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EXPERIMENTAL AND NUMERICAL SIMULATION OF SEDIMENT FLUSHING WITHIN STORAGE TUNNELS

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ABSTRACT

Many sewer systems in the UK are combined. These systems carry both foul and stormwater. In many of these sewer systems, it is necessary to store sewage temporarily, in structures such as storage tunnels, usually in order to prevent flooding during rainfall events. One consequence of storing sewage is that any sediment within the sewage can settle and form deposits. One method to manage these sediments is to generate flush waves using movable gates to erode any in-pipe deposits and to transport any sediment downstream to treatment. In this paper, a shallow water Finite Volume (FV) model based on the full St Venant equations was constructed and coupled with algorithms to estimate sediment erosion, deposition and movement within sewer systems structures, such as storage tunnels. A Monotone Upwind Scheme for Conservative Laws (MUSCL)-Hancock scheme together with a Harten Lax van Leer (HLL) Riemann solver was used to give stable and accurate discretization of the FV model throughout the domain as they can resolve discontinuities accurately. The simulation has been compared against experimental findings in the literature of sediment transport during dam break flow for validation. Various configurations of water and sediment depth have also been tested to study the performance of sediment flushing in a sewer storage tunnel in order to discover whether the flushing of sediment within such structures can be optimized.

Keywords: sewer sediments; storage; sediment flushing; shallow water flow; St Venant equations; Finite Volume (FV) model; MUSCL-Hancock scheme; HLL Riemann solver.

1. INTRODUCTION

Sediment deposition within storage tunnels has been recognised as a key operational issue. In smaller diameter structures such as sewer pipes, methods such as jetting, rodding and using moving erosive balls are techniques that have been used to remove sediment blockages and in-pipe deposits (Ashley et al., 2004). However these techniques are not practical in larger structures.

Cleaning of large diameter sewers and storage tunnels using artificially generated flush waves has been investigated as a method of sediment management. It involves the installation of gates that can be used to store water upstream that is released suddenly to generate flushing waves. These waves can entrain sediment deposits that can then be moved and the sediments collected further downstream by a sediment interception device or removed by manual operation. Flush cleaning experiments in large diameter sewers in Hannover have highlighted the ability of flushes in pipes to remove sediments over quite large distances (Lorenzen et al., 1996).

A number of advantages of flush cleaning as a method of sediment management are recognised; it is predominantly self-controlled and requires little maintenance, no fresh water is required and there are minimal energy requirements. Small scale experiments have been carried out in which a saturated sediment deposit was subjected to a series of repeated 'dam-break' flushing waves. These tests indicated that the weight of sediment scoured by each flush remained almost constant for each experiment, however, a greater head of water upstream of the gate and smaller grain sizes of sediment both resulted in a greater mass of sediment being moved (Campisano et al., 2004). This paper aims to describe the development of a numerical model able to simulate the movement of sediment in such situations. It will be used to investigate which parameters can be adjusted to maximise sediment removal and transport.

In situations in which there is a dominant streamwise free surface flow the St Venant approach is popular for simulating such water flow. The St Venant equations have been used in many different applications, in which there is unsteady flow over varying bed topography (Tseng, 2003), and flow with a mobile bed (Wu and Wang, 2008). In such applications, the use of traditional three dimensional numerical schemes is usually inefficient and computationally very costly and so impractical to be used in engineering situations (see Mingham and Causon, 2000, and Hu et al., 2006).

To simulate sediment flushing in an unsteady flow, the sediment continuity approach is usually used. It is used for a wide range of simulation purposes, e.g. alluvial aggradation-degradation flow simulation by Kassem and Chaudhry (1998), and the dam-break sediment transport flow simulation by Capart and Young (1998). This equation utilises the volumetric transport rate of sediment to determine the temporal and spatial changes in bed elevation . Armanini and Di Silvio (1998) suggested the sediment continuity-concentration equations, and this improved the sediment transport representation by linking directly the exchange effect of the sediment bed and suspended loads with the deposit. In this paper, the sediment continuity-concentration equations are used to simulate sediment flushing. It is combined and coupled with the St Venant equations to simulate unsteady dam break flow under sediment laden conditions.

2. MATHEMATICAL MODEL

A depth-averaged mathematical model used in this study resolves the streamwise variation of velocity and water depth, and calculates the sediment depth and concentration. The flow model is based on the assumption of hydrostatic pressure distribution. For a 1-D analysis the equations of motion can be reduced to the following form:

$$\frac{\partial \mathbf{U}}{\partial t}+\frac{\partial \mathbf{F}}{\partial x}=\mathbf{\Omega}$$

$$\text{where, } \mathbf{U}=\begin{bmatrix} h \\ hu \\ hC \end{bmatrix}, \ \mathbf{F}=\begin{bmatrix} hu \\ hu^2+\frac{1}{2}gh^2 \\ huC \end{bmatrix}, \ \text{and } \mathbf{\Omega}=\begin{bmatrix} (E_b-D_b)/(1-p') \\ gh\left(S_{ox}-S_{fx}\right) \\ E_b-D_b \end{bmatrix} \tag{1}$$

And, the deposition (erosion) rate of the mobile bed reduces to (Valiani et al., 2001)

$$(1-p')\frac{\partial z_b}{\partial t}=D_b-E_b \tag{2}$$

In equations (1) – (2), h and u are the water depth and streamwise velocity respectively; whereas, x and t denote the streamwise space and time respectively. z_b is the sediment bed level; g is the gravitational acceleration; p' is the porosity of the sediment deposit; C is the flow vertical-averaged concentration [presented in equation (4)]; and, S_{ox} and S_{fx} are the bed and friction gradient in the streamwise direction respectively. In the source terms of concentration and sediment depth in equation (2), the relation of sediment entrainment rate, E_b, and deposition rate, D_b, can be determined by the following empirical formula (Valiani et al., 2001):

$$E_b - D_b = \frac{w}{\Lambda}\left(C_s - C\right) \tag{3}$$

$$C = \frac{V_s}{V_f + V_s} \tag{4}$$

In equations (3) – (4), w is the sediment falling velocity; V_s is the volume of sediment; and V_f is the volume of water flow. The non-dimensional adaptation length scale for the sediment, Λ, in equation (3) can be represented in a form as below (Armanini and Di Silvio, 1998)

$$\Lambda = \frac{a_1}{h} + \left(1 - \frac{a_1}{h}\right)\exp\left[-1.5\left(\frac{a_1}{h}\right)^{-1/6}\frac{w}{U_s}\right] \tag{5}$$

where, a_1 is a reference level to compare the flow depth with sediment grain size ($a_1 = 2d$ is used in this paper by referring to Valiani et al., 2001); d is median size of the sediment; and U_s is shear stress/friction velocity. In equation (3), C_s which is the equilibrium sediment concentration, and can be expressed as

$$C_s = \alpha F_s^{\beta} \tag{6}$$

where, α and β are empirical constants, and F_s in equation (6) is shown as follows

$$F_s = \sqrt{\frac{u^2}{\left(S_g - 1\right)gd}} \tag{7}$$

which, S_g = specific gravity.

3. NUMERICAL SCHEME

In this study, the FV method was chosen because of its robust nature in representing shock-wave flow accurately (Mingham and Causon, 2000, and Sanders, 2001). In the inviscid flux modelling of the proposed FV model, the Godunov-type Hancock scheme is used with a 2-stage predictor-corrector temporal discretization. The Godunov-type Hancock scheme is coupled with Harten Lax van Leer (HLL) approximate Riemann solver for upwinding volumetric discretization. The slope limiter method is used in the HLL solver to ensure the spatial discretization scheme satisfies a flux-limiting property (Mingham and Causon, 2000). MUSCL (Monotone Upwind Scheme for Conservation Laws) scheme is used to update the variables in spatially. The MUSCL and Hancock schemes keep the proposed FV model at a second order of accuracy in spatial and temporal domains respectively. The overall conservative solution of this FV model can be indicated by

$$\frac{\partial}{\partial t}\int_A \mathbf{U}dA + \oint_S \mathbf{F}.ds = \int_A \mathbf{\Omega}dA \tag{7}$$

where, A is the time-dependent area enclosed by the control surface S. For time stepping of this FV model, the Courant-Friedrichs-Lewy criterion is followed to ensure the utilised time step does not exceed its maximum allowable limit. In the source term, Ω, a first order derivative is used for its discretization. No complex discretization method is needed as HLL-type scheme is having competence to capture source term solution accurately (Hu et al., 2006).

4. RESULTS AND DISCUSSIONS

A dam-break flow is a well-known discontinuous flow event with a highly turbulent nature. Most of the numerical studies on such events tend to ignore the effect of sediment transport. In this paper, two tests have been constructed to investigate the proposed numerical model to represent sediment transport in the shallow water dam break flow. In the first test, the sediment laden dam break flow is used for the testing. This test results are compared to the experimental work by Capart and Young (1998). Then, the model is further tested on a large sluice-gate equipped storage tunnel. The storage tunnel is tested with different flushing regime (from long to short regime) and sediment depth (from high to low depth). The schematic layout of both tests is shown at Figure 1.

In the first test, the experimental measurements by Capart and Young (1998) are used for the validation of the proposed numerical model. A rectangular channel with dimensions of 12.0m long, 0.2m wide and 0.7m height is used. The sediment used in the bed has a size of 6.1mm, a density of 1048kg/m^3, and a falling velocity of 0.076m/s. The flow is initially at rest, and it has an initial water depth of 0.1m upstream of the "dam" and dry downstream, with a layer of 0.06m sediment on the bed throughout the channel from upstream to downstream. The dam, which is originally located at 4m upstream, breaks entirely at the start of the simulation. The simulation is run for up to 0.4s and the results are recorded for 0.1s and 0.4s. The short time duration of simulation used in this study aims to detect the rapid change in the water flow conditions, and it also tests the shock capturing capability of the proposed model.

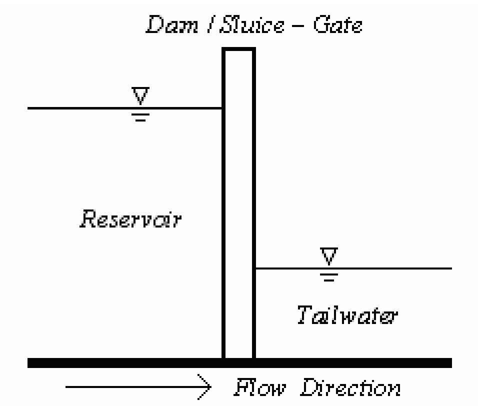

Figure 1 Schematic layout for the dam break flow and sluice-gate equipped tunnel problems

The results of the water surface and sediment bed profiles at the mid-stream location are presented in Figure 2 for 0.1s and 0.4s. At 0.1s, the flow wave surface shows a double-peak profile in experiment; whereas the proposed numerical model only shows one. The maximum water depth percentage difference of the water wave profile between the numerical prediction and the experimental measurement is about 30%. Apart from that, the numerical predicted water surface and bed level profiles show good accuracy when compared to the experimental measurements. After 0.4s, the proposed numerical model has predicted the experimentally measured water surface and bed level profiles well. The numerical predicted water surface elevation in both bore and depression waves (travel in opposite direction) show good agreement with the experimental measurements. The bed level after 0.4s shows that the numerical model has under-predicted the bed erosion at the start of the bed transport by about 28% (from about 3.9m to 4.0m); and over-predicts the bed erosion by about 15% at about 4.2m. Besides that, both numerical and experimental results compare well after 0.4s of flow. This demonstrates that the developed model can accurately simulate the movement from a sediment deposit under unsteady flush type waves.

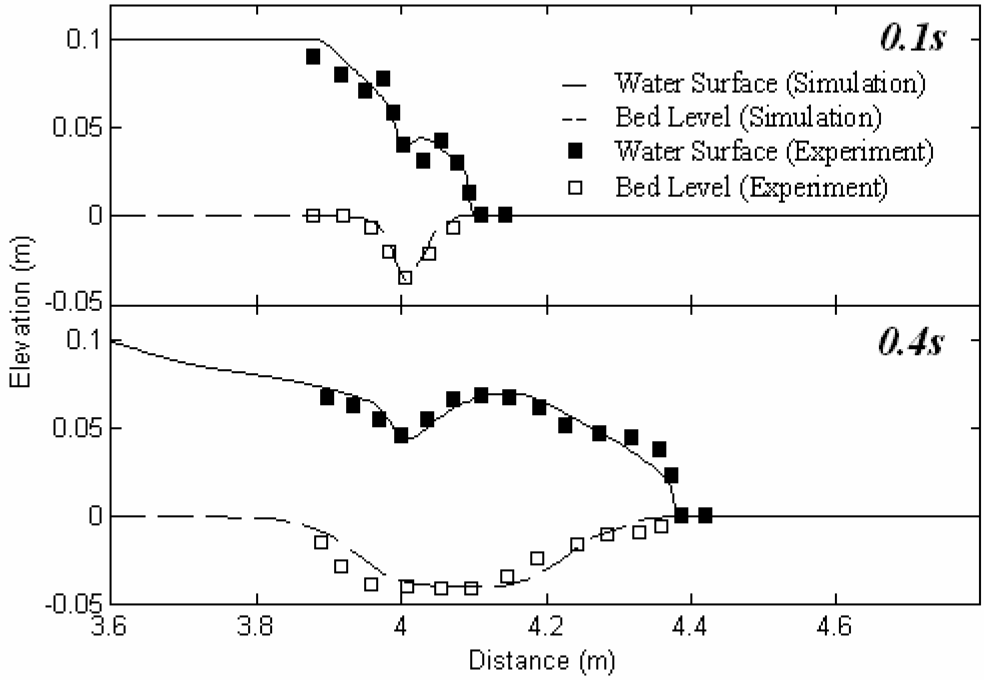

Figure 2 Comparison of water surface and bed level of experiment by Capart and Young (1998) and the proposed numerical model

Case	Tunnel Regime Separated by Sluice-Gate (m)¶		Sediment Depth (m)		% Sediment Removal After 24 Hours $[(V_I-V_R)/V_I\times100\%]$§
	Upstream	Downstream	Upstream	Downstream	
A	5000	5000	0.85	0.20	9.14
B	2500	2500	0.85	0.20	0.26
C	1250	1250	0.85	0.20	0.22
D	5000	5000	0.16	0	59.50
E	2500	2500	0.16	0	1.19
F	1250	1250	0.16	0	0.89

¶ - All results plotted at Figure 3 – 4 are using the normalised tunnel regime, which is length/total length

§ - V_I is the Initial Sediment Volume; V_R is the Remaining Sediment Volume

Table 1 Setting of Case A – F with reading of percentage of sediment removal after 24 hours

In the second stage, the proposed model is further applied to a theoretical large storage tunnel equipped with sluice-gate for sediment flushing. The purpose of this test is to find out the effects of different flushing regimes (from long to short regime) and sediment depths (from high to low depth) towards the efficiency of sediment flushing. The setting of the tunnel regime and sediment height is presented at Table 1, where the tunnel's upstream and downstream are separated by the sluice-gate. The sediment used in this test has a size of 0.35mm, a density of 2600kg/m^3, and a falling velocity of 0.06m/s. This sediment is characteristic of the granular sediment found in stormwater flows (Ashley et al. 2004), so could be reasonably expected to settle in a stormwater storage tunnel. Initially, the flow is at rest with an initial water depth of 7.2m at the upstream, and a dry downstream. A constant and uniform discharge of 8.8m^3/s is set at the outlet of the tunnel. The sluice is located at the middle of the tunnel, and it is lifted totally at the start of the simulation. The simulation is run for 24 hours to observe the sediment profile after that period. In Table 1, Case A – C are set to have a high upstream and downstream initial sediment depths; whereas, Case D – F have lower amounts of settled sediment. For each setting, a different flushing regime is used to represent different amounts of stored water used in the flushes. Table 1 also shows the percentage of sediment removal of each case after 24 hours.

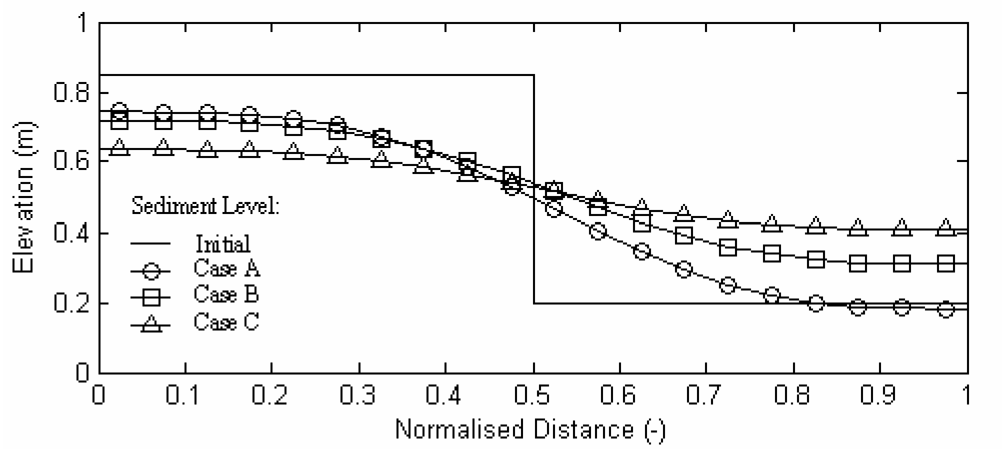

Figure 3 Sediment levels of different flushing regime of pessimistic initial sediment depth setting after 24 hours (distance is normalised using the length of the tunnel)

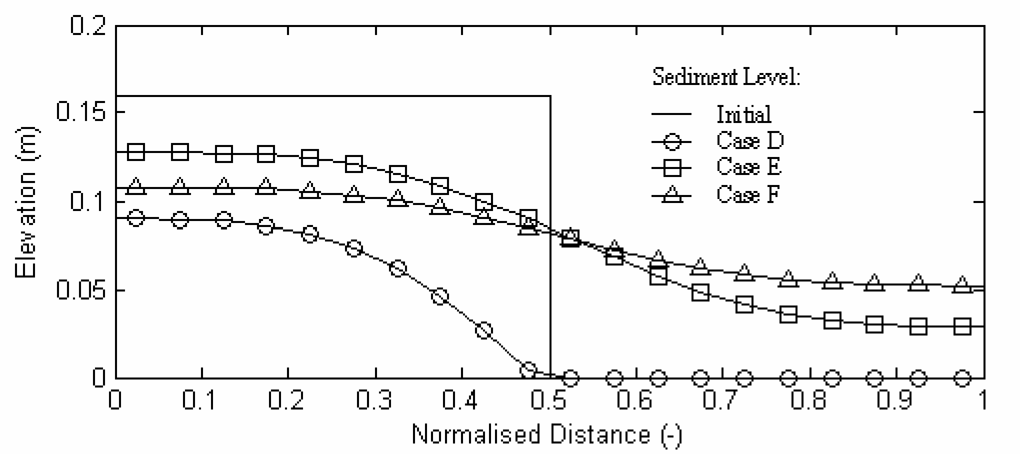

Figure 4 Sediment levels of different flushing regime for optimistic initial sediment depth setting after 24 hours (distance is normalised using the length of the tunnel)

In Figure 3, the final sediment level profile of flow with the high initial sediment depth setting is presented. It can be observed that the flushing regime with the most upstream stored water in Case A gives the best sediment flushing by removing 9.14% of sediment (refer to Table 1) after 24 hours. Both the average flushing regime (0.26% of sediment flushing – Case B in Table 1) and small water volume flushing regime (0.22% of sediment flushing – Case B in Table 1) tests do not show significant difference in sediment flushing effectiveness with each other. Figure 4 shows the final sediment level profiles of flow with the low initial sediment depth setting. The flushing regime with the most stored water (Case D) shows the best capability to flushing away all the sediment at the downstream and a large part of sediment at the upstream (59.50% of sediment flushing – Case D in Table 1). It is the most effective setting of the sediment flushing for all tests. The average flushing regime (1.19% of sediment flushing – Case E in Table 1) and the flushing regime with the smallest amount of stored water (0.89% of sediment flushing – Case F in Table 1) tests in Figure 4 show almost similar sediment flushing capability. Figure 5 shows the flow concentration [in dimensionless ratio as shown in equation (4)] at the middle and the end of the tunnel for the Case A test (most effective sediment flushing test) in time. It can be observed that the flow concentration at these two locations is more rapid change in the first 5 hours, then it maintains an almost uniform value. It is due to the opening of sluice-gate at start of the simulation that causes rapid exchange between the water and sediment phases of the flow; and this exchange is settles to a smaller and more uniform rate after about 5 hours. It can also be observed from Figure 5 that the end of the tunnel the sediment concentration always has higher value than the mid-tunnel. This concentration difference becomes larger with time; until after about 10 hours of simulation when the difference becomes constant. This indicates that as time progresses that it becomes increasingly difficult to move sediment that had been deposited upstream of the flushing gate.

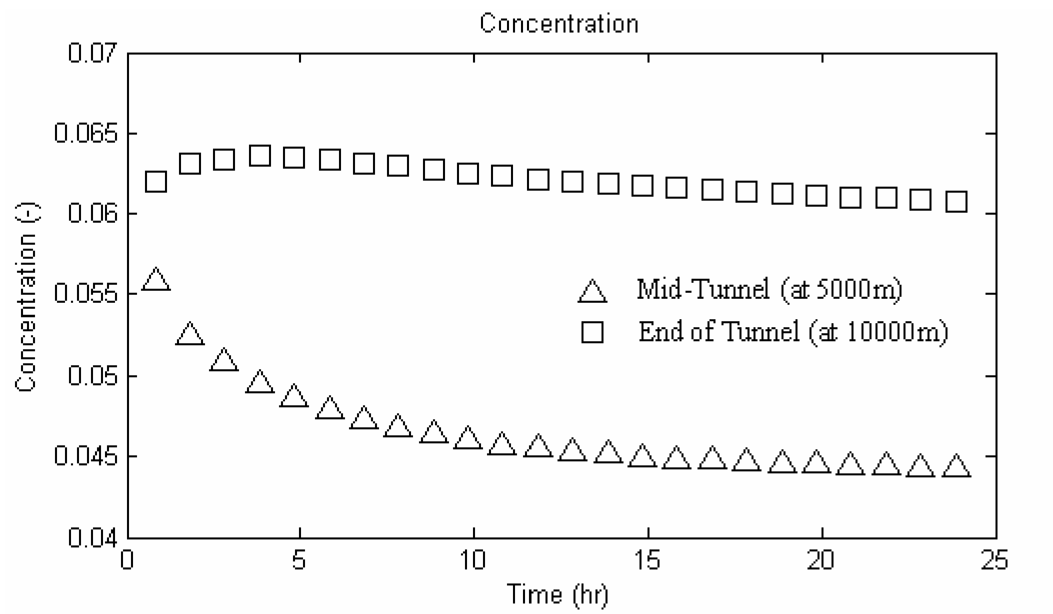

Figure 5 Concentration against time at the middle and end of tunnel for Case A test

5. CONCLUSIONS

In this paper, a Finite Volume (FV) numerical model was presented to analyse shallow flow with sediment transport. The numerical model is based on a MUSCL-HANCOCK scheme with an HLL Riemann solver. The model was applied to tests data from experiments with dam break flow and sediment transport in order to test the transport of sediment under discontinuous and unsteadiness nature of the flow. The results of these simulations were compared with experimental data from literature for validation. It is found that the proposed numerical model predicts the results with a good correspondence with the experimental observations. A numerically modelled sluice-gate equipped storage tunnel with different flushing regimes and sediment deposits was then simulated using the developed numerical model to test the affects of flushing regime and initial sediment depth on the effectiveness of sediment flushing as a sediment management technique. It is found that the both the volume of water stored by the gate and the depth of the initial sediment deposit had a significant influence on the amount of sediment removed within a flush. . However, none of the settings tested in this paper have successfully flushing away all sediment in a single flush. This suggests that the use of hydraulic flushes needs to be carefully considered especially for larger storage tunnels in which the volume of sediment to be removed is large. Sediment flush waves are much more efficient at removing sediment downstream of the gate that creates the flush wave, that sediment that is deposited upstream of the gate.

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