with the sand bed, the scour process began very slowly in front of the pile cap, scouring being absent downstream, along the edge of the pile cap upper face. The scour depth measurements were carried out at point 2 (Fig. 6). Equilibrium was not reached either, after 647 hours.

In test 8, the scour process began in front of the column until the scour hole partly uncovered the top of the pile cap (after approximately 48 hours); during this period, measurements were made at point 1 (Fig. 6). The scour depth did not evolve during approximately 410 hours, after which the scour process resumed in front of the pile cap; measurements were then made at point 2 and there seems to be a reasonable approximation to equilibrium.

In test 9, which is a particular situation of the buried pile cap, the scour process was only influenced by the column since the maximum scour depth was smaller than the distance of the original bed and the top of the pile cap. In this case, the scour depth evolution was similar to the evolution at single, piers as shown in Figure 10.

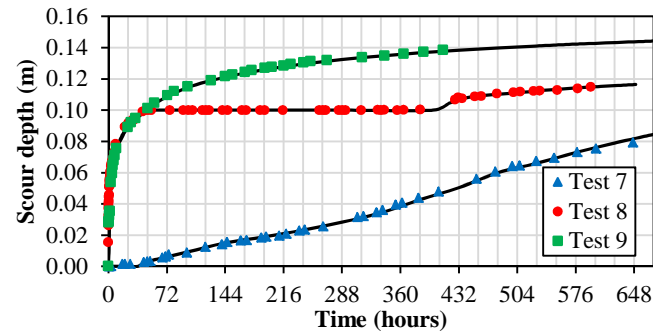


Figure 10. Scour depth evolution for situation 4

5.2 Maximum scour depth

Table 1 summarizes the final scour depths of the nine tests reported (Figs 7-10). The pile cap top elevation, H_c , and the final scour depth, d_s , were normalized by the flow depth, h . Figure 11 shows the variation of the normalized scour depth (horizontal axis) with the normalized pile cap elevation (vertical axis).

For the adopted complex pier geometry, the maximum scour depth occurred when the pile cap was partially buried in the bed, $H_c/h = 0.18$ as shown in Figure 11. In this situation, the pile cap thickness obstructing the flow is capable of generating significant downflow and, therefore, important scouring around the pile cap.

Table 1. Maximum scour depth at tests duration

Test	H_c (m)	Time (h)	d_s (m)	H_c/h (-)	d_s/h (-)
1	0.300	310.0	0.114	1.50	0.570
2	0.200	412.7	0.139	1.00	0.693
3	0.133	310.6	0.123	0.67	0.614
4	0.067	351.4	0.113	0.33	0.565
5	0.037	576.6	0.145	0.18	0.725
6	0.010	671.1	0.095	0.05	0.476
7	0.000	646.5	0.079	0.00	0.396
8	-0.100	594.6	0.115	-0.50	0.574
9	-0.300	412.4	0.138	-1.50	0.690

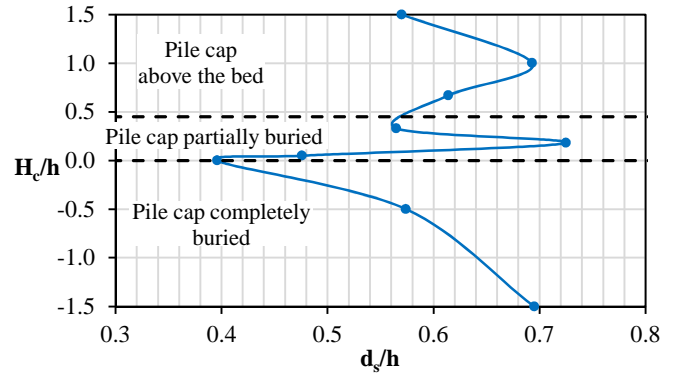


Figure 11. Scour depth variation as function of pile cap position

When the top of the pile cap was close to or at the bed level ($H_c/h = 0.05$ and $H_c/h = 0.0$) the scour process began very slowly as mentioned before. This is due to the combination of two factors: first, the small pile cap thickness that is in contact with the water column, possibly incapable of generating a significant downflow jet; second, the lateral and frontal extensions of the pile cap upper face out from the column perimeter that protects the bed from scouring. This observation is in line with observations of Ataie-Ashtiani et al. (2010) who reported that the pile cap extensions in both directions of the pier protect the bed from scour and postpone the beginning of the scour process.

5.3 Assessing the equilibrium scour depth

Regarding the time needed to reach equilibrium scour at complex piers, t_e , some authors (Coleman 2005, Melville et al. 2006, Ataie-Ashtiani et al. 2010) suggest the same criterion as Melville & Chiew (1999) for the time to equilibrium at single piers. This time is defined as the time required for the rate of scour to reduce to 5% of the smaller of the foundation length or the flow depth in a 24 hours period. The equilibrium scour depth corresponds to the scour depth measured at time to equilibrium.

Figure 12 shows the rate of scour depth evolution for tests 2, 4, 7 and 8, in which the equilibrium times

according to the mentioned criterion are 8.5, 5.0, 1.5 and 2.8 days, respectively.

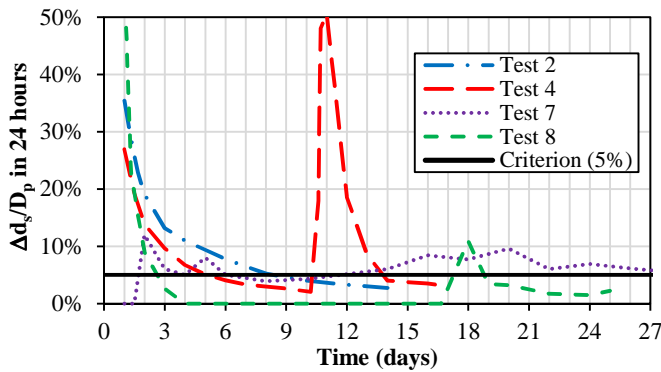


Figure 12. Rate of scour depth as function of time

In accordance with Figure 12, if, hypothetically, test 4 had finished immediately after 5 days (respecting the referred 5% criterion) the scour hole would not have reached the pile group. In this test, the scour process began in the upstream piles after 10 days, when the rate of scour depth evolution increased very rapidly. A similar result would have been obtained in test 8, in which the scour hole in front of pile cap began approximately after 17 days, while the 5% criterion would recommend the test to be stopped after 2.8 days.

Table 2 summarizes the equilibrium time with associated with criterion of 5% suggested by Melville & Chiew (1999) and the corresponding scour depths.

Table 2. Scour depth results with criterion of 5%

Test	H_c (m)	Time (h)	d_s (m)	H_c/h (-)	d_s/h (-)
1	0.300	216.0	0.109	1.50	0.545
2	0.200	204.0	0.125	1.00	0.625
3	0.133	202.0	0.113	0.67	0.565
4	0.067	120.0	0.074	0.33	0.370
5	0.037	348.0	0.123	0.18	0.615
6	0.010	36.0	0.001	0.05	0.001
7	0.000	36.0	0.002	0.00	0.001
8	-0.100	67.0	0.100	-0.50	0.500
9	-0.300	185.0	0.125	-1.50	0.625

Figure 13 compares the scour depth values recorded in Table 2 with the deepest scour values measured in the present study. The maximum difference is registered for the situations where the pile cap is partially or completely buried close or at the bed surface. This happens because, in the scour process, the three structural elements of the complex pier are involved while the 5% criterion was developed for single piers conditions. The criterion allows obtaining acceptable results when the pile cap is out of the water and only the pile group is exposed to the flow, behaving as a single element (see Fig. 13, points corresponding to $H_c/h = 1.5$).

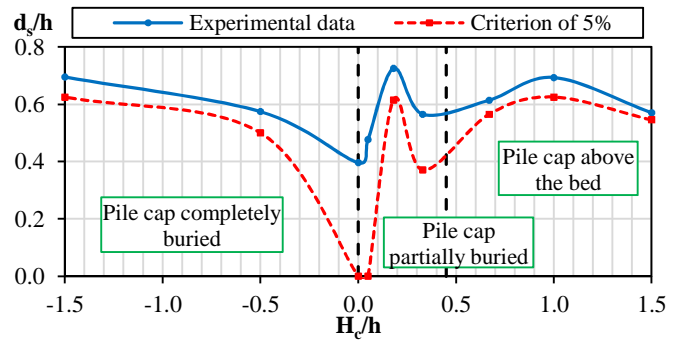


Figure 13. Scour depth comparison

According to the results of this study, it is evident that the 5% criterion may lead to tests durations that are much shorter than the minimum duration required to cover a significant part of the scouring process. In the particular case of the studied complex pier, the following approximate minimum duration were observed to be needed (i) 330 hours (14 days) when the pile cap was above the bed (situations 1 and 2), (ii) 430 hours (18 days) when the pile cap was partially buried in the bed (situation 3) and (iii) 500 hours (21 days) when the pile cap was completely buried in the bed (situation 4). In the latter case, test 9, where the scour depth is only influenced by the column, the minimum experiment duration of approximately 14 days seems appropriate, as for situations 1 and 2.

5.4 Applicability of existing scour depth predictors

The deepest measured scour depths were compared with the outputs of the three predictors described in the section 2. The results of the comparison are shown in Figure 14 as a function of the pile cap elevation. The axes in Figure 14 were normalized with the flow approach water depth.

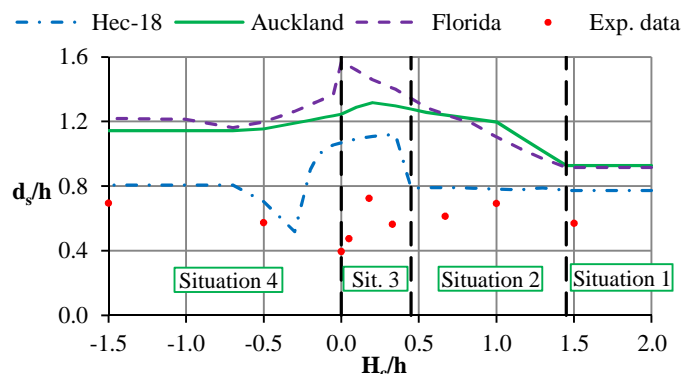


Figure 14. Scour depth variation as a function of the pile cap position

According to Figure 14, the three available methods predict safe scour depths. The HEC-18 method is the one that leads to the best predictions, differences being acceptable when the pile cap is both

completely buried in the bed (tests 8 and 9) or near the water surface (situations 1 and 2). The most important differences are observed for tests 5, 6 and 7, where, according to Figures 9-10, the scour process did not reach equilibrium and any technique to assess the scour depth at infinite time would render doubtful outputs. This lack of equilibrium may possibly explain the over-predictions of the HEC-18 method in spite of the fact that the present data come from experiments much longer than those behind the method.

In order to further evaluate the three predictors, the data obtained by Coleman 2005 (US pier) and Ataie-Ashtiani et al. 2010 (Model II) were added to the present data set; these cases were chosen because the tested complex piers were similar to the present model. Table 3 summarizes the values of the control variables and results of the studies selected literature data.

It should be noticed here that the sediment coarseness ratio of the piles, D_p/d_{50} , is, respectively, 24 and 26 in the studies by Coleman (2005) and Ataie-Ashtiani et al. (2010), while it is 58 in the present study. Likewise, the flow intensity, U/U_c , is of 0.74 to 0.83 in those studies and 0.93 in the present one. These differences as well as the long duration of present tests should, *a priori*, lead to deeper scour holes in the present study as compared with those of the literature.

Table 3. Summary of experimental conditions in complex piers tests in selected literature

Researchers	Coleman (2005)	Ataie-Ashtiani et al. (2010)
h (m)	0.330	0.140 – 0.154
d_{50} (mm)	0.84	0.60
U (m/s)	0.340	0.229 – 0.262
U_c (m/s)	0.409	0.303 – 0.336
U/U_c	0.83	0.74 – 0.80
D_c (m)	0.100	0.042
L_c (m)	0.400	0.150
D_{pc} (m)	0.190	0.090
L_{pc} (m)	0.480	0.190
T (m)	0.08	0.042
m	8	3
n	3	2
D_p (m)	0.020	0.016
S_{pm} (m)	0.060	0.065
S_{pn} (m)	0.065	0.048

Figure 15 establishes the comparison of data with the predictions obtained by the HEC-18 method (Richardson & Davis, 2001). The line of perfect agreement and the $\pm 20\%$ error margins are included to ease the comparison. It is clear that the method tends to overestimate the scour depth, the exceptions being some cases of Coleman's data. According to Figure 15, overestimation is more pronounced in the experiments of this study corresponding to the situation 3 pile cap partially buried, (tests 4-6) and test 7 (top of the pile cap at the bed level). This overesti-

mation can be explained by the fact that the HEC-18 method has intrinsically carries conceptual limitations for these pile cap elevations as mentioned above.

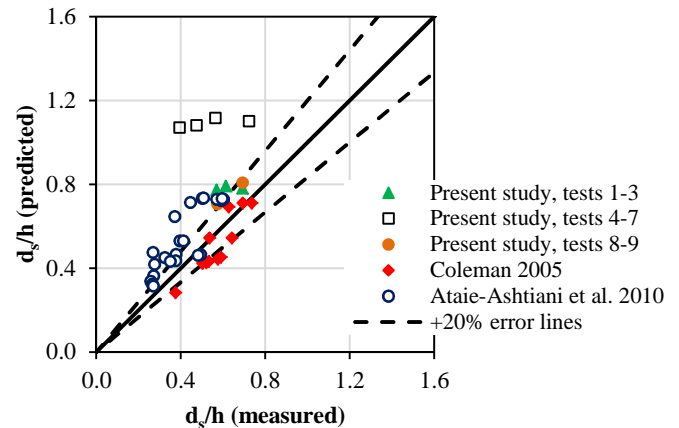


Figure 15. Comparison of the measured and predicted scour depths using HEC-18 method (Richardson & Davis 2001)

Figure 16 shows the similar comparison of the experimental data with the scour depths as predicted through the Auckland method (Coleman, 2005). The overestimation is more pronounced than in the case of HEC-18 especially in what concerns the present data (situations 2, 3 and 4). The increase of over-prediction comes to no surprise since the Auckland method was derived so as to guarantee safe predictions of Coleman's test cases. Overestimation in the situation 4, i.e., for the pile cap completely buried, (tests 7-8) can be explained by that the method was designed to predict higher scour depths in this particular case.

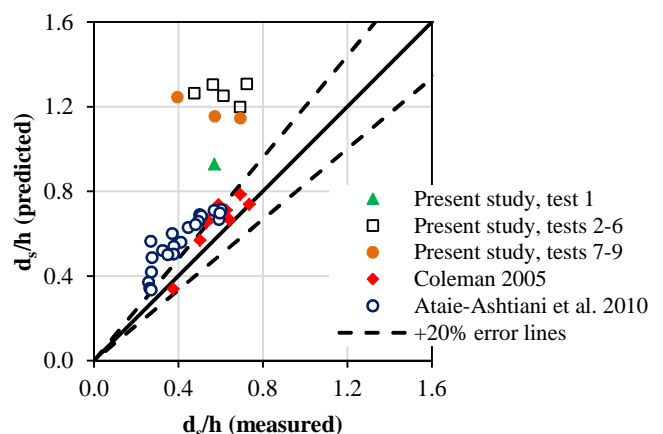


Figure 16. Comparison of the measured and predicted scour depths using Auckland method (Coleman 2005)

Figure 17 presents the comparison of measurements with the predictions obtained by the Florida method (Sheppard & Renna, 2010). Regarding the data of Coleman (2005) and the data of Ataie-Ashtiani et al. (2010) differences are similar to those

associated to the Auckland method. However, differences towards the data of the present study are even more evident, especially for the situation 3 pile cap partially buried, (tests 4-6) and test 7 (top of the pile cap at the bed level). As in the previous comparisons, over-predictions come to some surprise since the present experiments lasted much longer than those reported by other authors.

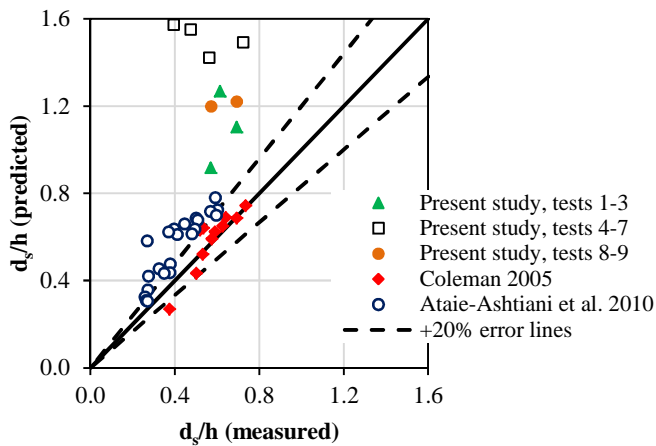


Figure 17. Comparison of the measured and predicted scour depths using Florida method (Sheppard & Renna 2010)

6 CONCLUSIONS

From the previous discussion, the following important conclusions can be drawn:

The equilibrium scour depth at complex piers is generally influenced by the pile cap elevation in relation to initial bed level. The maximum scour depth occurs when the pile cap is partially buried, $H_c/h=0.18$. The minimum scour depth occurs when the top of pile cap is near the bed, $H_c/h=0.0$, in which the scour process began very slowly and the scour depth evolution rate is smaller than for the others pile cap elevations.

It was confirmed that criterion established by Melville & Chiew (1999) for single piers to identify the equilibrium phase and stop the experiments does not apply to the case of complex piers, since it would imply test times much smaller than those required for the different scouring phases. Therefore, for experimental studies with models similar to the one of this study, test duration of, at least, two to three weeks should be guaranteed.

Applying three available predictors of the equilibrium scour depth to the nine situations tested in the present study, it became obvious that they lead to conservative estimates. Yet, particularly for this complex pier model, HEC-18 method leads to the best predictions, i.e., to the smaller overestimations.

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