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Research papers

Experimental calibration and validation of sewer/surface flow exchange equations in steady and unsteady flow conditions

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ABSTRACT

The linkage between sewer pipe flow and floodplain flow is recognised to induce an important source of uncertainty within two-dimensional (2D) urban flood models. This uncertainty is often attributed to the use of empirical hydraulic formulae (the one-dimensional (1D) weir and orifice steady flow equations) to achieve data-connectivity at the linking interface, which require the determination of discharge coefficients. Because of the paucity of high resolution localised data for this type of flows, the current understanding and quantification of a suitable range for those discharge coefficients is somewhat lacking. To fulfil this gap, this work presents the results acquired from an instrumented physical model designed to study the interaction between a pipe network flow and a floodplain flow. The full range of sewer-to-surface and surface-to-sewer flow conditions at the exchange zone are experimentally analysed in both steady and unsteady flow regimes. Steady state measured discharges are first analysed considering the relationship between the energy heads from the sewer flow and the floodplain flow; these results show that existing weir and orifice formulae are valid for describing the flow exchange for the present physical model, and yield new calibrated discharge coefficients for each of the flow conditions. The measured exchange discharges are also integrated (as a source term) within a 2D numerical flood model (a finite volume solver to the 2D Shallow Water Equations (SWE)), which is shown to reproduce the observed coefficients. This calibrated numerical model is then used to simulate a series of unsteady flow tests reproduced within the experimental facility. Results show that the numerical model overestimated the values of mean surcharge flow rate. This suggests the occurrence of additional head losses in unsteady conditions which are not currently accounted for within flood models calibrated in steady flow conditions.

1. Introduction

The increased frequency and magnitude of worldwide flood events in recent years (Ravazzani et al., 2016) has encouraged a critical examination of possible causes (Hundecha et al., 2016) and suitable options to reduce their impact. Urban flooding may occur when storm water exceeds the capacity of the local sewer or storm water system. Dual drainage hydraulic models have been developed to assess the risks associated with urban flooding, namely the potential damage to property and infrastructure (Martins et al., 2016; Mark et al., 2004), and to supply information for decision makers (Fernandez et al., 2016). Such modelling tools use steady state linking discharge equations to enable the coupling of below-ground pipe flow and free surface flow at computational (interface) nodes representing manholes/gullies (e.g. Leandro et al., 2009; Lee et al., 2015; Maksimović et al., 2009). In such forms of integrated flood modelling, interaction discharges are usually added as sinks or sources within the overland flow model (e.g. Seyoum et al., 2012; Chen et al., 2015; Leandro and Martins, 2016; Martins et al., 2016).

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State-of-the-art dual drainage models (Smith, 2006) couple 1D (one-dimensional) sewer network flow models to 2D floodplain model (Chen et al., 2015; Leandro et al., 2009; Lee et al., 2013; Schmitt et al., 2004) utilising weir or orifice equations to describe flow exchange between the surface and sewer systems as a function of relative hydraulic head in the sewer and surface systems (Djordjević et al., 2005; Chen et al., 2007). These discharge relationships were derived using the principles of energy or momentum conservation assuming a steady state flow, with discharge coefficient comprising energy losses. Nonetheless they are commonly employed within unsteady flow simulators on the assumption that the computational time step is small (i.e. dV/dt is assumed constant at each time step, V is the volume of flow). In reality, flows through these interaction nodes during flood events are highly complex, transient and three dimensional, with surcharge/drainage affecting local hydrodynamics around the interaction node, and the flow direction potentially altering several times during a flooding event (Balmforth et al., 2006). Although it may be more appropriate to apply 3D models to replicate such systems, equivalent models are computationally demanding and hence not applicable to real urban inundation events (Cea et al., 2010). In these cases the use full shallow water equation models or simplified models neglecting the inertial terms is deemed acceptable (Mignot et al., 2006). The representation of surface/subsurface flow exchange and energy loss processes is recognised as a potential source of uncertainty within urban flood models (Djordjević et al., 2005), especially because little guidance exists on a range of suitable weir/orifice discharge coefficients for use in a flood modelling context.

The three dimensional and rapidly changing nature of interaction flows mean that the accurate characterisation of hydraulic conditions around interaction nodes is extremely challenging. The uncertain nature of flood events and the difficulties in obtaining data at suitable spatial and temporal resolutions at a field site make full scale calibration and verification of linking equations implausible. Studies using physical scale models to calibrate and validate the performance of interaction points during flood flows are limited. Recently Russo et al. (2013) and Djordjević et al. (2013) investigated the hydraulic performance of gully pots, however they reported surface to sewer flow conditions only. Leandro et al. (2014), Lopes et al. (2015) and Martins et al. (2014) studied the hydrodynamic effects of the flow inside a gully without grate and proposed coefficients for the drainage flow. Bazin and Nakagawa (2014) quantified and modelled flow exchange between below and above ground systems but their tests were limited to scenarios with pressurised pipe conditions only and the scale of the model limited the range of flow Reynolds numbers tested. Fraga et al. (2015) validated a 1D-2D dual drainage model using a real-scale physical model to quantify rainfall-runoff transformation and presented a satisfactory performance (discrepancies were below 2%) of the numerical model when replicating water depth and discharge at several locations in a drainage network. However to date, no study has presented experimental validation of interaction flow modelling during unsteady tests featuring both surface to sewer and sewer to surface flow conditions.

To address this gap, this paper uses a physical model of a surface/pipe system linked via a scaled manhole to present experimental datasets of sewer-surface flow exchange. The tests conducted here are limited to conditions where no manhole lid is present, as may be the case where the lid has previously been removed or ejected due to surcharge. This conditions was chosen to enable a series of steady state experiments over a range of flow rates and exchange conditions are used to assess the applicability of weir and orifice equations to represent exchange flows as well as identify suitable discharge coefficients to represent energy losses. In order to validate these relationships in unsteady conditions, the functions are implemented within a Finite Volume (FV) Godunov-based numerical model, and its performance is compared to experimental datasets from a series of unsteady flow tests.

2. Overview of surface–subsurface linking equations

This section provides an overview of the weir and orifice linking equations which are commonly used to determine the exchange of flow at the interface within urban flood modelling.

2.1. Surface-to-sewer exchange

Considering an equivalent datum point, surface-to-sewer exchange through a linking node occurs in all cases when the hydraulic head of surface flow is greater than the hydraulic head of the pipe flow. Within coupled urban flood models (e.g. Chen et al., 2007; Djordjević et al., 2005; Leandro et al., 2009; Martins et al., 2016; Seyoum et al., 2012) this exchange flow is commonly quantified using equations originally derived for flow over a weir or through an orifice. When considering the surface elevation relative to the pipe flow hydraulic head (Z_{crest}), two conditions can be defined.

2.1.1. Head in the pipe network less than surface elevation

In this case, the free weir equation is normally used to describe flow exchange. Within urban flood models the length of the weir is taken as the manhole perimeter, and the hydraulic head of the flow is considered to be equal to the flow depth above the surface elevation (h_s), (i.e. velocity head is assumed to be negligible). Hence the linking equation is taken as:

$$-Q_e = \frac{2}{3} C_i \pi D_M (2g)^{1/2} (h_s)^{3/2} \tag{1}$$

where Q_e is flow exchange (m^3/s) and D_M is the manhole diameter (m), C_i is an energy loss coefficient that is included in order to account for losses due to viscous effects (–). Within existing flood models, the free weir scenario is considered applicable in all cases when pipe network hydraulic head does not exceed the surface elevation, although in Djordjević et al. (2005) it is noted that a somewhat reduced capacity should be considered at high flow rates when the manhole becomes submerged by the surface flow.

2.1.2. Head in the pipe network exceeds surface elevation

In this case, Chen et al. (2007) use a linkage based on a submerged weir equation, in which a term is included to account for the difference between surface flow depth and hydraulic head in the pipe network (h_p). This can be expressed as:

$$-Q_e = C_i \pi D_M (2g)^{1/2} (h_s) \left(h_s + Z_{crest} - h_p\right)^{1/2} \tag{2}$$

where Z_{crest} is the height difference from the invert of the pipe system to the level of the surface. This linkage is considered applicable when $h_p > Z_{crest}$ and $h_s < A_M/\pi D_M$ where A_M is manhole area (m^2). If $h_s > A_M/\pi D_M$ the link is considered fully submerged and the submerged orifice formula is expected to be a more suitable description of flow behaviour. The submerged orifice equation can be expressed to provide flow exchange as:

$$-Q_e = C_i A_M (2g)^{1/2} \left(h_s + Z_{crest} - h_p\right)^{1/2} \tag{3}$$

In this case, the discharge coefficient C_i accounts for energy losses due to flow through the orifice, the continued contraction of the jet as it passes through the restriction (vena contracta), and the assumption of negligible velocity head in the upstream (i.e. surface) flow.