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Understanding the interaction of sewer-to-surface flows in urban floods

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Dissemination Seminar: Modelling of Urban Flood Flows – 25th January 2018, 9:30 - 15:00, LT4, The Diamond, University of Sheffield, S3 7RD

Objectives of flood modelling?

- Sewer system re-design / optimisation
- Major system design
- Damage assessment
- Flood risk attribution
- Hazard maps
- Real-time management

- Support to rescue services
- Uncertainty analysis
- Pollution, health problems
- Climate change impacts
- Effects of urban growth

All these objectives require the **estimation of flow exchange between sewer and floodplain (especially associated with flooding events)**

Surface water flooding is recognised as the **hardest type of flooding to predict and defend against** (*Pitt Review*)

URBAN FLOODING

Sewer and floodplain flows

Interactions!

Outflowing

Inflowing

Q??

Paucity of real datasets to validate and calibrate numerical models!

Sub-surface/surface interactions

- An interaction point between two model types
- Flow rate between below-above ground depends on ?

↓

i. water level on the surface

ii. hydraulic head in the sub-surface element

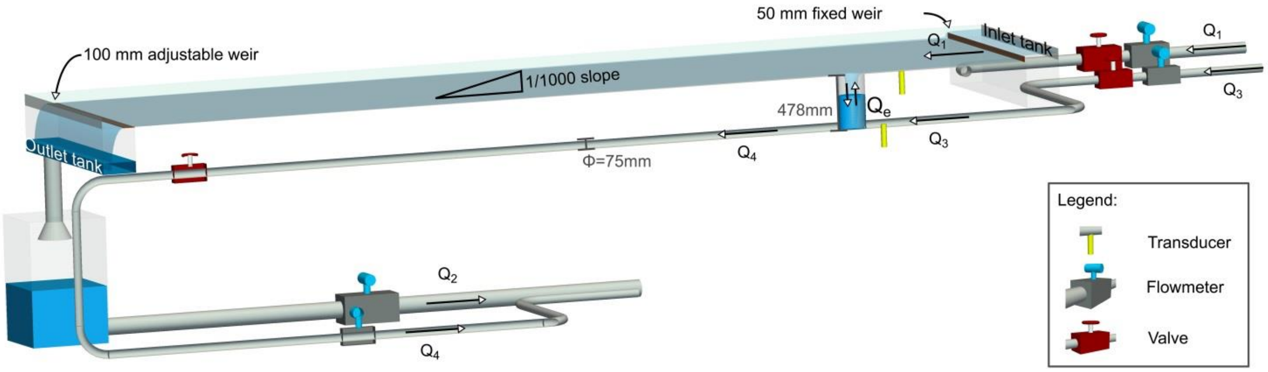
iii. local terrain level and slopes

iv. surface flow velocity and direction

v. geometry of the link (inlet/gulley/manhole)

vi. partial blockages (silted inlet/manhole cover)

Sub-surface/surface interactions



Inflow into unsurcharged sewer

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

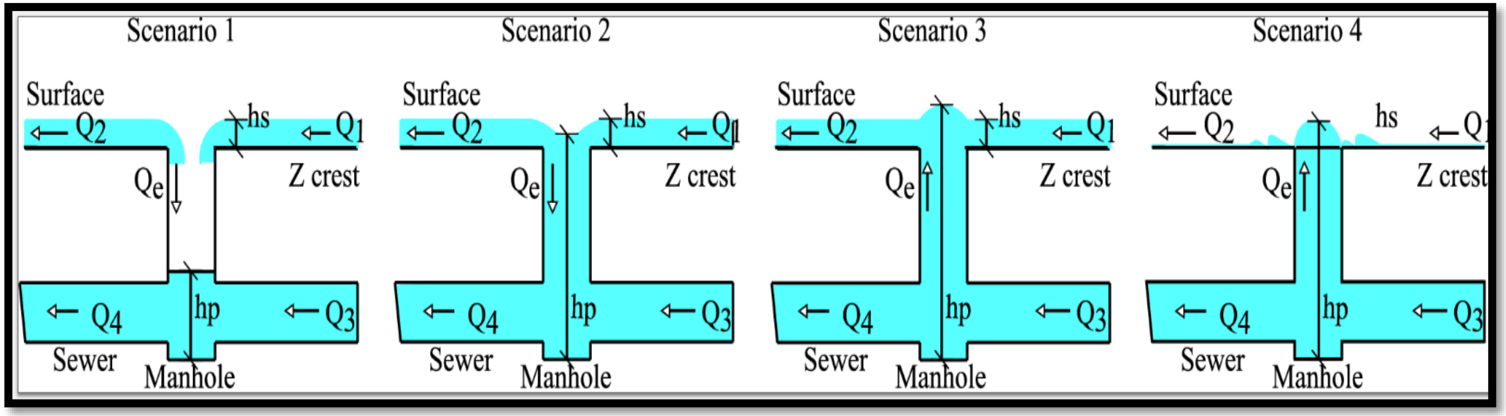
Inflow into surcharged sewer

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

Outflow over wet floodplain

Outflow over dry floodplain (h_s=0)

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



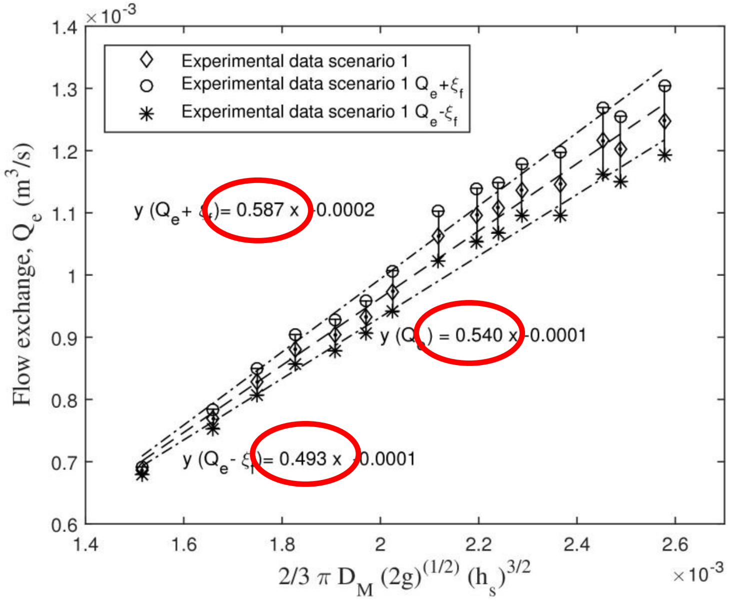
✓ ✓ ✓ ✓

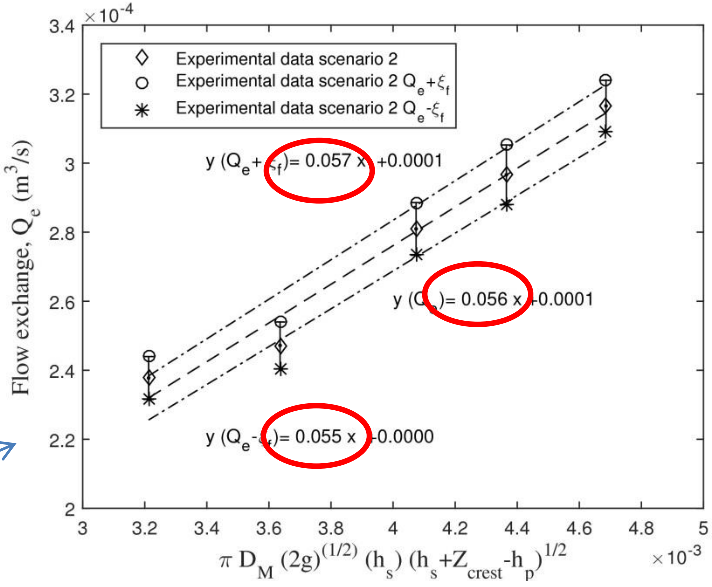
OUTPUTS (1)

M.Rubinato, R.Martins, G.Kesserwani, J.Leandro, S.Djordjevic, J.Shucksmith.

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

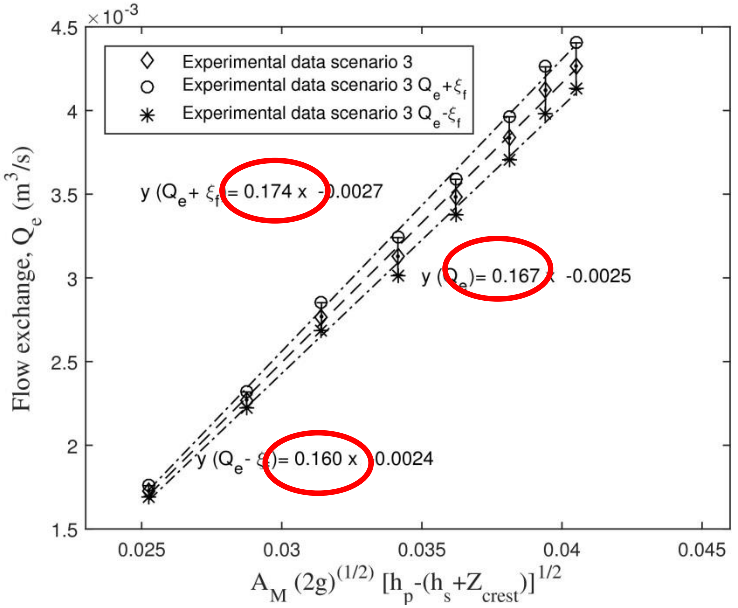
Journal of Hydrology, 2017, 552, 421-432, https://doi.org/10.1016/j.jhydrol.2017.06.024

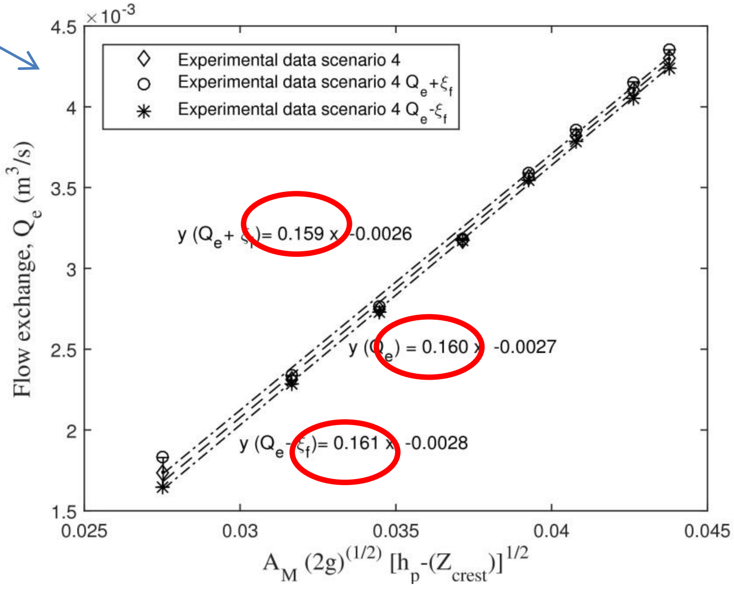


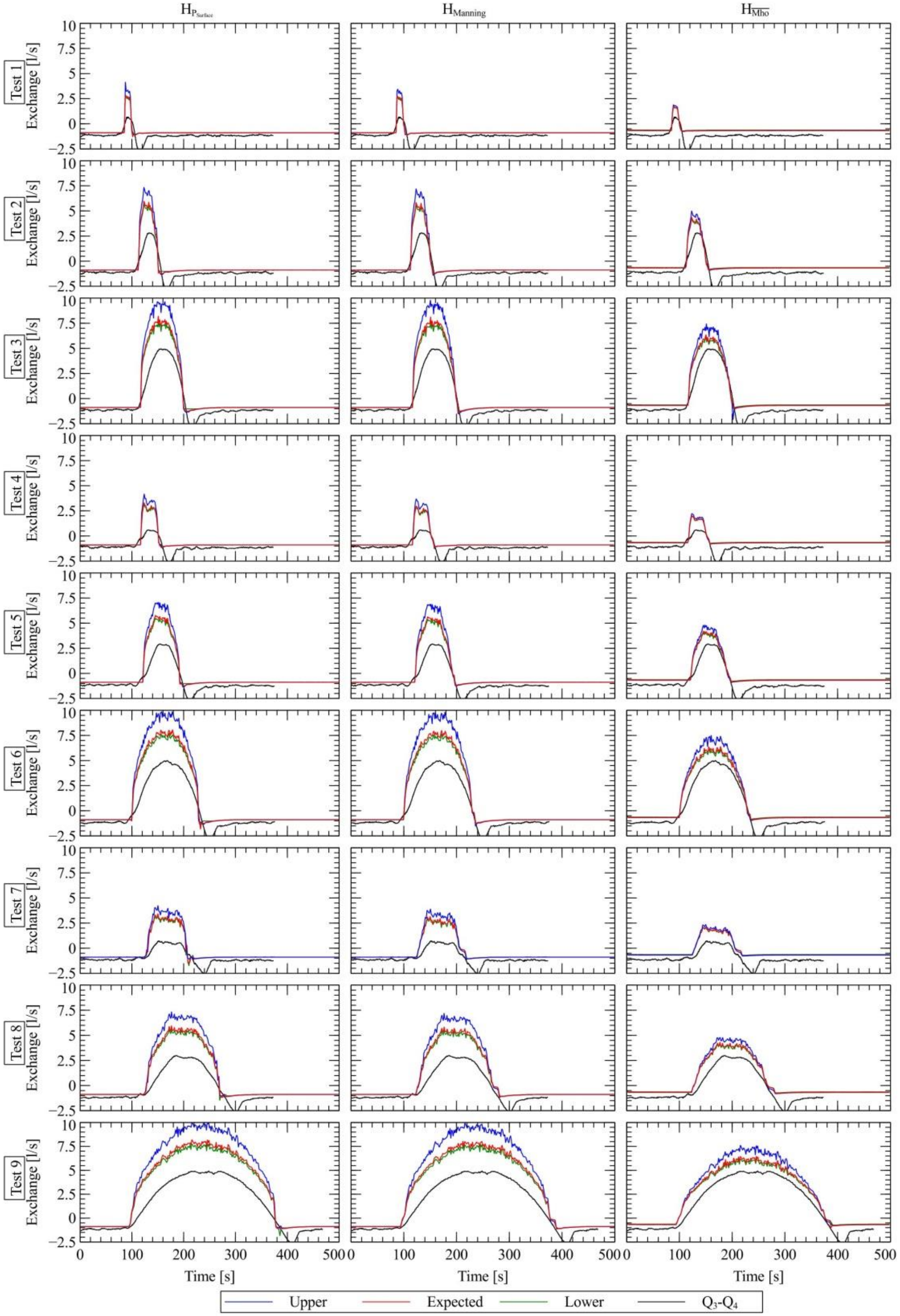


These results show that **existing weir and orifice formulae are valid for describing the flow exchange** for the present physical model

These results yield new calibrated discharge coefficients for each flow conditions







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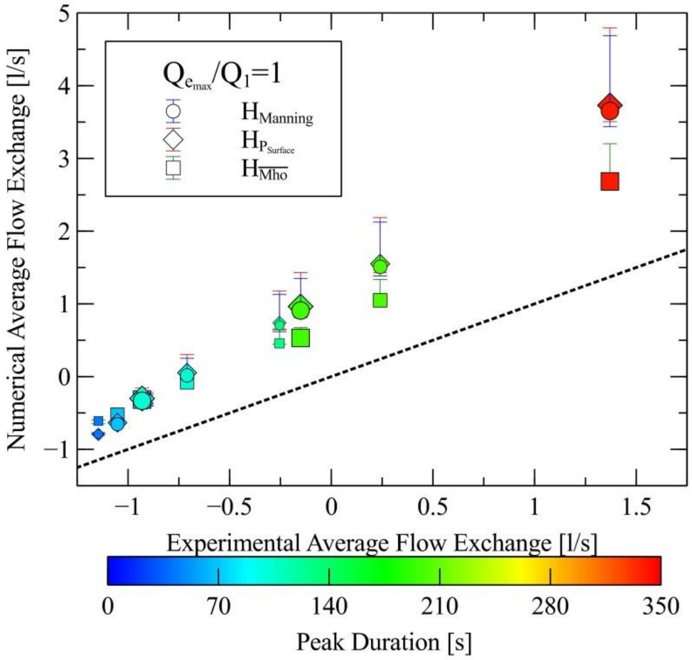
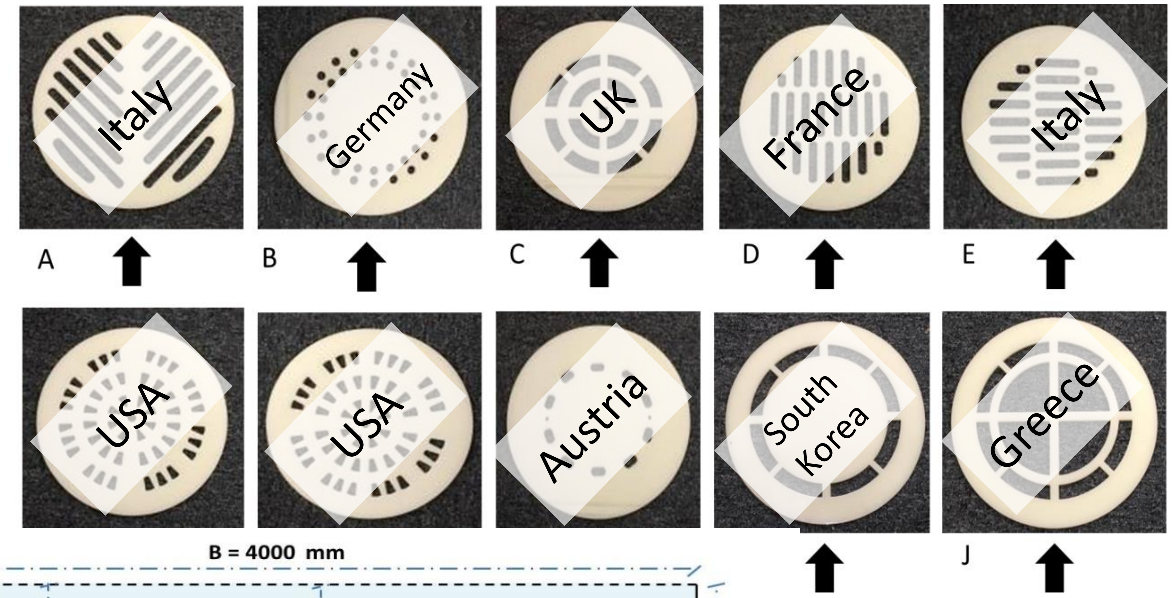
This suggests that in unsteady surcharging conditions, significant head losses are encountered over and above those in steady state flow (where the model provided high accuracy)



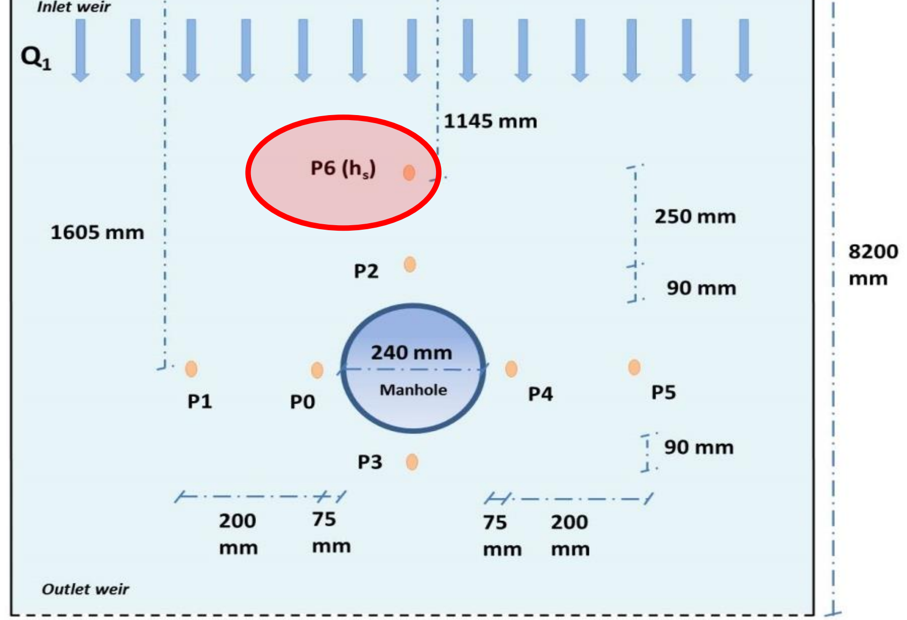
Linking equations are sensitive to calculations of relative head within pipe and surface systems.

OUTPUTS (2)

M.Rubinato, S.Lee, R.Martins, J.Shucksmith. **Surface to sewer flow exchange through circular inlets during urban flood conditions**. Journal of Hydroinformatics, under review.

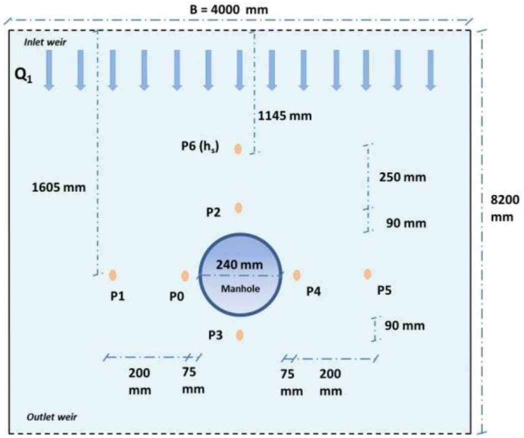


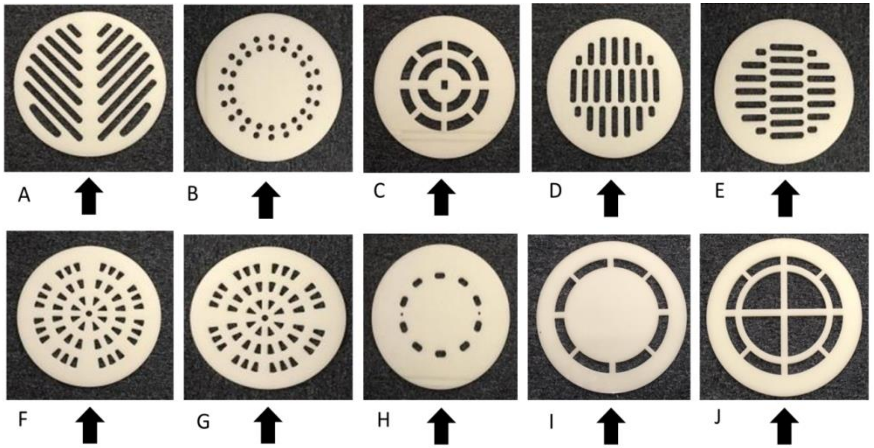
Grates applied on the top of the manhole (Black arrows show the primary direction of the facility inflow Q_1 and hence the orientation of each manhole grate).

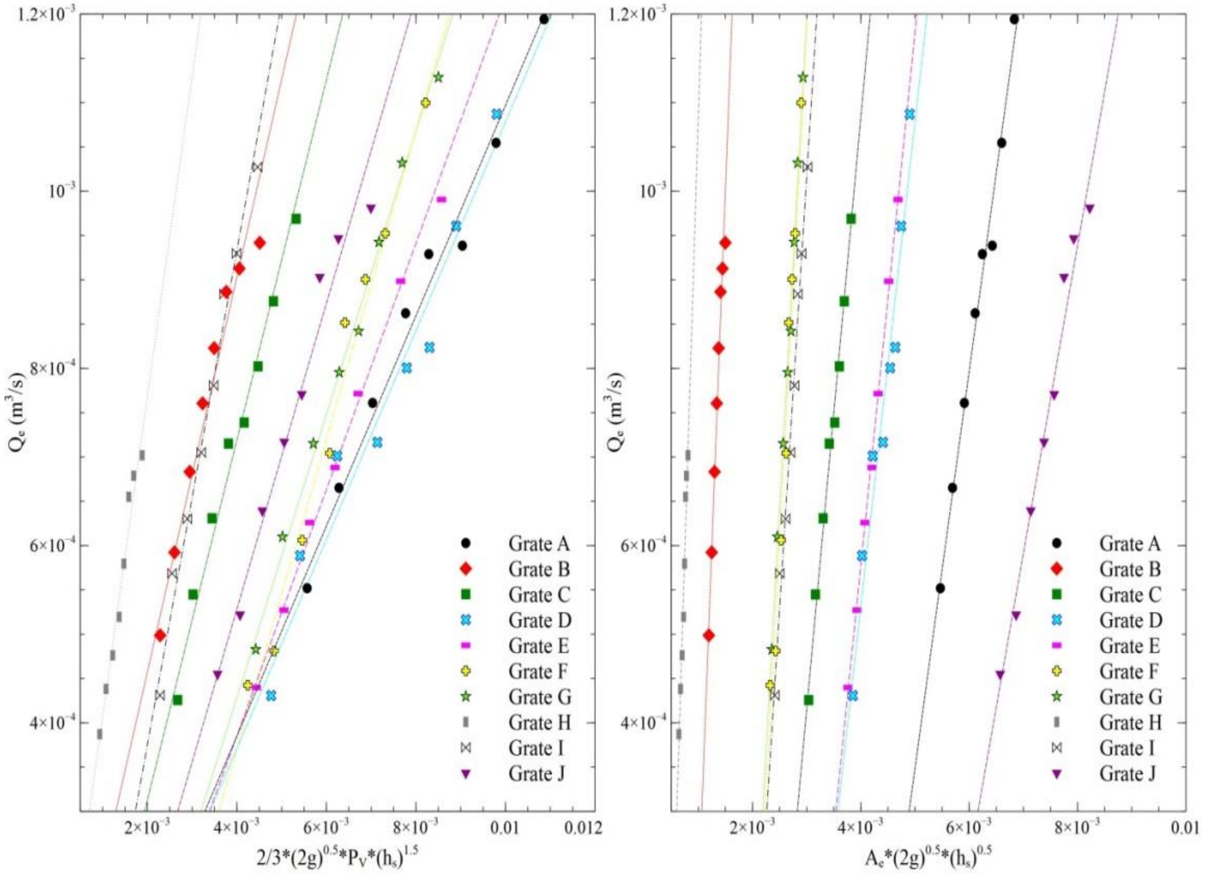


$$Q_e = \frac{2}{3} C_W P_v (2g)^{1/2} (h_s)^{3/2}$$

$$Q_e = C_O A_e (2g)^{1/2} (h_s)^{1/2}$$


B = 4000 mm
Inlet weir
Q_1
1145 mm
P6 (h_s)
250 mm
1605 mm
P2
90 mm
8200 mm
240 mm
Manhole
P1
P0
P4
P5
90 mm
P3
200 mm
75 mm
75 mm
200 mm
Outlet weir


A
B
C
D
E
F
G
H
I
J


1.2×10⁻³
10⁻³
8×10⁻⁴
6×10⁻⁴
4×10⁻⁴
Q_e (m³/s)
2×10⁻³
4×10⁻³
6×10⁻³
8×10⁻³
0.01
0.012
$2/3*(2g)^{0.5}*P_V*(h_s)^{1.5}$
$A_e*(2g)^{0.5}*(h_s)^{0.5}$
Grate A
Grate B
Grate C
Grate D
Grate E
Grate F
Grate G
Grate H
Grate I
Grate J

Q_e (m^3/s)

OUTPUTS (3)

M.Rubinato, R.Martins, J.Shucksmith. **Quantification of energy losses at a surcharging manhole**. Urban Water Journal, http://dx.doi.org/10.1080/1573062X.2018.1424217 .

ENERGY LOSSES ARE AFFECTED BY…

Depth ratio between upstream branches and the downstream channel (*Taylor et al., 1944; Hsu et al., 1998*)

Upstream and downstream hydraulic conditions (i.e. subcritical or supercritical, (*Hager et al., 2005; Del Giudice et al., 2000; Zhao et al., 2006; Gargano et al., 2002*)

Bed discordance over the manhole junction (*Biron et al., 1996*)

Other flow characteristics, e.g. the flowrates in the inlet pipes, whether the pipes are running gull or part-full, supercritical or subcritical, the effect of tail water level and the water level in the manhole (*O'Loughlin et al., 2002*)

Presence of a lateral pipe and variation in flow rates between the main pipe and lateral pipe (*Zhao et al., 2006; Ramamurthy et al., 1997*)

The joining angle between any lateral pipes and the main pipe (*Pfister et al., 2014*)

Existence of sump inside the manhole and benching effects (*Arao et al., 2011*)

Ratio between pipe diameter and manhole diameter (*Ramamurthy et al., 1997*)

Ratio between water depth in the manhole and pipe diameter (*Ramamurthy et al., 1997*)

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ENERGY LOSSES WERE INVESTIGATED IN…

90° bend junctions (*Marsalek, 1988*)

90° combining junctions (*Marsalek, 1985; Wang et al., 1998*)

25.8° combining junction with two inflows and one outflow (*Zhao et al., 2006*)

Pfister and Gisonni, (2014) presented an experimental extensive campaign on a physical model to investigate the local head losses of combining flows at 45° and 90° junction manholes on circular conduits, with various diameters and in the presence of sub and supercritical approaching flows

Oka and Ito, (2005) determined energy losses coefficients for smooth, sharp-edged tees of circular cross section for five branch angles which ranges from 45° to 135°

However, despite the important application of hydraulic models to urban flood events, <u>local energy losses in manholes during sewer to surface surcharge events were yet to be investigated</u>

The lack of reliable data sets during flood events means direct calibration of energy losses in surcharging flows is difficult *(Hunter et al, 2008)*

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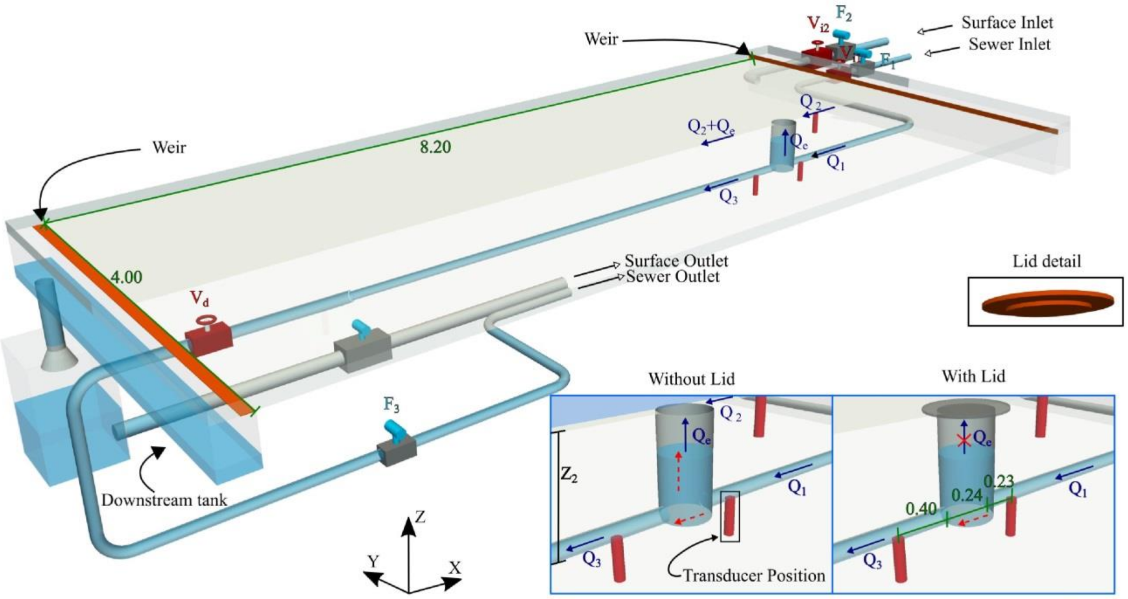

Set 1: A set of duplicate tests were conducted in which sewer inlet flow and surcharge rate was varied (Q_e ranged between 0 and 2.59 l/s), with (WL) and without (WoTL) the presence of a lid described above. Surface inflow (Q_2) was set as zero in all cases and the downstream sewer valve was set at a constant position (V_d= 48%).

Set 2: Tests were completed with two different flow conditions on the surface in combination with varying degree of closure of the downstream sewer valve (V_d presented in section 3.1) and surcharge rate (Q_e ranged between 0 and 7.28 l/s).

OUTPUTS (3)

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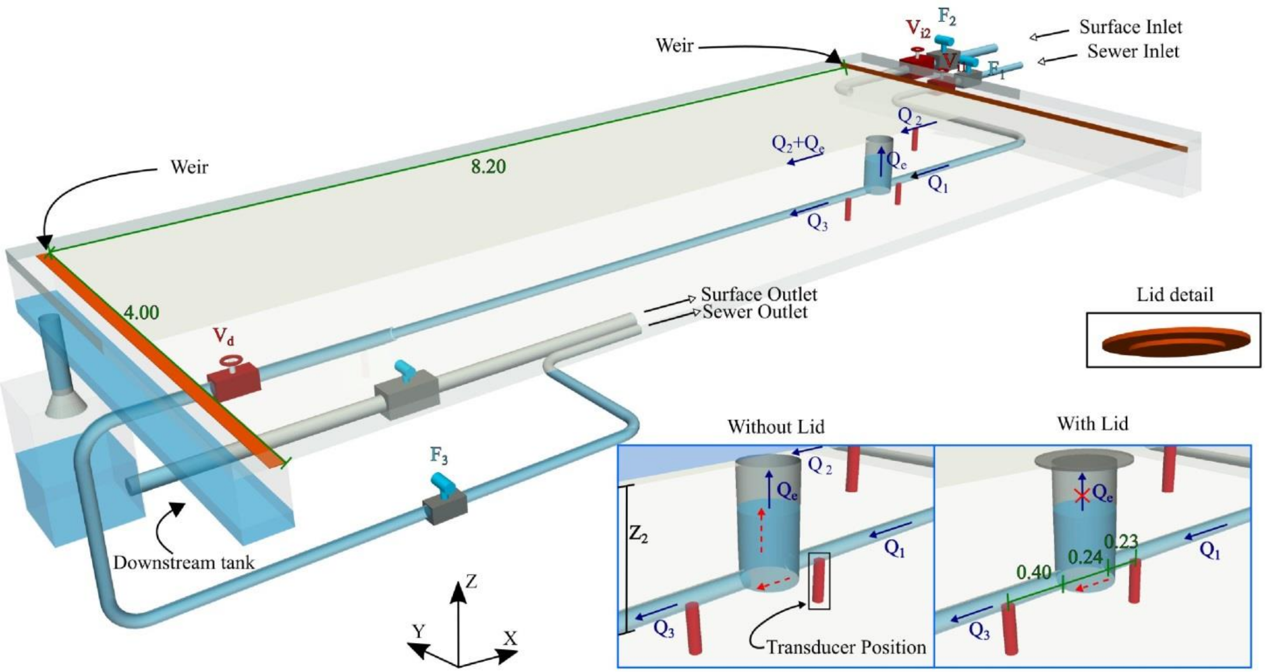
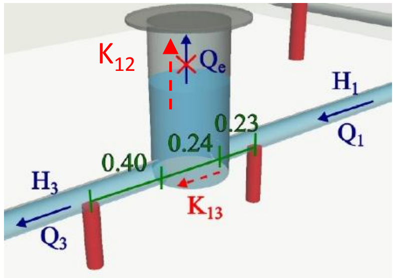


$$\rho g(H_1 Q_1) = \rho g(H_3 Q_3 + H_2 Q_e + \Delta H Q_1)$$

$$\Delta H = H_1 - (H_3 \frac{Q_3}{Q_1} + H_2 \frac{Q_2}{Q_1})$$



In this study we consider that this condition is analogous to a bifurcation, in which the flow splits into two streams, one continuing within the sewer, and one existing to the surface

$$K_{13} = \frac{H_1 - H_3}{u^2{}_1/2g} \qquad K_{12} = \frac{H_1 - H_2}{u^2{}_1/2g} \qquad K_{TOT} = \frac{\Delta H}{u^2{}_1/2g}$$

OUTPUTS (3)

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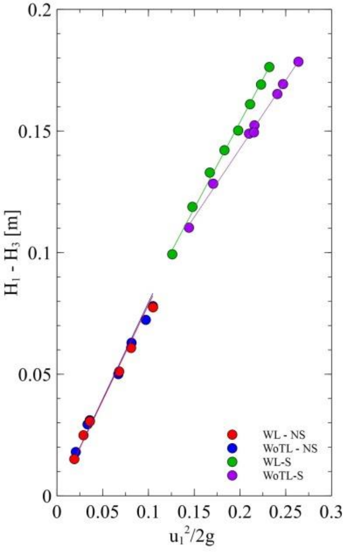
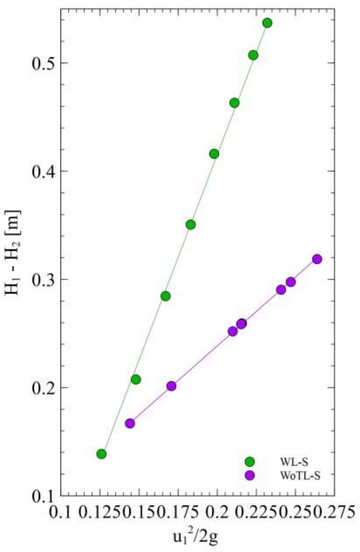
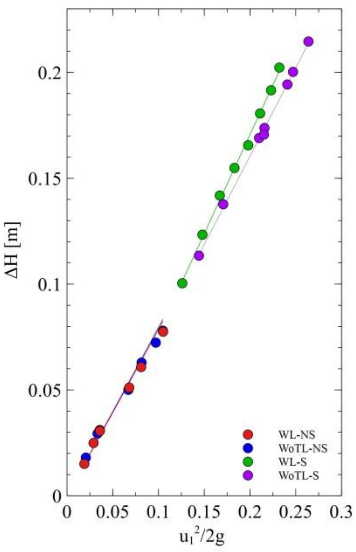
Set 1

R^2>0.986 for all the cases

In non-surcharging conditions, energy loss coefficients are unaffected by the presence of a manhole lid







In surcharging conditions, the coefficients (and hence energy losses) are lower when the lid is removed (from $K_{13,SWL}$=0.699 to $K_{13,SWoL}$=0.559; from $K_{12,SWL}$=3.865 to $K_{12,SWoL}$=1.269 and from $K_{TOT,SWL}$=0.933 to $K_{TOT,SWoL}$=0.836)

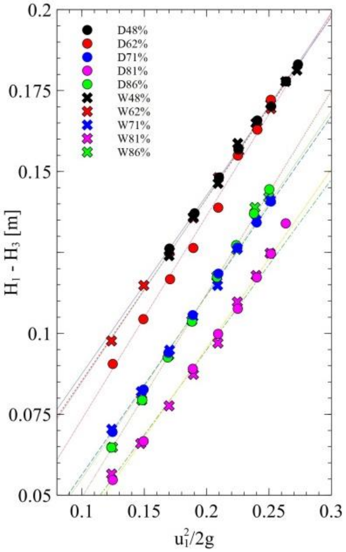
↓

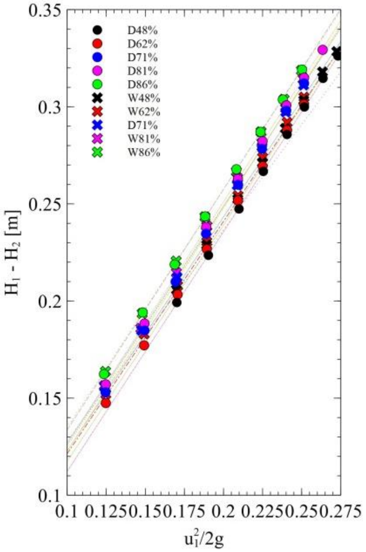
This suggests higher turbulent energy losses in conditions where the flow is forced past a lid than when compared to a condition in which flow can move freely to the surface.

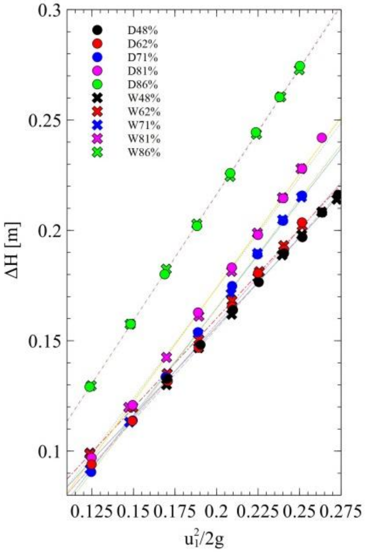
Set 2

R^2>0.993 for all the cases

K_{13} for V = 86% > K_{13} for V = 81%) which may be due to the high turbulent flow that is forced to escape the sewer system through the manhole.







OUTPUTS (3)

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SIPSON solves the full dynamic Saint-Venant equations in the pipes:

- $\frac{\partial z}{\partial t} = \frac{1}{b}\frac{dQ}{dx} = 0 \qquad (1)$
- $\frac{\Delta Q}{\Delta t} + \frac{\partial\left({Q^2}/{A}\right)}{\partial x} + gA\frac{dz}{dx} + gAS_f = 0 \qquad (2)$

The mass and energy conservation are computed at each node through:

- $A_n\frac{dz_n}{dt} = Q_n + \sum_{m=1}^{M} \pm Q_m\,, \qquad z + \frac{u^2{}_{cs}}{2g} = z_n \pm K\frac{u_{cs}|u_{cs}|}{2g} \qquad (3)$

A Preissmann four-point implicit Finite differences scheme is used with the conjugate gradient method to solve the system of equations (1), (2) and (3).

→ SIPSON calculates minor losses inside the manhole using the node cross sectional velocity u (3).

OUTPUTS (3)

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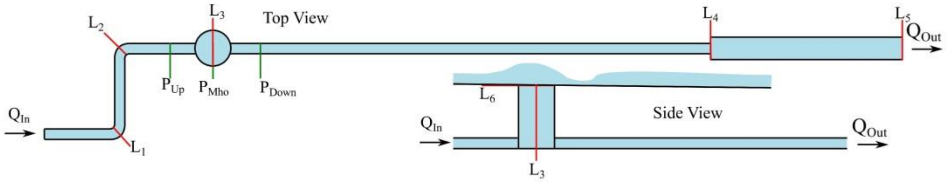
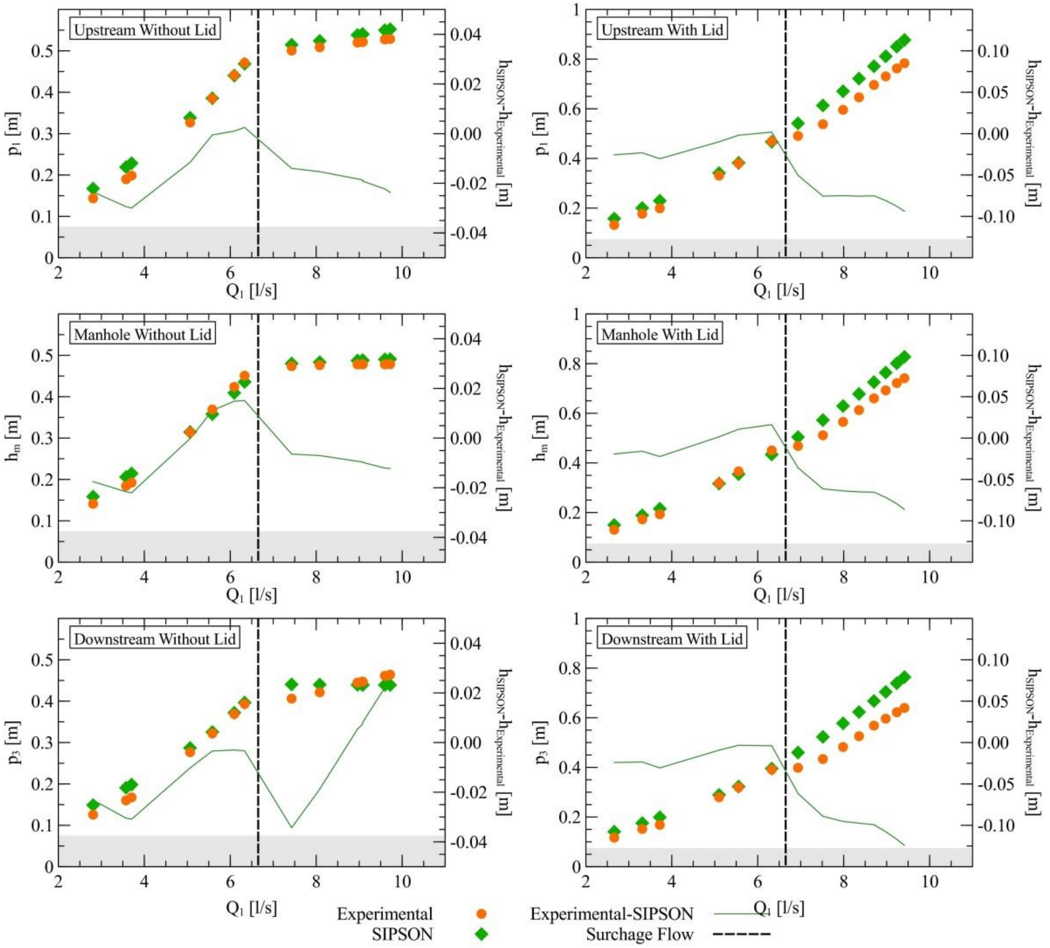
Secondary head loss parameter (L_1, L_2, L_3, L_4, L_5, L_6):

L_1 = 0.36 (*Lencastre,1996*)
L_2 = 0.36x2 (*Lencastre,1996*)
L_3 = NSWL=0.757, NSWoTL=0.760, SWL=0.699, SWoTL=0.559 (from experimental data SET 1)
L_4 = 0.0625 (sudden expansion *Idelcik, 1948*)
L_5 = 1.5 (gate valve losses *Puppini 1947*)
L_6 = 1.269 (from experimental data SET 1)



R^2> 0.982 in all the cases



For the non-surcharging conditions, **discrepancies are very close**, between 0-0.04 m without the application of the lid on the top of the manhole and within the range 0-0.025 m with the lid application.

When considering surcharging conditions, **SIPSON tends to overestimate** experimental pressure results. **Dissimilarities are greater for tests conducted with the application of the lid** (up to 0.1 m) whilst in no-lid cases the deviations between experimental and numerical do not exceed 0.04 m.

FUTURE WORK

- Investigate additional head losses during net sewer-to-surface exchange in unsteady conditions to reduce errors in flood modelling applications

- Explore the relationship between water depth and flow exchange under different street profiles replicated on the urban surface

Thanks a lot for your attention, any questions?

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