

Experimental study on local scour around bridge piers in rivers

J. S. Antunes do Carmo

*IMAR – University of Coimbra, Department of Civil Engineering,
Portugal*

Abstract

Scour in the vicinity of structures may be caused by the combined effects of local, contraction and natural scour, and may also be aggravated by the effects of navigation. It may occur as a result of natural changes of flow in the channel, as part of longer-term morphological changes to the river, or through man's activities, such as building structures in the channel or dredging material from the bed. This paper presents a study on the scour process in a fluvial environment, in order to prevent a construction from failing. The experiments related to this research on scour in rivers and bridge failures were conducted in the Hydraulics Laboratory of the Civil Engineering Department of the University of Coimbra. We hope these results may be useful for testing and calibrating design methods for estimating equilibrium depths of local scour around bridge piers.

Keywords: local scour, piers in rivers, scour protection, experimental study.

1 Introduction

Structures built in rivers and estuaries are prone to scour around their foundations. If the depth of scour becomes significant, the foundations' stability may be endangered, with a consequent risk of the structure suffering damage or failure. CIRIA [1] refers to several cases of bridge failure due to scour, some causing loss of life and most resulting in significant transport disruption and economic loss. Richardson and Davies [2] cite several studies of bridge failures in the USA. In a report for the Transport and Road Research Laboratory, Wallingford et al. [3] listed some notable bridge failures due to scour. Melville and Coleman [4] cite 31 case studies of bridge failures and damage in New



Zealand due to scour since the 1930s, and state that “at least one serious bridge failure each year (on average) can be attributed to scour of the bridge foundations”. In March 2001 a major accident occurred in Portugal. Bridge failure led to the deaths of 59 people in the river Douro. A bus and three cars were on the bridge at the time of the accident. It has been found that the combined effects of unplanned, uncontrolled, continuous and increasing dredging of sand in the last few years, and the scour produced by five floods between December 2000 and March 2001 were the primary causes of this accident. These events and the associated material and human losses justify a thorough study of this phenomenon, particularly in the vicinity of structures built in rivers.

2 Failures due to hydraulic action

2.1 Scour failures

As CIRIA [1] makes clear, most rivers have beds and banks of potentially mobile material, and these may be subject to erosion during floods, leading to a fall in bed level or retreat of a bank. Scour in the vicinity of structures may be due to the combined effects of natural, contraction and local scour, and may also be worsened by the effects of navigation. A bridge constructed on spread footings is at risk of scour failure when the scour reaches the level of the base of the footing. However, it could be at risk with less scour if the substructure is subject to lateral ground pressure and water forces. Scour adjacent to piled foundations may result in a loss of skin friction and load-bearing capacity. The piles may also be subject to unanticipated bending stresses from lateral loads and hydrodynamic forces. As has been noted, local scour at a bridge pier is normally greatest near the upstream nose of the pier, which may lead to the pier first being undermined at the upstream end, and thus tilting. Similar effects may occur at abutments or with groups of piles. Variations in bed and flow conditions may lead to tilting in other directions.

2.2 Bank erosion and channel migration

Many rivers tend to change their course over time. A bridge or other structure that is sited to suit a particular location of the main channel may become progressively vulnerable to scour failure as the river attempts to migrate. Abutments or piers located on the original floodplain may be undermined or otherwise destabilized if not designed to accommodate channel migration. Changes in channel alignment can also be caused by poorly designed training works or by the uncoordinated construction of other structures upstream: examples of the latter include jetties and rock weirs, or groynes, built to provide improved conditions for fish and other fauna. Maintenance repairs by placing rock protection around bridge piers can reduce the flow area of the main span and divert the flow towards other channels or openings. Early warning of channel degradation problems at a structure requires the long-term monitoring of changes in the river for significant distances, both upstream and downstream.



2.3 Hydraulic forces on piers

Water flowing past a pier or abutment exerts forces that increase markedly as the velocity increases (by about the square of the velocity). In an ideal situation the forces are in line with the axis of the channel and piers, but in some cases channel movements and other factors may affect the flow direction and the resulting forces to a degree not anticipated by the design. The ability of a structure to withstand the hydraulic forces depends on the foundation design, and it may be compromised by the scour which occurs. Debris accumulation may also play a part.

3 Design methods

A wide variety of formulas are described in the literature. The required input data common to all the formulas can be summarized as follows: the flow conditions h and U for the design flood; the sediment properties expressed either as d_{50} and d_{90} or d_{max} ; pier size D or E and F (Figure 4 below), and pier shape and alignment. Factors which account for shape and alignment effects have been published by many researchers. A selection of shape and alignment effects is given in Melville and Sutherland [5], Dongol [6], or Melville [7]. Design methods can also be found in these references.

4 Experimental study

4.1 Description of the equipment

The experiments were conducted in a rectangular cross-section flume, 0.30 meters wide and 7.5 meters long, at IMAR (University of Coimbra) (Figure 1). The velocity and the discharge of the water flow were measured and calculated using an Acoustic Doppler Velocimeter (ADV) and a Moulinet.



Figure 1: Flume in which the experiments took place.

The ADV is an instrument for measuring the point velocity of water flow. With the use of appropriate software, the values of the velocity are gathered and stored in a computer. In these experiments a 2-dimensional side-looking probe was used for measuring water velocity in two directions, horizontally and perpendicularly (Figure 2). The Moulinet is a device for measuring the point velocity of the water flow. In contrast to the ADV, which uses sonar signals, the Moulinet's principle is based on the mechanical properties of the water. The device consists of a stem with a small propeller at the end, which is submerged in the water. The main body of the device is a processor that counts the propeller frequency and this number is displayed on a screen. The point velocity is calculated by means of a diagram which illustrates the relation between the propeller frequency and point velocity.

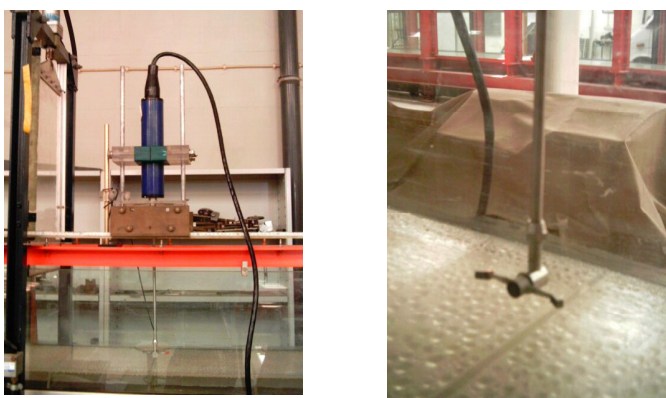


Figure 2: ADV equipment and its sensor.

4.2 Description of the experiments

A flat horizontal layer of sand was formed in the flume, 5 cm deep and 190 cm long. The piers were fixed to the bottom to simulate real piers of a bridge in a river. Different kinds of pier were used in terms of size, cross-section shape and number. The mean sand grain size was $d_{50} = 0.27 \text{ mm}$. When the sand was ready the flume was filled with water up to a depth of initial 8 cm.

The pump was turned on and the water flow started. The flow depth, and consequently the discharge, for each experiment could be modified by adjusting the pump's pressure. The experiment began when the water reached the desired depth and it was stabilized at that level. The experiment's duration, that is, the period during which scouring would take place, was measured and noted. In the meantime, point velocities were recorded to form the velocity profile. When scouring was practically finished the pump was turned off and the flume was emptied and allowed to drain so that the sand pit dimensions around the piers could be measured. The sand pit extension and the scour duration both depended on the flow discharge.

The ADV apparatus was used to take samples of the point velocity at five depths: 20%, 40%, 60%, 80% and 100% (surface) of the total flow depth. At each point approximately 200 samples of the velocity were taken: 10 sample values per second over 20 sec. Average point velocities were calculated for the five depths, and the *velocity profile curve* was drawn. Using appropriate software, the integral under the curve was calculated. Dividing this by the flow depth gave the mean water velocity. The discharge was determined by multiplying the mean velocity by the width and the flow depth.

In the same way, samples of the flow's point velocity were taken by the Moulinet at four points. The procedure for calculating the mean velocity was the same as for the ADV. The velocity profiles obtained were then compared. Whenever discrepancies were detected, other velocity measurements were taken. Figure 3 presents the velocity profiles obtained.

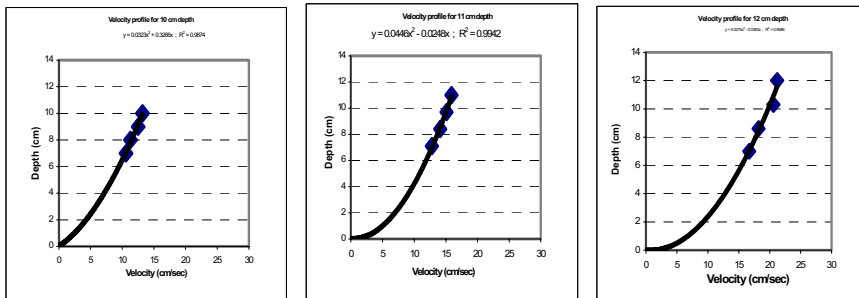


Figure 3: Velocity profiles for different depths.

The results of each experiment were recorded. The distance between the edge of the sand and the pile, and the depth of the sand at the four points A, B, L and R, upstream, downstream, left and right, were all measured, as shown in Figure 4 a) and b) below. The maximum scour and elevation of the sand, as well as the point at which they took place, were also recorded. Finally, the geometrical characteristics of the piers; the diameter D for the circular piers and the parameters E and F for the oval cross-section piers were noted.

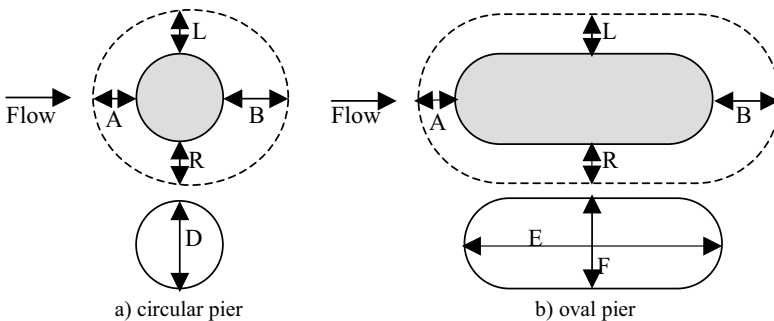


Figure 4: Types of pier and parameters.

In the experiments, piers with four kinds of cross-section shape were used: two were circular-shaped and two had an oval cross-section. The diameter of the larger cylindrical pier was $D = 4.0 \text{ cm}$ and the smaller one was $D = 2.0 \text{ cm}$. The E and F parameter dimensions of the oval-shaped piers were $E = 9.5 \text{ cm}$ and $F = 4.0 \text{ cm}$ for the bigger one and $E = 5.8 \text{ cm}$ and $F = 2.0 \text{ cm}$ for the smaller one (Figure 4). All pier models were made of metal. Two different flow depths were tested. Moreover, for the small piers, experiments were carried out using double piers to study the effects of having more than a single pier. Finally, our search for a solution to the problem of scouring in bridges led us to conduct other experiments, with two piers, one of which was protected. All in all, sixteen experiments were conducted in the course of this research.

4.3 Scour mechanism

Part of the flow approaching the pier is deflected downwards to the bed and rolls up to create what is often described as a “horseshoe vortex” around the front face of the structure; the vortex intensifies the local flow velocities and acts to erode sediment from the scour hole and transport it downstream. Vertical “wake” vortices caused by flow separation from the sides of the pier can also erode the bed, but, in fluvial conditions, the deepest scour tends to occur at the upstream face of the structure, as a result of the action of the “horseshoe vortex”. Material eroded from this hole is deposited towards the downstream end of the pier, to a level above that of the surrounding bed. The “wake” vortices are transported downstream by the flow and can create twin longitudinal scour holes; this type of scour may need to be considered if there is another structure farther downstream that is located within the “wake” created by the first pier.

As the scour develops, the increase in local flow depth reduces the strength of the erosive action at the bed; as a result, the rate of scour decreases and eventually reaches equilibrium. Clear-water scour occurs when the flow is no longer able to transport sediment particles out of the scour hole.

4.4 Results

Results of the experiments are presented in Tables 1 and 2. Table 1 concerns the single-pier experiments while Table 2 concerns the double-pier ones. Figure 5 is a picture taken after one of these experiments. The following expression seems to provide a good representation of the scour $h_s (m)$ (*maximum or equilibrium depth*):

$$h_s = 0.24 * \left(Q / \sqrt{d_{50}} \right)^{1/3} - h_0$$

where $d_{50} (m)$ is the mean sand grain size, Q is the flow ($m^3 s^{-1}$) and $h_0 (m)$ is the undisturbed approach flow depth.

5 Scour protection

5.1 Riprap

There are two basic approaches to protecting the bridge foundations (Chiew [8]):



1. To enhance the ability of the bed material to resist erosion. This is normally achieved by placing a protective layer of coarse granular materials or riprap at the base of the pier; and,
2. To reduce the power of the eroding agents, i.e., the downflow and “horseshoe vortex”, to erode the bed material, which is normally achieved by placing an extended base plate or collar at or around the bed level. Caissons and footing below the pier shaft can also effectively protect against scour at bridge piers.

Riprap is one of the most versatile and commonly used types of revetment, as it can usually be readily sourced, easily placed and can be specified to suit particular flow conditions. It is flexible and can accommodate small ground movements and some loss of stones without failure. Suitably sized riprap is appropriate for protecting against very high velocities and turbulence. It can be used to protect banks with slopes up to 1V:1.5H, without requiring additional restraint. Because of the flexibility in the shape of the area that can be covered, it is useful for protecting small awkwardly shaped areas and transitions between hydraulic structures and natural channels. Based on previous data and other studies and comparisons, Christine and Melville [9] proposed the following functional relationship for the minimum critical stone size:

$$d_{50} = 0.3 * K_D K_S K_a K_\gamma F^{1.2} * h_0$$

Table 1: Results of the single pier experiments.

Test No.	FLOW			PIER		LOCAL SCOUR										FINAL TIME		
	h (cm)	v (cm/sec)	Q _s (m ³ /sec)	Cross-section shape	Pier No.	Max depth h_s (cm)	Distances and Depths (cm)											
							A					B		R			L	
							Dist.	Depth	Dist.	Max. Height	Depth	Dist.	Depth	Dist.	Depth			
1	10.5	8.68	0.273x10 ⁻²	circular 4.0 cm	1	1.80	1.6	1.35	5.4	-0.70	-0.10	1.9	1.00	1.6	0.95	1h 45m		
2	11.0	9.78	0.323x10 ⁻²	circular 4.0 cm	1	3.00	4.0	2.20	6.2	-2.10	2.00	3.6	2.10	3.3	1.80	2h 10m		
3	10.5	8.68	0.273x10 ⁻²	circular 2.0 cm	1	1.90	2.3	1.60	4.0	-1.20	0.50	1.9	1.30	1.8	1.30	1h.50m		
4	11.0	9.78	0.323x10 ⁻²	circular 2.0 cm	1	2.40	2.9	1.70	4.8	-1.40	1.00	2.7	1.80	2.8	1.70	1h 55m		
5	10.9	9.56	0.313x10 ⁻²	oval F=4.0 cm E=9.5 cm	1	3.05	4.6	2.40	10.5	-1.90	-0.30	6.0	0.30	6.0	0.30	2h 00m		
6	11.2	10.51	0.353x10 ⁻²	oval F=4.0 cm E=9.5 cm	1	3.40	6.2	3.40	9.9	-2.10	-0.80	7.0	1.70	6.7	1.70	2h 50m		
7	11.0	9.78	0.323x10 ⁻²	oval F=2.0 cm E=5.8 cm	1	2.45	3.3	1.90	2.3	-1.30	-0.85	3.6	0.90	4.0	1.00	2h.35m		
8	11.5	11.61	0.401x10 ⁻²	oval F=2.0 cm E=5.8 cm	1	2.90	4.1	2.65	6.9	-1.80	0.50	4.6	2.00	4.6	2.10	3h 10m		

The negative numbers in Tables 1 and 2 indicate elevations of the riverbed in comparison to the initial conditions.



where $F^2 = U^2 / (gh_0)$, U the depth-averaged flow velocity; and the K -factors are dimensionless expressions describing the influence of each parameter (K_D for the pier diameter-to-bed material ratio, K_s and K_α for pier shape and alignment, respectively, and K_γ for placement depth). Each of the K -factors can be defined using experimental data (Christine and Melville [9]).

Table 2: Results of the double pier experiments.

Test No.	FLOW			PIER		LOCAL SCOUR										FINAL TIME
	h (cm)	v (cm/sec)	Q (m ³ /sec)	Cross-section shape	Pier No.	Max depth h_s (cm)	Distances and Depths (cm)									
							A		B		R		L			
							Dist.	Depth	Dist.	Max. Height	Depth	Dist.	Depth	Dist.	Depth	
9	10.9	9.56	0.313 x10 ⁻²	circular 2.0 cm	1	2.80	4.0	2.50	9.1	-2.10	1.80	3.9	2.35	3.7	2.25	2h 05m
				circular 2.0 cm	2	2.80	4.0	2.60	9.3	-2.00	1.80	3.9	2.20	3.8	2.30	
10	11.2	10.51	0.353 x10 ⁻²	circular 2.0 cm	1	3.15	4.9	2.90	12.6	-2.15	2.20	4.7	2.70	4.5	2.60	2h 20m
				circular 2.0 cm	2	3.20	5.0	2.90	12.3	-2.15	2.10	4.7	2.65	4.6	2.70	
11	11.0	9.78	0.323 x10 ⁻²	oval F=4.0 cm E=9.5 cm	1	2.65	3.9	2.40	8.0	-1.60	0.10	3.8	1.70	3.8	1.70	2h 00m
				oval F=4.0 cm E=9.5 cm	2	2.75	4.0	2.45	8.7	-2.00	0.10	4.0	1.80	4.0	1.80	
12	11.5	11.61	0.401 x10 ⁻²	oval F=2.0 cm E=5.8 cm	1	3.40	5.0	3.15	14.1	-2.30	1.50	5.3	2.30	5.3	2.40	2h 20m
				oval F=2.0 cm E=5.8 cm	2	3.30	4.8	3.00	14.2	-2.60	1.70	5.2	2.20	5.2	2.20	



Figure 5: Picture from an experiment with a double circular cross-section pier.

5.2 Preparation of the protection structure

The experiments on the protection pier were conducted with the double pier. Only one of the piers was protected by riprap so the results for the two piers can be compared. A hole was dug around the pier in which the protection was going to be placed. The hole dimensions were diameter $10.0\text{ cm} \leq D \leq 14.0\text{ cm}$ and depth $d = 1.0\text{ cm}$. The hole was then filled with granular material $d_{50} = 2.0\text{ mm}$ and $d_{90} = 4.0\text{ mm}$ in diameter (Figure 6).

5.3 Granulometric composition of the protection material

The material used for the protection was of a different dimension from that of the bed material. In the preparation of the riverbed, sand with $d_{50} = 0.27\text{ mm}$ diameter was used, and in the preparation of the protection the sand mean diameter was $d_{50} = 2.0\text{ mm}$. The granulometric curve of the protection material is presented below (Figure 6). Figure 7 shows a protected and unprotected double pier.

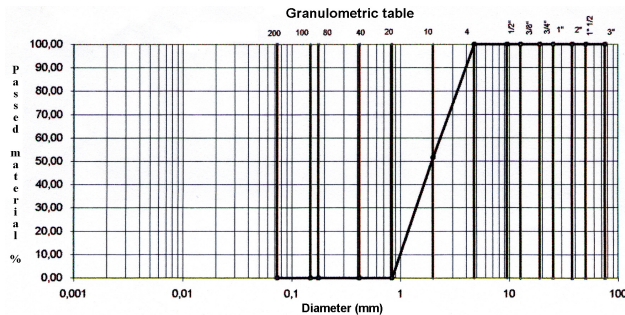


Figure 6: Granulometric curve of the protection material.

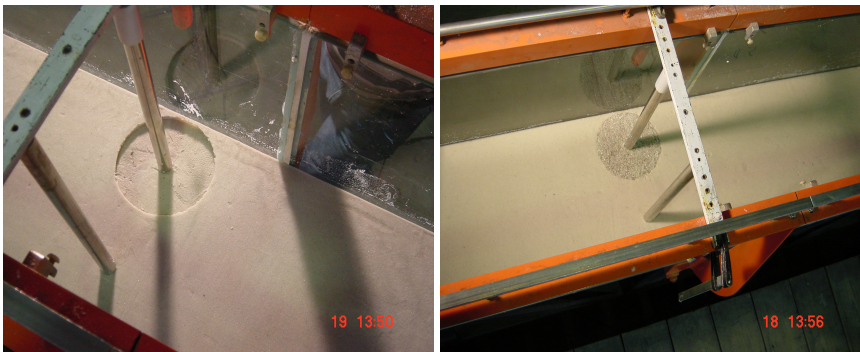


Figure 7: Protected and unprotected double circular cross-section pier.

5.4 Results

The results of the experiments are presented in Table 3. In every experiment the second pier is the protected one. Figure 8 presents a picture obtained after the

experiment had been running for 2 hours, with a mean velocity much greater than those presented in Table 3 (a destructive test).

The negative numbers in Table 3 indicate an elevation of the riverbed in comparison to the initial conditions.

Table 3: Results of the experiments with double piers and protection. The second pier in every experiment is the protected pier.

Test No.	FLOW			PIER		LOCAL SCOUR										FINAL TIME	
	h (cm)	v (cm/sec)	Q (m ³ /sec)	Cross-section shape	Pier No.	Max depth h_s (cm)	Distances and Depths (cm)										
							A		B			R		L			
							Dist.	Depth	Dist.	Max. Height	Depth	Dist.	Depth	Dist.	Depth		
13	11.3	10.94	0.371 x10 ⁻²	circular D=2.0 cm	1	3.3	4.5	2.80	9.0	-2.00	3.00	4.6	2.50	5.0	2.50	2h 10m	
				circular D=2.0 cm	2	-	-	-	-	-	-	-	-	-	-		
14	11.5	11.61	0.401 x10 ⁻²	circular D=2.0 cm	1	3.3	5.2	3.10	13.0	-2.50	2.30	5.3	2.90	5.2	2.90	2h 20m	
				circular D=2.0 cm	2	-	-	-	-	-	-	-	-	-	-		
15	11.5	11.61	0.401 x10 ⁻²	oval F=2.0 cm E=5.8 cm	1	3.2	5.0	2.80	10.5	-1.90	0.95	4.5	2.50	4.5	2.30	2h 15h	
				oval F=2.0 cm E=5.8 cm	2	-	-	-	-	-	-	-	-	-	-		
16	11.9	11.61	0.414 x10 ⁻²	oval F=2.0 cm E=5.8 cm	1	3.5	6.2	3.35	14.8	-2.10	2.00	5.7	2.80	5.0	2.50	2h 20m	
				oval F=2.0 cm E=5.8 cm	2	-	-	-	-	-	-	-	-	-	-		



Figure 8: Picture obtained after an experiment with two circular cross-section piers, one of them with protection (destructive test).

6 Conclusions

This paper has described the results of an experimental study on scour around bridge piers in rivers. We hope the data presented can be utilized to test and

calibrate appropriate design methods for estimating equilibrium depths and the extent of the sand pits developed around bridge piers, as a consequence of the hydraulic action. The reduced dimensions of the flume may mean, however, that some important problems might arise which have not been considered. In particular, the scale effects could prove to be important and may have to be taken into account.

Acknowledgment

This work has been carried out under the Project POCTI/ECM/2503/2001, financed by FCT – Fundação para a Ciência e a Tecnologia, Portugal.

References

- [1] CIRIA, 2002 (May, R, Ackers, J & Kirby, A). *Manual on scour at bridges and other hydraulic structures*. Construction Industry Research and Information Association.
- [2] Richardson, E V & S R Davies, 1995. *Evaluating scour at bridges*, 3rd edition. Hydraulic Engineering Circular No 18. Publication No FHWA IP-90-017. US Department of Transportation. Federal Highway Administration (Washington).
- [3] HR Wallingford and T P O'Sullivan & Partners, 1991. *Hydraulics of highway structures*. Report EX2404 Part 1. Contract Rep. for Transport and Road Research Lab.
- [4] Melville, B.W. & Coleman, S E, 2000. *Bridge scour*. Water Resources Publications. LLC (Colorado).
- [5] Melville, B W & A J Sutherland, 1988. Design method for local scour at bridge piers. *Journal of Hydraulic Engineering*, Vol. 114, n° 10.
- [6] Dongol, D.M.S., 1994. *Local scour at bridge abutments*. PhD thesis. Report N° 544, School of Engineering, University of Auckland, New Zealand.
- [7] Melville, B.W., 1997. Bridge abutment scour: Integrated approach. *Journal of Hydraulic Engineering*, ASCE, Vol. 123, N° 2, 125-136.
- [8] Yee-Meng Chiew, 1992. Scour protection at bridge piers. *Journal of Hydraulic Engineering*, Vol. 118, n° 9.
- [9] Christine S L & B W Melville, 2001. Riprap protection at bridge piers. *Journal of Hydraulic Engineering*, Vol. 127, b° 5.

