Experimental and numerical analysis of erosion and sediment transport of flushing waves

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ABSTRACT
This paper presents an investigation on the scouring effects of flushing waves on sewer sediment deposits. The investigation concerns both laboratory experiments and numerical modelling.

The experimental campaign is carried out at the laboratory of the Department of Hydraulic and Environmental Engineering of the University of Pavia. The first set of experiments is developed without sediment and the water levels during the flushing processes are monitored by a digital video-camera. The next tests are carried out with uniform layers of sediment (sand or sand and clay mixtures) at the bottom of the channel and the effects on sediment beds produced by flushes are analysed.

The numerical model solves the De Saint Venant equations describing unsteady flow in open channel and the continuity equation for the conservation of the sediment mass. For this purpose, the MacCormack explicit finite difference scheme is introduced. A formula that links the rate of erosion to an excess of shear stress at the bottom is included in the model.

The computed results are compared with the experimental data obtained on the laboratory channel.

KEYWORDS
Laboratory flushing waves; sand and clay mixtures; sediment erosion and transport model

INTRODUCTION
The accumulation of deposits inside sewer systems causes both hydraulic and environmental problems, such as the reduction of the flow capacity and stronger first flush pollution discharged through CSOs into water bodies (Arthur and Ashley, 1997; 1998; Fan et al., 2003). Also deposits at the bottom of stormwater storage tanks are critical being that they cause sanitary problems and odours.

The periodical removal of deposits is necessary in order to reduce these problems. For this purpose, several techniques based on the use of hydraulic flushing devices have been recently set up (Chebbo et al, 1996; Lorenzen et al., 1996; Pisano et al., 2003; Fan et al., 2003; Bertrand-Krajewski et al., 2003, 2004; Guo et al., 2004).

Some experimental studies have investigated the scouring effects produced by flushes in laboratory channels and in sewer collectors (Lorenzen et al., 1996; Linehan, 2001; Tait et al., 2003; Guo et al., 2004).
Also numerical models for erosion and sediment transport of flushing waves have been developed (Lin and Guennec, 1996; Saiedi, 1997; Kassem and Chaudhry, 1998; Campisano et al., 2004).

In this paper an investigation of the scouring effects of flushing waves on sediment deposits is presented. The investigation concerns both laboratory experiments and numerical modelling (Todeschini, 2007).

LABORATORY EXPERIMENTS
The experimental system (Figure 1) is made up of a 5.5 m long laboratory channel with plexiglass bottom and side walls characterised by a Manning roughness coefficient equal to 0.0082 s m\(^{1/3}\). The channel has a rectangular cross section 0.1 m wide and 0.3 m deep, the longitudinal slope can be varied.

The entrance section was closed and a steel sluice gate was positioned 1 m from the upstream end being able to activate in-line storage (when closed) and to generate flushing waves propagating downstream (after the gate opening).

The first set of experiments is developed without sediment and the water levels during the flushing processes are monitored by a digital video-camera. Two different initial values (0.15 m and 0.25 m) for the water head upstream the gate are considered. The investigation considers four different slope values: 0.5-1-1.5-2%. The flushing waves are generated by the sudden opening - 0.1 m high - of the sluice gate.

The second set of experiments is carried out with uniform layers of sediment at the bottom of the channel. These experiments consider a water level upstream the gate equal to 0.25 m and a longitudinal slope of the channel equal to 0.02.

First tests are carried out with a cohesiveness sediment. Three different types of sand (type 1: 0.2-0.35 mm, \(d_{50}=0.25\) mm, bulk density 1520 kg/m\(^3\); type 2: 0.4-0.6 mm, \(d_{50}=0.48\) mm, bulk density 1530 kg/m\(^3\); type 3: 0.8-1.2 mm, \(d_{50}=0.90\) mm, bulk density 1560 kg/m\(^3\)) are used.

In further experiments different clay contents (montmorillonite: bulk density 1080 kg/m\(^3\)) are added to these three types of sand.

Table 1 summarises the sediment bed conditions of the tests.

Experiments are carried out both with a dry (\(D\)) and a wet (\(W\)) sediment; two different bed thicknesses (\(T_b\)) are considered (0.5 - 1 cm). Sediments are placed at the bottom of the channel and levelled with a steel plate. The consolidation time for the wet sediments is one night. An homogeneous mixed sediment is obtained mechanically mixing dry sand and clay: percentages from 0 to 25% clay by weight are added to sand. The sediment porosity (\(p\)) is evaluated with a laboratory test. The values of Manning roughness coefficient at the bottom
(n_b) due to the three sandy beds are obtained by preliminary experiments in steady state conditions. A same roughness coefficient is used for a mixed sediment of a certain type of sand assuming that clay content doesn’t change significantly bed roughness. For the flushing experiments, a basket equipped with a synthetic cloth is inserted into the tank at the downstream end of the channel in order to intercept the scoured sand.

Table 1. Sediment bed conditions of the experiments.

<table>
<thead>
<tr>
<th>Sand type</th>
<th>T_b [cm]</th>
<th>Sediment condition</th>
<th>n_b [s m⁻¹/³]</th>
<th>p</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand type I</td>
<td>0.5</td>
<td>W/D</td>
<td>0.0143</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>W/D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand type II</td>
<td>0.5</td>
<td>W/D</td>
<td>0.0154</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>W/D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand type III</td>
<td>0.5</td>
<td>W/D</td>
<td>0.0175</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>W/D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand type I - clay 7.5%</td>
<td>0.5</td>
<td>W</td>
<td>0.0143</td>
<td>0.40</td>
</tr>
<tr>
<td>Sand type II - clay 7.5%</td>
<td>0.5</td>
<td>W</td>
<td>0.0154</td>
<td>0.40</td>
</tr>
<tr>
<td>Sand type III - clay 7.5%</td>
<td>0.5</td>
<td>W</td>
<td>0.0175</td>
<td>0.39</td>
</tr>
<tr>
<td>Sand type I - clay 15%</td>
<td>0.5</td>
<td>W</td>
<td>0.0143</td>
<td>0.39</td>
</tr>
<tr>
<td>Sand type II - clay 15%</td>
<td>0.5</td>
<td>W</td>
<td>0.0154</td>
<td>0.39</td>
</tr>
<tr>
<td>Sand type III - clay 15%</td>
<td>0.5</td>
<td>W</td>
<td>0.0175</td>
<td>0.39</td>
</tr>
<tr>
<td>Sand type I - clay 20%</td>
<td>0.5</td>
<td>W</td>
<td>0.0143</td>
<td>0.38</td>
</tr>
<tr>
<td>Sand type II - clay 20%</td>
<td>0.5</td>
<td>W</td>
<td>0.0154</td>
<td>0.38</td>
</tr>
<tr>
<td>Sand type III - clay 20%</td>
<td>0.5</td>
<td>W</td>
<td>0.0175</td>
<td>0.38</td>
</tr>
<tr>
<td>Sand type I - clay 25%</td>
<td>0.5</td>
<td>W</td>
<td>0.0143</td>
<td>0.38</td>
</tr>
<tr>
<td>Sand type II - clay 25%</td>
<td>0.5</td>
<td>W</td>
<td>0.0154</td>
<td>0.38</td>
</tr>
<tr>
<td>Sand type III - clay 25%</td>
<td>0.5</td>
<td>W</td>
<td>0.0175</td>
<td>0.37</td>
</tr>
</tbody>
</table>

MATHEMATICAL MODEL

The numerical model solves the De Saint Venant equations describing unsteady flow in open channel (1) (2) and the continuity equation for the conservation of the sediment mass (3). The sediment erosion relationship of Parchure and Metha (1985) (4) is included in the computations. This formula links the rate of erosion to an excess of bed shear stress and has some successful applications for mixed cohesive/non-cohesive sediments (Torfs, 1995; Skipworth, 1996).

- **Continuity equation for water:**
  \[
  \frac{\partial h}{\partial t} + \frac{\partial q}{\partial x} = 0
  \]  (1)

- **Momentum equation for water:**
  \[
  \frac{\partial q}{\partial t} + \frac{\partial}{\partial x} \left( \frac{q^2}{h} + \frac{gh^3}{2} \right) + gh \frac{\partial z}{\partial x} + ghS_f = 0
  \]  (2)

- **Continuity equation for sediment:**
  \[
  \frac{\partial}{\partial t} \left( (1 - p)c + \frac{q_z h}{q} \right) + \frac{\partial q_z}{\partial x} = 0
  \]  (3)
Evolution of the bed elevation and sediment erosion relationship of Parchure and Metha (1985):

$$\frac{\partial z}{\partial t} = \frac{\varepsilon}{\rho_s}$$

$$\varepsilon = \varepsilon_m\left(\frac{\tau_b - \tau_{cr.}}{\tau_{cr.}}\right)^\alpha$$

in which $h$ is the flow depth; $q$ the water discharge per unit width; $z$ the bed elevation; $q_s$ the unit sediment discharge; $S_f$ the friction slope; $x$ the distance along the channel; $t$ the time; $p$ the porosity of the bed layer; $z_0$ the initial bed elevation; $\varepsilon$ the rate of erosion; $\rho_s$ the sediment density; $\varepsilon_m$ and $\alpha$ two parameters; $\tau_b$ the bottom shear stress; $\tau_{cr.}$ the critical bottom shear stress.

The friction slope $S_f$ is evaluated using the Chézy - Manning relation:

$$S_f = \frac{q^2n^2}{h^2R^3}$$

with $R$ the hydraulic radius and $n$ the Manning roughness coefficient which is evaluated by the Einstein relation, taking into account different roughness values on the wetted perimeter $P$:

$$n = \left(\frac{P_{sw}n_{sw}^{3/2} + P_bn_b^{3/2}}{P}\right)^{2/3}$$

being $sw$ and $b$ relative to the channel side walls and to the channel bottom.

The bottom shear stress $\tau_b$ is obtained by the following expression:

$$\tau_b = \rho_w g S_f \left(\frac{n_b}{n}\right)^{1/2}$$

in which $\rho_w$ is the water density.

The flushing process produces both bed load and suspension load transport but the model doesn’t distinguish between them. Moreover, this model doesn’t include loading law equations for the lag between sediment transport capacity and sediment discharge (hypothesis of immediate adaptation), being the transport essentially a bed load transport.

Equations (1) (2) (3) are numerically solved using the second order MacCormack scheme (Bhallamudi e Chaudhry, 1991). An additional step based on the theory of the total variation diminishing (TVD) is also applied to the hydraulic variables in order to reduce the numerical oscillations generated in correspondence to high gradients in water levels and flow rates (Sweby, 1984; Garcia-Navarro and Saviron, 1992).

Boundary conditions for the hydraulic variables $q$ and $h$ are imposed by applying the theory of characteristics (Abbot e Minns, 1998): both the variables are imposed at the upstream end for supercritical flow; the variable $q$ is imposed at the upstream end and a relation between $h$ and $q$ (Froude number equal to 1) is imposed at the downstream end for subcritical flow.

A boundary condition for the sediment is imposed at the upstream end for subcritical flow (variable $q_s$) and one at the downstream end for supercritical flow (variable $z$). The continuity
equation for sediment furnishes the variable not imposed (Cao et al., 2002).

**NUMERICAL AND EXPERIMENTAL RESULTS**

**Hydraulic validation of the code**

The numerical code was first applied to the set of experiments without sediment. The initial condition is the sluice gate in the closed position with water levels in the storage upstream the gate equal to 0.15 m (flush A) and to 0.25 m (flush B). Different longitudinal slopes $S$ are considered: 0.5-1-1.5-2%

A spatial step $\Delta x$ of 0.1 m was used for the numerical integration while the time steps $\Delta t_k$ were determined imposing a *Courant-Friedrichs-Lewy* number (CFL) equal to 0.9 for the scheme stability.

Figure 2 shows the good agreement between computed and experimental water depths for flush A - $S$ equal to 0.01 and for flush B - $S$ equal to 0.02 (cell 7 is 0.4 m upstream the gate, while cells 12 - 22 - 47 are respectively 0.1 - 1.1 - 3.6 m downstream the gate).

Referring to flush B and to a longitudinal slope of 0.02 Figure 3 shows a comparison between numerical and experimental water levels in each cell after a fixed time from the opening of the sluice gate. Experimental water depths are very close to those obtained with the code both after 2 and 4 seconds from the gate opening.

**Calibration of the erosion and transport model**

The numerical code can model the evolution of the sediment bed during the flushing operations. However, the continuative monitoring of the erosion process is impossible being the initial sediment thickness very low. For this reason, only the global effects of flushing waves produced by the opening of the gate are studied.

**Experiments with sand**

Table 2 summarises the experimental results for different initial conditions in terms of washed sediment volume ($V_{s,w}$) and washed sediment percentage ($%_w$).

The washed sediment volume is obtained dividing the weight of the washed sediment ($W_{s,w}$) once drained in the synthetic cloth by the bulk density of such a wet sediment ($\rho_{\text{wet}}$) obtained with a laboratory test.

The washed sediment percentage is obtained by the following formula:

$$%_w = \frac{W_{s,w}}{W_{s,i}} \cdot \frac{\rho_{\text{wet}}}{\rho_{\text{dry}}} \cdot 100$$

In which $\rho_{\text{dry}}$ is the bulk density of dry sediment and $W_{s,i}$ the initial weight of the dry sediment placed at the bottom of the channel.

The critical bottom shear stress $\tau_{cr}$ comes from the *Shields* formula (Yalin and Karahan, 1979). An higher value of critical shear stress is adopted for wet sand (instead of that assumed for the same type of dry sand) in order to take into account the major compactness of a wet sediment.

The parameter $\alpha$ is assumed equal to 1.
**Flush A S=0.01**

Cell 7

**Flush B S=0.02**

Cell 7

Cell 12

Cell 22

Cell 47

---

**Figure 2.** Comparison between measured and simulated water depths.

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**Figure 3.** Comparison between measured and simulated water depths after a fixed time from the opening of the sluice gate (flush B S=0.02).
Table 2. Results of the experiments with sand.

<table>
<thead>
<tr>
<th></th>
<th>$T_b$ [cm]</th>
<th>$D/W$</th>
<th>$V_{s,w}$ [dm$^3$]</th>
<th>$%_w$</th>
<th>$\tau_{cr}$ [N/m$^2$]</th>
<th>$k_{er}$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand type I</td>
<td>0.5 D</td>
<td>1.42</td>
<td>61.9</td>
<td>0.40</td>
<td>5.0 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5 W</td>
<td>1.28</td>
<td>56.6</td>
<td>0.43</td>
<td>4.9 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0 D</td>
<td>1.46</td>
<td>31.8</td>
<td>0.40</td>
<td>4.9 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0 W</td>
<td>1.34</td>
<td>29.6</td>
<td>0.43</td>
<td>5.1 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5 D</td>
<td>0.97</td>
<td>43.4</td>
<td>0.60</td>
<td>5.0 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5 W</td>
<td>0.90</td>
<td>39.7</td>
<td>0.65</td>
<td>5.0 $10^3$</td>
<td></td>
</tr>
<tr>
<td>Sand type II</td>
<td>1.0 D</td>
<td>1.03</td>
<td>23.0</td>
<td>0.60</td>
<td>5.2 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0 W</td>
<td>0.94</td>
<td>20.7</td>
<td>0.65</td>
<td>5.2 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5 D</td>
<td>0.80</td>
<td>35.3</td>
<td>0.80</td>
<td>5.0 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5 W</td>
<td>0.75</td>
<td>33.2</td>
<td>0.85</td>
<td>5.1 $10^3$</td>
<td></td>
</tr>
<tr>
<td>Sand type III</td>
<td>1.0 D</td>
<td>0.85</td>
<td>18.7</td>
<td>0.80</td>
<td>5.3 $10^3$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0 W</td>
<td>0.77</td>
<td>17.0</td>
<td>0.85</td>
<td>5.2 $10^3$</td>
<td></td>
</tr>
</tbody>
</table>

The parameter $k_{cr} = \varepsilon_m / \rho_s$ is evaluated for each test imposing the global continuity of the sediment mass at the downstream end of the channel:

$$\sum_{k=1}^{K-1} \left( \frac{(q_{i,N+1})_{k}^i + (q_{i,N+1})_{k+1}^i}{2} \right) \Delta t_k = V_{s,w} (1 - p)$$

(9)

in which $k$ is the time index, $K$ is the time index indicating the end of flush in the last spatial cell (cell $N+1$), $\Delta t_k$ is the time interval between the instants denoted by $k$ and $k+1$ and $p$ the porosity of the washed sand used in the experiment.

The left term of equation (9) comes from the numerical simulation while the right one is furnished by the flushing experiment.

It is important to notice that before the calibration procedure a preliminary congruity test of the numerical model is carried out. The numerical results satisfy the global continuity of the sediment mass being that the left term of equation (9) is equal to the scoured sediment volume evaluated from the numerical values of the bed elevation at the beginning and at the end of the flushing process:

$$\sum_{k=1}^{K-1} \left( \frac{(q_{i,N+1})_{k}^i + (q_{i,N+1})_{k+1}^i}{2} \right) \Delta t_k = \sum_{i=1}^{N} \int_{z_{i}^0}^{z_{i+1}^0} \left( \frac{z_{i+1}^K + z_{i+1}^K}{2} \right) \Delta x \left( 1 - p \right)$$

(10)

in which $i$ is the space index and cell $i$ equal to 11 corresponds to the gate, $z_{i}^0$ is bed elevation for cell $i$ at the beginning of the flush and $z_{i}^K$ is bed elevation for cell $i$ at the end of the flush.

For the same type of sand and equal thickness, dry sand has higher volume and percentage scoured than wet sand being the wet sediment more compact and so more difficult to remove. As expected the percentage of the washed sand decreases as grain size of sand increases.

The parameter $k_{cr}$ is almost equal for all the experiments with sand supporting the adopted sediment erosion relationship.

Visual observations show that the sediment is essentially transported as bedload.

Experiments with sand and clay mixtures

Figure 4 shows the washed sediment weight ($W_{s,w}$) of the experiments carried out with sand and clay mixtures, while Table 3 furnishes the washed sediment volume ($V_{s,w}$) and the washed sediment percentage ($\%_w$) of these experiments.

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These values are obtained adopting the same procedure described in the previous paragraph. However, some modifications are necessary being that the synthetic cloth isn’t able to retain all the clay watered.

The weight of washed clay ($W_{c,w}$) is furnished by the following expression assuming that the washed clay percentage by weight and the washed sediment one are the same.

$$W_{c,w} = \%_w \cdot W_{c,i} \quad (10)$$

in which $W_{c,i}$ is the initial weight of the clay placed at the bottom of the channel. The weight of washed clay passing through the cloth is assumed equal to the total weight of the washed clay. This assumption comes from the visual observations of washed sediment in the synthetic cloth.

The adopted values of critical bottom shear stress $\tau_{cr}$ are those found in previous laboratory experiments with sand and clay mixtures: the experiments of Torfs (1995) for sand types 1 and 2 and those of Campisano and others (2006) for sand type 3.

As shown in Table 3, the critical shear stress increases significantly when clay is added to sand being that the particles of sand and clay develop binding forces that increase during the consolidation time.

The parameter $\alpha$ is assumed equal to 1 according with the findings of Torfs (1995).

![Figure 4. Washed sediment weight for different clay content.](image)

<table>
<thead>
<tr>
<th>$T_b$ [cm]</th>
<th>$D/W$</th>
<th>$V_{s,w}$ [dm$^3$]</th>
<th>$%_w$</th>
<th>$\tau_{cr}$ [N/m$^2$]</th>
<th>$k_w$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand type I - clay 7.5%</td>
<td>0.5</td>
<td>W</td>
<td>1.26</td>
<td>53.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Sand type II - clay 7.5%</td>
<td>0.5</td>
<td>W</td>
<td>0.91</td>
<td>38.7</td>
<td>1.0</td>
</tr>
<tr>
<td>Sand type III - clay 7.5%</td>
<td>0.5</td>
<td>W</td>
<td>0.72</td>
<td>30.5</td>
<td>1.3</td>
</tr>
<tr>
<td>Sand type I - clay 15%</td>
<td>0.5</td>
<td>W</td>
<td>1.21</td>
<td>49.0</td>
<td>0.78</td>
</tr>
<tr>
<td>Sand type II - clay 15%</td>
<td>0.5</td>
<td>W</td>
<td>0.90</td>
<td>36.0</td>
<td>1.05</td>
</tr>
<tr>
<td>Sand type III - clay 15%</td>
<td>0.5</td>
<td>W</td>
<td>0.70</td>
<td>27.9</td>
<td>1.4</td>
</tr>
<tr>
<td>Sand type I - clay 20%</td>
<td>0.5</td>
<td>W</td>
<td>0.66</td>
<td>25.4</td>
<td>1.4</td>
</tr>
<tr>
<td>Sand type II - clay 20%</td>
<td>0.5</td>
<td>W</td>
<td>0.63</td>
<td>24.0</td>
<td>1.6</td>
</tr>
<tr>
<td>Sand type III - clay 20%</td>
<td>0.5</td>
<td>W</td>
<td>0.54</td>
<td>20.4</td>
<td>1.9</td>
</tr>
<tr>
<td>Sand type I - clay 25%</td>
<td>0.5</td>
<td>W</td>
<td>0.54</td>
<td>20.2</td>
<td>1.65</td>
</tr>
<tr>
<td>Sand type II - clay 25%</td>
<td>0.5</td>
<td>W</td>
<td>0.51</td>
<td>18.9</td>
<td>1.8</td>
</tr>
<tr>
<td>Sand type III - clay 25%</td>
<td>0.5</td>
<td>W</td>
<td>0.49</td>
<td>18.0</td>
<td>2.1</td>
</tr>
</tbody>
</table>
The parameter \( k_{er} \) is evaluated for each test by the expression (9) taking into account the clay which is not retained by the synthetic cloth.

Sediment volume and percentage washed decrease as clay content increases for each type of sand. This result confirms other experimental studies carried out on cohesive/non-cohesive mixtures. Sand and clay mixtures develop binding forces between particles which are broken by higher shear stresses than those that characterise the start of the erosion process in a sandy sediment. For this reason the critical bottom shear stress increases with clay content.

The values of parameter \( k_{er} \) are equal in the experiments with a clay content of 20 and 25%. However, these values are major (almost double) than that of sand without clay. This is due to the fact that the erosion behaviour of a cohesive sediment (sand with a clay content of 20 and 25%) is different than that of a non-cohesive sediment (sand without clay). While the erosion of a non-cohesive sediment is a particle-by-particle erosion, the mode of erosion of a cohesive sediment is characterised by a rapid dislodgement of large pieces of sediment immediately removed by the flow when the shear strength of the material is exceeded (mass erosion). The material being detached from the bed is mostly transported as bedload.

The parameter \( k_{er} \) for mixtures with a clay content of 7.5% and 15% assumes an intermediate value between that of sand and that of sand with a clay content of 20 and 25%. The reason of this result is due to the intermediate erosive behaviour of mixtures with a clay content of 7.5% and 15% between a non-cohesive behaviour and a cohesive one. Also Torfs (1995) has found the transition between non-cohesive and cohesive behaviour for a montmorillonite content in a sandy bed of almost 7-13%.

**CONCLUSIONS**

An experimental and numerical investigation on the scouring effects of flushing waves on sediment deposits is presented.

The experiments were carried out at the laboratory of the Department of Hydraulic and Environmental Engineering of Pavia on different uniform layers of sediment.

A numerical code based on the solution of the De Saint Venant - Exner equations was developed adopting the TVD-McCormack scheme. The sediment erosion relationship of Parchure and Metha (1985) was implemented in the code in order to obtain the bed evolution during the flushing process.

The comparison between the numerical results and the laboratory measurements consents to calibrate the parameters describing the erosion process consequent to the flushing operations. Variations in the sediment composition affect the erosion process and the mode of erosion. The erosion process of a sediment bed occurs with increasing shear stresses as clay is added to sand but once this process is started the rate of erosion is major for a cohesive sediment (sand with a clay content of 20 and 25%) than for a non-cohesive one (sand without clay).

While the mode of erosion of a non-cohesive sediment is a particle-by-particle erosion, the erosion of a cohesive sediment is characterised by a rapid dislodgement of large pieces of sediment immediately removed by the flow when the shear strength of the material is exceeded.

Mixtures with a clay content of 7.5% and 15% show an intermediate erosive behaviour between a cohesiveness behaviour and a cohesive one.

Both for sand and for sand and clay mixtures the material being detached from the bed is mostly transported as bedload.

The results of this study can be adopted for deriving indications on the set-up and design of flushing devices in sewer systems and in stormwater storage tanks. However, further investigations are necessary in order to consider bed layers with different grain size distributions, cohesive characteristics and consolidation times.
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