A decision support tool for optimal placement of sewer mining units: Coupling SWMM 5.1 and Monte-Carlo simulation

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Abstract

Rapid urbanization and potential water shortages due to supply side impacts of climatic change have led to the development of innovative water and wastewater reuse strategies. A mid-scale decentralized option that can provide recycled water for numerous uses, including agriculture and urban applications, is that of sewer mining. The idea is to provide reclaimed water by extracting wastewater from the sewers, treating it at the point of demand and, in some cases, returning treatment residuals back to the sewer system. Public perception, inadequate regulatory frameworks, as well as engineering issues, are some of the challenges that pose barriers in adapting such solutions. One of these challenges is hydrogen sulfide build-up, which can cause odor, corrosion and human health-related problems. In order to address the latter issue, we propose a method that couples the advantages of Monte-Carlo simulation with SWMM 5.1 model. The method is able to identify potential locations for sewer mining placement and simultaneously account for the network characteristics and dynamics (i.e., wastewater flow and BOD_5 fluctuations). The overall scheme was applied in a future sewer network in Greece providing useful results and can therefore serve as a guideline in up-scaling sewer mining at a city level.

Keywords: decentralized water treatment; sewer mining; hydrogen sulfide; SWMM simulation model; Monte-Carlo method

1. Introduction

Nowadays, there is an increasing need for new strategies concerning the management and treatment of water. This need is mostly dictated by the capacity limitations due to urbanization and population increase, the necessity for a more sustainable use of water, as well as the climate change-related water shortages. Thus, several steps have been taken towards decentralized and satellite approaches (Crites and Tchobanoglous, 1998). One decentralized approach, applicable at the development level, (for example, up to 5000 households) (Makropoulos and Butler, 2010) is that of sewer mining. It is a process involving the extraction of wastewater from a wastewater system and the following treatment of the extract for the production of recycled water. Some of the by-products of this process can be returned to the network under strict standards (Barwon Water, 2011). Sewer mining can provide recycled water for multiple uses, including toilet flushing, irrigation of green areas (sports fields, golf courses, parks etc.), and applications in commercial buildings and industrial sites (Marleni *et al.*, 2013, Ødegaard, 2012). Although this practice has been successfully implemented in numerous plants, most of them in Australia (McFallan and Logan, 2008), the public still regards it with skepticism. The absence of adequate regulatory framework and the concern about recycled water quality, combined with its pricing, raise barriers to the promotion of sewer mining (McFallan and Logan, 2008). Additionally, some engineering challenges emerge from the implementation of sewer mining in a wastewater system. The extraction of the sewage and the subsequent changes in the downstream wastewater flow and mass loading, along with the potential disposal of the sludge back into the system, cause alterations to sewage biochemical processes (Marleni *et al.*, 2013). Hydrogen sulfide (H_2S) production inside the pipes, which can lead to network degradation, is also affected. This study presents a tool for optimal placement of sewer mining (SM) units, with respect to H_2S build-up, using the EPA SWMM 5.1 platform. Monte-Carlo method is combined with kinematic wave routing for the network simulation and H_2S production is predicted through empirical models.

2. Modelling and Methodology

2.1. Storm Water Management Model (SWMM)

Storm Water Management Model (SWMM), developed by the United States Environmental Protection Agency is a dynamic rainfall-runoff simulation model. It is applicable for single event or long-term simulation of the quantity and quality of runoff originated mainly from urban areas (Rossman, 2015). SWMM can operate many hydrologic processes which produce runoff and is capable of hydraulic modelling by routing runoff and external inflows through a network containing nodes, pipes, storage units and a number of hydraulic designs. According to the conditions of each simulation, SWMM has three options for flow routing: steady flow, kinematic wave and dynamic wave routing. In cases of part filled conduits, all three routing methods employ the Manning equation (Eq. (1)).

$$Q = \frac{1}{n} A R^{2/3} J^{1/2} \qquad (1)$$

Where: Q $(m^3\ s^{-1})$ is the flow, n $(s\ m^{-1/3})$ is Manning's roughness factor, A (m^2) is the cross-sectional area of flow, R (m) is the hydraulic radius, and J $(m\ m^{-1})$ is the water surface slope.

When using steady flow routing, flow is considered steady and uniform at every time step of the simulation. Consequently, the inflow hydrographs are translated from the upstream end of a conduit to its downstream end with no delay or change in shape. This method is the simplest of the three routing methods, insensitive to the time step employed and is mostly recommended for a preliminary analysis. Kinematic wave routing considers unsteady and uniform flow. This method solves the complete Saint Venant continuity equation (Eq. (2)) and a simplified form of the Saint Venant momentum equation (Eq. (3)) for each conduit, where it is assumed that the slope of the water surface equals the conduit slope. Inflow hydrographs are delayed and attenuated because of the routing through the conduit. Both kinematic wave and steady flow routing method are limited only to dendritic networks (a single outflow for each node). This method gives sufficient results for long-term simulations.

$$\frac{dA}{dt} + \frac{dQ}{dx} = 0 \qquad (2)$$

$$\frac{dQ}{dt} + \frac{d(Q^2/A)}{dx} + g\ A\ \frac{dH}{dx} + gAS_f + gAh_L = 0 \qquad (3)$$

Where: Q $[L^3\ T^1]$ is the flow rate, A $[L^2]$ is the cross-sectional area of flow, x $[L]$ is the distance, t $[T]$ is the time, H $[L]$ is the hydraulic head of water in the conduit, S_f $[L\ L^{-1}]$ is the friction slope (head loss per unit length), h_L $[L]$ is the local energy loss per unit length of conduit, and g $[L\ T^{-2}]$ is the acceleration of gravity. Dynamic wave routing assumes unsteady and varied flow and solves the complete Saint Venant equations (Eq. (2), (3)), in addition with a volume continuity equation at nodes, which calculates the change in hydraulic head at the node with respect to time (Rossman, 2006, Rossman, 2015). In contrast to the other two routing methods, this method can be used for channel storage, backwater, entrance or exit losses and flow reversal, therefore it is appropriate for every type of network layout. It is also applicable to networks with pressurized pipes. Dynamic wave routing produces the most accurate results, however it requires smaller time steps and is more time-consuming than the other routing methods.

2.2. Methodology description

The whole project of optimal placement of SM units, while taking into account the hydrogen sulfide build-up inside the sewer pipes, is based on three steps (Tsoukalas *et al*., 2016). The first step includes the collection of data concerning the network topology, layout and characteristics. Knowledge about the extent of the network, its assets, its connection with other networks or treatment facilities, as well as about the flow direction, is also gathered. The nodes suitable for the possible placement of the units can be found included within the limits set by a buffer zone (here, with a width of 10 m) around each green area. In dendritic layouts, the path from each node to the exit node is unique. Moreover, information about the hydraulic characteristics of the network pipes and nodes (diameter, slope, elevation etc.) is collected. Land uses for the identification of areas that benefit from sewer mining are determined at this stage. The second step includes the implementation of Monte-Carlo simulations, which are stochastic processes based on the use of random numbers. The aim is to take into concideration the uncertainties in the model results due to input data uncertainties. Furthermore, Monte-Carlo simulations allow for the expression of the final results through the definition of appropriate probabilistic functions and metrics. Examples of input parameters with uncertainties in a sewer network are the peaking factors related to daily and hourly flow and BOD_5 loading fluctuations. After these parameters have been identified, random values, selected from a probability distribution (e.g. uniform distribution) are given to them. The process continues with N simulations of the model and the calculations of the quantities of interest (flow rate, velocity, hydraulic depth, BOD_5 concentration) for each pipe. Another option, instead of running Monte-Carlo simulations, is that of considering separate scenarios for various loading conditions (worst, middle or high), where proper values are assigned to uncertain input parameters. The processing of the final results from the simulations is carried out in the third and final step. The quantification of hydrogen sulfide generation for each pipe, and afterwards for the chain of pipes forming the path from each node to the exit node, is conducted using metrics (e.g. utility functions, risk functions) or relationships derived from literature. Regarding the optimal placement of SM units, a multi-criteria optimization is carried out. An example of criteria for this optimization is the minimization of hydrogen sulfide production inside the pipes, along with the maximization of the size of the area that benefits from sewer mining. Finally, since the optimization refers to conflict criteria, the most fit solutions can be found on a Pareto front and correspond to the optimal possible locations for the units.

2.3. Calculation of design discharge

The total design discharge equals the sum of sewage discharge (Q_s) and dry weather flow (Q_{DWF}). In this study, sewage discharge is calculated using the following equation (Koutsoyannis, 2011):

$$Q_s = \frac{qE}{86400}\lambda_L\lambda_S\lambda_1\lambda_2 \qquad (4)$$

Where: Q_s $(l\ s^{-1})$ is the sewage discharge, q $(l\ d^{-1}\ cap^{-1})$ is the indicative daily water consumption per capita, E (cap) is the serviced population, λ_L $(-)$, is a loss coefficient of the water distribution network, λ_S $(-)$ is a coefficient about the water percentage that ends up in the sewage network through runoff, λ_1 $(-)$ is a seasonal coefficient and λ_2 $(-)$ is a coefficient of peak discharge. Dry weather flow, Q_{DWF} $(l\ s^{-1})$, can be estimated with respect to sewage discharge, as shown in Eq. (5), where λ_{DWF} $(-)$ is a dry weather coefficient:

$$Q_{DWF} = Q_s\lambda_{DWF} \qquad (5)$$

The values of coefficients λ_L, λ_s, λ_1 and λ_2 depend on numerous factors. Some examples are the type and age of the network, the population and the standards of living. It should also be noted that fluctuations of the design flow during the day are taken into account by using appropriate daily patterns.

2.4. Quantification of hydrogen sulfide production

In literature, there are several approaches addressing the estimation of hydrogen sulfide concentration in the sewage (see Boon and Lister, 1975, Lahav *et al*., 2006, Pomeroy and Parkhurst, 1977, Yongsiri *et al*., 2005). In this study, 'Z formula' (Eq. (6)), is employed for the quantification of the possibility of H_2S build-up inside each pipe (Bielecki and Schremmer, 1987, Pomeroy, 1990).

$$Z = \frac{0.3\ 1.07^{T-20}[BOD_5]_i}{J_i^{1/2} Q_i^{1/3}} \frac{P_i}{B_i} \qquad (6)$$

Where: i is the pipe index, T *(^oC)* is the sewage temperature, BOD_5 *(mg l^{-1})* is the concentration of Biochemical Oxygen Demand of 5 days, J *(m m^{-1})* is the pipe slope, Q *(m^3 s^{-1})* is the discharge, P *(m)* is the wetted perimeter of the pipe wall and B *(m)* is the surface width of the stream. Additionally, velocity inside each pipe must be greater than or equal to a certain threshold, V_{min} *(m s^{-1})*, given by Eq. (7) (Koutsoyannis, 2011).

$$V_{min,i} = \frac{1.07^{T-20}[BOD_5]_i}{590} \qquad (7)$$

In case there is a chain of pipes, instead of a single pipe, Z indicator can be calculated as shown in Eq. (8).

$$MZ_C = \sum_{i=1}^{n} a_i Z_i \qquad (8)$$

Here, a_i is a weight coefficient for pipe *i*. In this study, a_i coefficients are calculated from the formula, $a_i = L_i/L_{tot}$, where L_i is the length of pipe *i*, and L_{tot} is the total length of pipes of the chain $i=1,...,n$ (Tsoukalas *et al*., 2016). The equation above is employed for each path from every node to the exit node, for the N simulations conducted. It is therefore possible to express the results using a quantile. Here, $Q[MZ_c]_{75}$, which stands for the quantile for probability 75 %, is calculated. This means that 75 % of all MZ_c values from the N simulations are less than or equal to the $Q[MZ_c]_{75}$ value. After this process follows the multi-criteria optimization. One criterion is the minimization of H_2S generation inside the pipes (expressed through lower values of $Q[MZ_c]_{75}$) and the other is the maximization of the water amount provided for the irrigation of green areas (expressed through the area size of each park). Other approaches are also applicable, as well as the use of data regarding the actual water demand from each green area. The optimal nodes for the placement of SM units are shown on a Pareto front. For a more in-depth approach, Pomeroy formula concerning the total sulfide concentration inside the pipes is also employed (Eq. (9)) (Pomeroy and Parkhurst, 1977). In order to avoid critical conditions, total sulfide concentration inside each pipe must be less than *1 mg l^{-1}* (Koutsoyannis, 2011), thus only the paths including pipes where this limit is not exceeded are accepted.

$$\frac{d[S]_i}{dt} = \frac{M\ 1.07^{T-20}[BOD_5]_i}{R_i} - \frac{m(J_i V_i)^{3/8}[S]_i}{d_i} \qquad (9)$$

Where: i is the pipe index, S *(mg l^{-1})* is the total sulfide concentration in the aqueous phase, t *(h)* is retention time, R *(m)* is the hydraulic radius, V *(m s^{-1})* is the velocity of the stream and d *(m)* is the mean hydraulic depth. M and m are empirical constants which take the values 0.32 x 10^{-3} m h^{-1} and 0.64 (s m^{-1})$^{3/8}$ h^{-1} respectively, in cases of partly filled pipes. BOD_5, T and J are the same as defined in Eq. (6).

3. Case study

3.1. Study area

The case study is based on a future sewer network located in Kalyvia Thorikou, east Attica, Greece. It is part of a larger engineering project of Saronikos municipality concerning the extention of the existing sewer network of coastal zone. This network serves an area of approximately 118 ha, which consists of 98 ha of residential areas, 1 ha of sports facilities and 19 ha of green areas (Figure 1). It has a population of ~ 10000-15000 people. The network cosists of 1031 nodes (one of them being the exit node) and 1030 pipes, with diameters ranging between 0.2 and 0.5 meters and slopes ranging between 2 ‰ and 150 ‰ (average slope: 35 ‰). The total length of the network is about 38 km.

3.2. Problem setting

In this study, the design period of the sewer network is set to *T=40* years. The design population is estimated using the formula $N_t = N_o\ (1+r)^t$, where N_o is the current population, r is the increase rate (assumed here 1.5%) and t is the extrapolation year ($t=0,...,T$). Regarding Eq. (4), q is set to *250 l cap^{-1} d^{-1}* for $t=0$ and *300 l cap^{-1} d^{-1}* for $t=40$ years. Additionally, coefficients λ_L and λ_S are *0.725* and *0.625* for $t=0$ and *0.850* and *0.650* for t=40 years, respectively. For intermediate years, their values can be calculated using linear interpolation. Parameters λ_1 and λ_2 are deemed uncertain and their values are selected through Monte-Carlo selection from uniform distribution. BOD_5 mass loading is varied using five scenarios: *40*, *45*, *50*, *55*, *60* and *65 g cap^{-1} d^{-1}*. A pattern derived from literature (Linsley *et al*., 1992), is altered and adapted according to local conditions, and then used to address daily flow and BOD_5 loading fluctuations. Regarding SWMM model, each analysis lasts 24 hours and includes the modelling of flow routing and water quality. Kinematic Wave routing is chosen as flow routing method and the routing step is set to 30 seconds. MATLAB is used for all calculations and the conducting of the SWMM model simulations as well. The results of interest are stored in MATLAB for further evaluation.

4. Results and discussion

The final results are shown on a Pareto front (Figure 2), as mentioned previously. Red dots represent the optimal areas for the placement of SM units (areas with ID 3 and ID 22), while blue dots represent the areas which have been discarded. In the former case, these areas are selected as the two criteria of the optimization are simultaneously met, giving better results than the other alternatives.

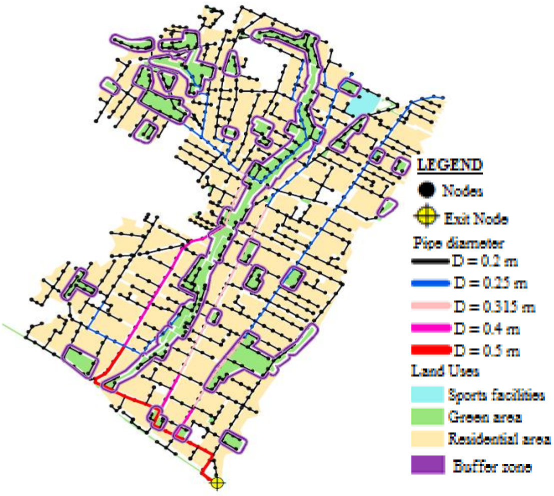

Figure 1. The case study sewer network in Kalyvia

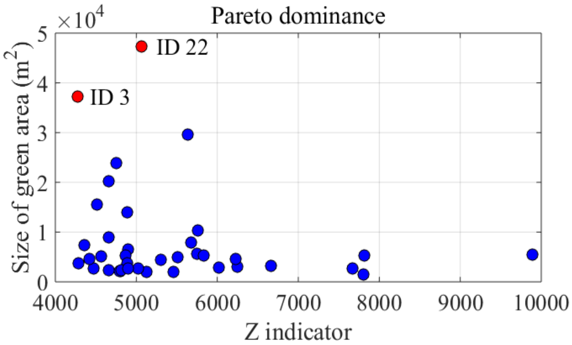

Figure 2. Pareto front obtained from the optimization

If emphasis is placed on the minimization of H_2S production, then area with ID 3 is more suitable for selection. The path from the optimal node for placement (in ID3) to the exit node has the lowest MZ_c value among all paths from the other nodes of ID3. The optimal path of ID3 is demonstrated on the map in Figure 3 (red line). Figure 4 illustrates the cross-section of the optimal path of ID3. X-axis includes all the pipes of the path which starts from pipe C215 (area with ID 3) and ends to pipe C122 (exit node). The first panel depicts the value of Z indicator for each pipe across the path. It is observed that all values are under the threshold of Z=7500, as is required. The second panel shows the probability of non-exceedance of the threshold value. Since non- exceedance probabilities are high (~90-100%), high reliability is achieved. The third panel presents the total sulfide concentration (in mg l^{-1}) in the stream across the optimal path, as derived from Eq. (9), which complies with the limit of 1 mg l^{-1} for every pipe. The H_2S concentration inside each pipe is calculated using the quantile values for every parameter of Eq. (9).

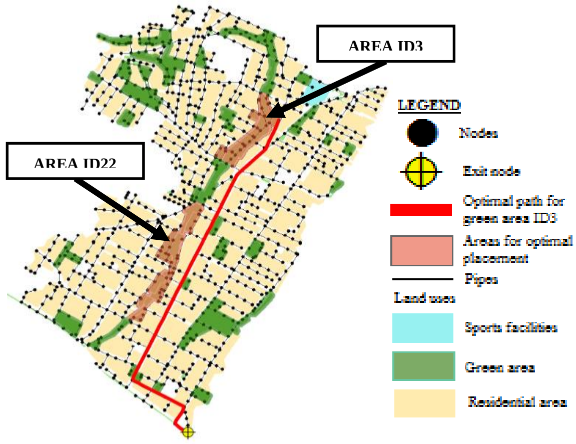

Figure 3. Proposed areas for optimal placement of sewer mining units

5. Conclusion

Sewer mining is an innovative technology which can sufficiently provide recycled water of good quality, alleviating the problem of water shortage in numerous cases. Nonetheless, disruptions of the processes inside existing sewer networks can occur, hence careful planning and implementation of sewer mining is crucial. One important challenge concerning sewer mining applications is H_2S production inside the pipes, since it is responsible for problems related primarily to odour and corrosion. In this study, a Monte-Carlo based method, coupled with the use of SWMM 5.1 model, is proposed for optimal placement of SM units, in terms of minimizing H_2S generation inside the pipes.

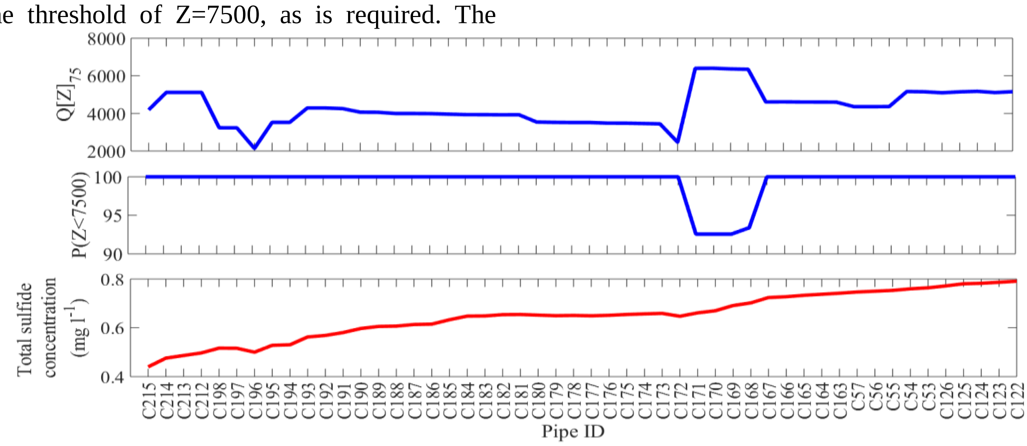

Figure 4. Cross-section of the optimal path. The first panel shows the quantile values of Z indicator across the pipes of the path. The second panel depicts the probability of non-exceedance of the threshold value of *Z*=7500 with respect to these pipes. The third panel shows the total sulfide concentration (mg l^{-1}) inside the pipes, according to Eq. (9).

The presented method takes into account both spatial and hydraulic characteristics regarding the sewer network. It indicates locations for sewer mining application, where potential intervention will have minimum effects to wastewater quality and quantity. The results show that the proposed method could provide useful guidelines and can assist at the stage of setting up a sewer mining scheme. However, further study is needed for a more meticulous assessment of hydrogen sulfide concentration inside the stream and gas pressure in the atmosphere of the pipes.This also highlights the need for fixed guidelines describing the necessary procedures and limits of critical parameters.

Acknowledgements

The research leading to these results has received funding from the European Union Seventh Framework Programme under grant agreement no 619039 (ENV.2013.WATER INNO&DEMO-1), for the research project DESSIN "Demonstrate Ecosystem Services Enabling Innovation in the Water". The research and its conclusions reflect only the views of the authors and the European Union is not liable for any use that may be made of the information contained herein.

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