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Optimal Control of Combined Sewer Overflows

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Abstract: Combined sewer networks carry wastewater and stormwater together. Capacity limitation of these sewer networks results in combined sewer overflows (CSOs) during high-intensity storms. Untreated CSOs when directly discharged to the nearby natural water bodies cause many environmental problems. Controlling existing urban sewer networks is one possible way of addressing the issues in urban wastewater systems. However, it is still a challenge, when considering the receiving water quality effects. This paper presents an evolutionary constrained multi-objective optimization approach to control the existing combined sewer networks. The control of online storage tanks was taken into account when controlling the combined sewer network. The developed multi-objective approach considers two important objectives, i.e. the pollution load to the receiving water from CSOs and the cost of the wastewater treatment. The proposed optimization algorithm is applied here to a realistic interceptor sewer system to demonstrate its effectiveness.

Key words: Combined sewer systems, effluent quality index, genetic algorithms, constrained evolutionary multi-objective optimization, on-line storage tanks.

1. Introduction

Combined sewer networks carry wastewater and stormwater together. Capacity limitation of these sewer networks results in combined sewer overflows (CSOs) during high-intensity storms. CSOs are one of the leading causes of the water pollution in natural water bodies. Constructing additional storage facilities, increasing conduit capacity, expanding pumping capacity and application of controlling strategies to utilize the existing storage in sewer network are the common mitigation solutions of CSOs. Controlling existing urban sewer networks is an interesting solution, when considering the limited availability of space and funds. However, it is still a challenge, when considering the receiving water quality effects.

Most of the literature on controlling combined sewer systems is based on volumetric measures [1-3]. However, they failed to address the issue of water quality in both combined sewers and receiving waters.

In addition, some previous work was carried out with simplified hydraulic models [4]. This is due to the complexity of the problem. Furthermore, the cost of wastewater treatment at downstream wastewater treatment plant, in general was not considered previously in the literature.

However, a multi-objective optimization approach in controlling combined sewer systems based on the receiving water quality due to CSOs and downstream wastewater treatment cost has been presented by Rathnayake and Tanyimboh [5-6]. More importantly, this approach was based on the full dynamic unsteady sewer flows.

This paper presents an improvement to the approach presented in Rathnayake and Tanyimboh [5-6]. Storage tanks proposed in the combined sewer network in Rathnayake and Tanyimboh [5-6] were off-line storage tanks. By contrast, on-line storage tanks are proposed in this paper as an alternative and the flow control in these on-line storage tanks is presented. A multi-objective optimization approach based on the pollution load to the receiving water from CSOs and the cost of wastewater treatment is

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proposed. The performance of the optimization approach developed is demonstrated here on an interceptor sewer system with promising results.

2. Problem Formulation and Solution

Schematic diagram of an interceptor sewer system and a combined sewer chamber are presented in Fig. 1. Inflows from catchments' dry weather flow (DWF) and stormwater runoffs (I_i) are introduced to CSO (combined sewer overflow) chambers. Depending on the capacity of the sewer chambers and the interceptor sewer, CSOs (O_i) occur.

The first objective function (F_I) was formulated to minimize the pollution load from CSOs to the receiving water. Effluent quality index (*EQI*) was used to formulate this pollution load. The *EQI* is an integration of five important water quality parameters, including total suspended solids (*TSS*), chemical oxygen demand (*COD*), five-day biochemical oxygen demand (*BOD*), total Kjeldahl nitrogen (*TKN*) and nitrates/nitrites (*NOX*). A detailed explanation of this EQI can be found in Rathnayake and Tanyimboh [5-6]; Rathnayake [7]. The formulation of the first objective function is given in the following equation (Equation 1).

$$Minimize\ F_1 = \sum_{i=1}^{n} P_i \tag{1}$$

where n and P_i are the number of interceptor nodes or CSO chamber points and the pollution load to the receiving water from the i^{th} CSO chamber respectively. P_i can be expressed as

$$P_i = EQI_i \tag{2}$$

whereEQI_i is the effluent quality index at node i.

The second objective function was formulated to minimize the wastewater treatment cost at downstream wastewater treatment plant (Equation 3).

$$Minimize\ F_2 = C_T \tag{3}$$

where C_T (€/year)is the treatment cost at treatment plant. This C_T is expressed as a function of the wastewater volume flow rate (q_T) to wastewater treatment plant (Equation 4).

$$C_T = \begin{cases} 1642353.082 \times {q_T}^{0.659}, & q_T \le 3 \times DWF \\ A + \dfrac{2}{3}B, & 6 \times DWF \ge q_T \ge 3 \times DWF \\ A + \dfrac{2}{3}C, & q_T > 6 \times DWF \end{cases} \tag{4}$$

where

$$A = 1891.154 \times DWF^{0.659} \tag{5}$$

$$B = 1.69 \times \{(q_T - 3 \times DWF) + 11376\} \tag{6}$$

$$C = 5.07 \times DWF + 11376 \tag{7}$$

where q_T (m^3/s) is the treated wastewater volume flow rate. A detailed explanation on the derivation of this generic cost function is given in Rathnayake and Tanyimboh [5-6]; Rathnayake [7].

With reference to Figure 1, the following continuity equations can be listed.

$$Q_i + q_{i-1} - q_i = 0 \tag{8}$$

$$A_C \frac{\Delta h_C}{\Delta t} = I_i - Q_i \ ; \ h_C < h_S \tag{9}$$

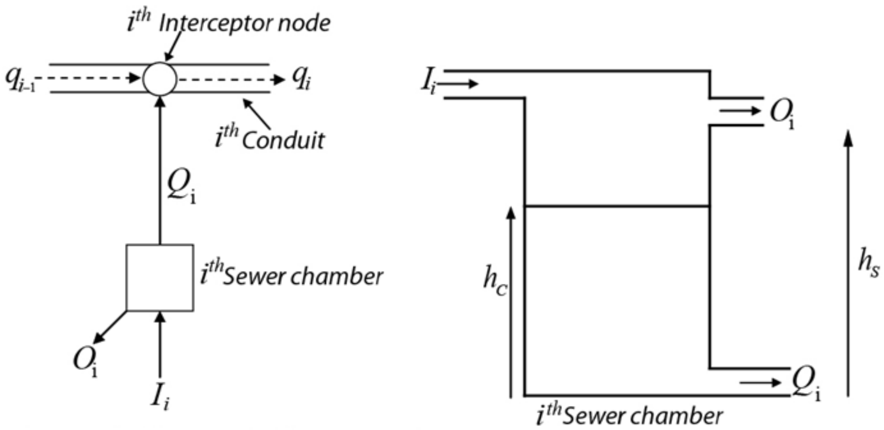
$$A_C \frac{\Delta h_C}{\Delta t} = I_i - Q_i - O_i \ ; \ h_C > h_s \tag{10}$$

$$0 \le q_i \le q_{\max,i} \tag{11}$$

where A_C is the surface area of the CSO chamber and $q_{max,i}$ is the maximum flow rate at i^{th} conduit.

Figure 2 shows the schematic diagram of an on-line storage tank. q_s and h_{ST} are flow to the storage tank from CSO chamber and the water level of the storage tank, respectively. When the water level of the sewer chamber (h_c) reaches the spill level of the chamber (h_s), the storage tank starts filling. Flow to the storage tank (q_s) stops when the storage tank reaches its maximum capacity. This will then lead to CSOs through the corresponding CSO chamber. These controls are formulated inside the hydraulic simulation model by using the control rules.

U.S. EPA SWMM 5.0 [8] is a powerful hydraulic and water quality simulation model, which is capable



I_i – Catchment inflow to node *i*
Q_i – Flow from i^{th} sewer chamber to interceptor node *i*
q_i – Through flow in interceptor sewer at node *i*
Oi – Combined sewer over flow discharge at node *i*
h_C – Water level in sewer chamber
h_S – Spill level of sewer chamber

Fig. 1 Schematic diagram of interceptor sewer system.

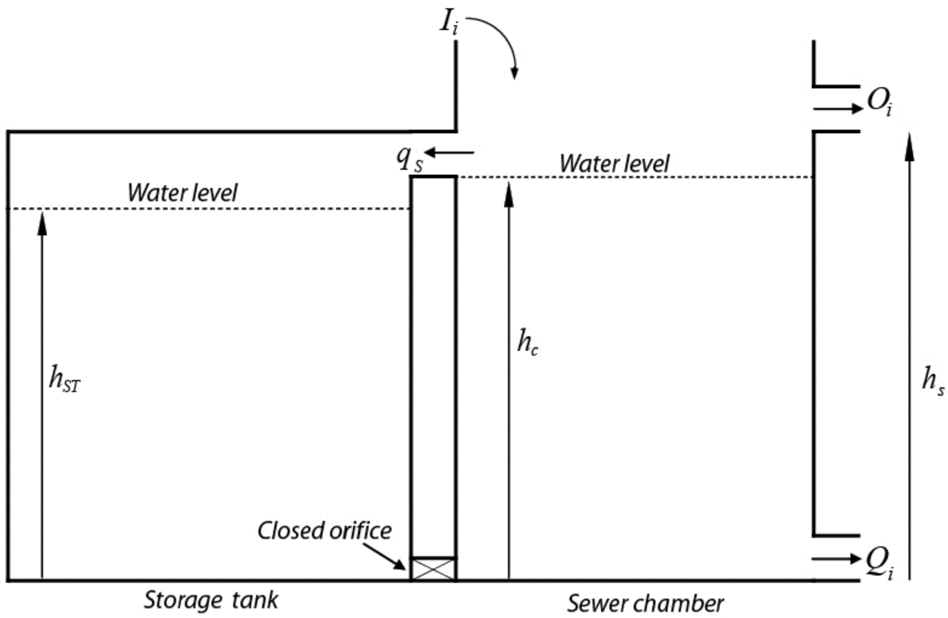


Fig. 2 Schematic diagram of sewer chamber with on-line storage tank.

of simulating stormwater runoff and routing processes, including water quality routing. SWMM 5.0 was coupled with NSGA II [9] using C programming language. The Non-dominated Sorting Genetic Algorithm (NSGA II) is a multi-objective optimization algorithm that has been applied successfully to many practical optimization problems. This includes solving optimization problems in urban wastewater systems [10-12].

The wastewater flow from a particular CSO chamber to the interceptor sewer is assumed to be controlled by a rectangular orifice at the bottom of the CSO chamber. The orifice openings were initially generated randomly. Then, a full hydraulic simulation, including water quality routing was carried out using SWMM 5.0.The results from this hydraulic and water quality simulation were used to calculate the two objective functions (F_1&F_2) stated above (Equations 1&3).

The hydraulic simulation model SWMM 5.0 automatically satisfies the mass balance and the conservation of energy of the system. However, the flow rates in interceptor sewer, described at Equation 11, were satisfied in the multi-objective optimization algorithm with a tournament constraint handling technique [9]. It uses a binary tournament selection, where two potential solutions are picked at random from the population and the better solution is selected. More details of this constraint handling technique are found in Deb *et al.* [9].

3. Case study

The interceptor sewer system described in Thomas [13] was modified for this study as explained in Rathnayake and Tanyimboh [5-6]; Rathnayake [7]. Figure 2 shows the modified interceptor sewer system. The CSO chambers T1 to T7are described in Thomas [13]. Two on-line storage tanks (T8 and T9) were introduced at the T2 and T5 CSO chambers [7]. As stated in the preceding section, the on-line storage tanks were controlled to store wastewater. These controls were applied in the hydraulic simulation model by using the control rules in SWMM. Maximum flow rate allowed through C1, C2 and C3is 3.26 m^3/s and that of C4, C5, C6 and C7is 7.72 m^3/s. The diameter for C1 to C3is 1.66 m and that of C4 to C7is 2.44 m. Depth of the CSO chambers (T1 to T7) and storage tanks (T8 and T9) are 5.42, 6.91, 7.95, 8.04, 8.18, 8.47, 9.26, 6.91 and 8.18 m, respectively.

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Average dry-weather flow rates for foul or sanitary wastewater were assumed for the T1 – T7 CSO chambers without the diurnal variations together with the flow hydrographs for a single storm. More details

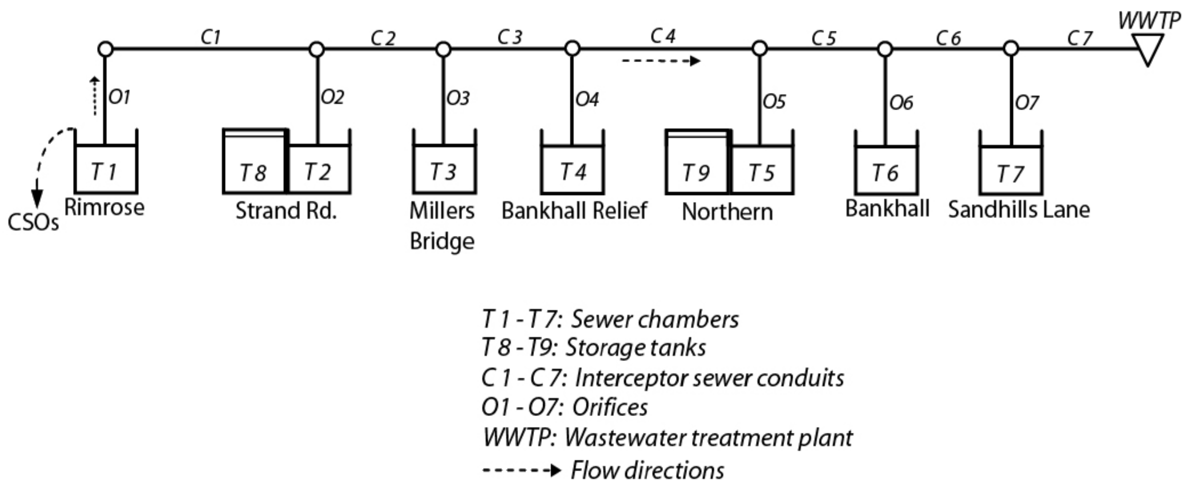

Fig. 3 Generic interceptor sewer system.

on these hydrographs are available in Thomas [13]. Furthermore, the pollutographs adopted for the five different water quality constituents included in the effluent quality index (i.e. total suspended solids; chemical oxygen demand; biochemical oxygen demand; nitrates and nitrites; and total Kjeldahl nitrogen) were based on diverse hypothetical land-uses including residential, industrial, commercial, agricultural and mid urban. More details are found in Rathnayake and Tanyimboh [14]; Rathnayake [7].

With NSGA II, the optimization was done with a population of 100; 100 generations; real coding; polynomial mutation probability of 0.4; simulated binary crossover probability of 1; and distribution indices for crossover and mutation of 20 each, respectively. Many optimization runs with different random seeds were conducted. Routing time-step in SWMM 5.0 was kept at 30 seconds, and the results were obtained at 15 minutes. Each optimization run took about 9 minutes on a Pentium 4 desktop personal computer with a 2.7 GHz Core 2 Duo processor and 4 GB of RAM.

4. Results and Discussion

Figure 4 shows the achieved Pareto optimal front for a storm whose duration is 15 minutes. Solutions A_{T1} to H_{T1}along the Pareto optimal front were selected for further assessment. Solution A_{T1} gives the minimum pollution load to receiving water. In other words, it has the maximum wastewater treatment cost. On the other hand, solution H_{T1} gives the minimum wastewater treatment cost at downstream wastewater treatment plant and has the maximum pollution load to the receiving water.

Figure 5 is quoted from Rathnayake and Tanyimboh [6] and presented to compare the results against the new approach.

It can be clearly seen that the Pareto optimal curve has moved to its left in the X axis. In other words, the improved approach produces better results than that of Rathnayake and Tanyimboh [6]. The reduction of pollution load in new approach is mainly due to the introduction of online storage tanks and their optimal control.

Orifice openings for these selected solutions (A_{T1} – H_{T1}) were obtained and the hydraulic simulations were carried out accordingly. Results from full hydraulic simulations for these selected solutions (A_{TI} – H_{T1}) are presented in the following tables (Tables 1 to 3).

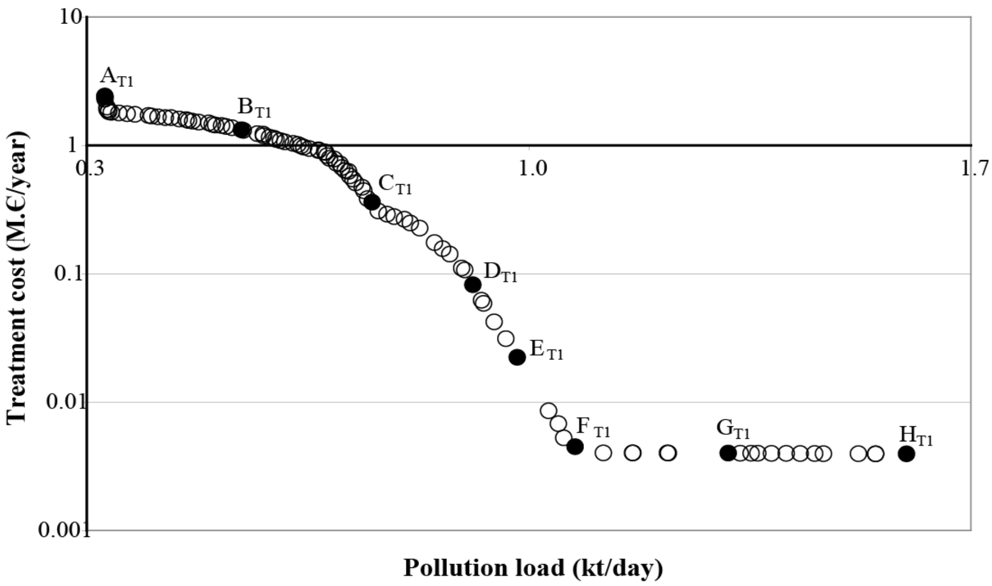

Fig. 4 Pareto optimal front achieved for 15 minutes

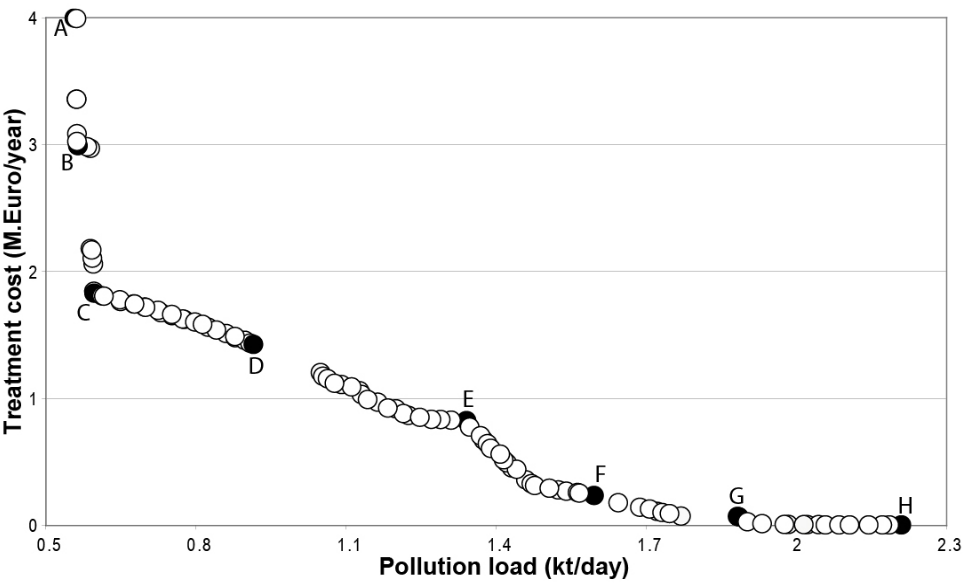

Fig. 5 Pareto optimal front for optimal control without the online storage tanks [6].

Table 1 Flow rates in interceptor sewer after 15 minutes for selected solutions.

Solutions	Interceptor sewer flow rates(m^3/s)						
	C1	C2	C3	C4	C5	C6	C7
A_{T1}	2.74	1.65	3.25	5.85	4.89	3.02	1.80
B_{T1}	2.76	1.61	1.83	3.68	3.00	1.61	0.72
C_{T1}	2.72	1.62	3.21	3.16	2.28	0.65	0.10
D_{T1}	2.75	1.65	2.20	2.08	1.25	0.21	0.01
E_{T1}	2.77	1.59	1.65	1.48	0.73	0.07	0
F_{T1}	2.76	1.66	0.73	0.35	0.03	0	0
G_{T1}	2.44	1.41	0.41	0.11	0	0	0
H_{T1}	0.39	0.10	0.01	0	0	0	0

Table 1 gives the flow rates through the interceptor sewer sections at 15 minutes for solutions A_{T1} to H_{T1}. As stated in the preceding (Case Study) section the flow rates through sewer conduits were constrained to the respective maximum flow rates. Maximum flow rate allowed through C1, C2 and C3 is 3.26 m^3/s and that of C4, C5, C6 and C7 is 7.72 m^3/s. It can be clearly seen in Table 1 that the flow rates through these conduits are less than the maximum allowed flow rates for all the tabulated cases. This observation shows that the developed multi-objective optimization approach produces feasible solutions.

Table 2 shows the CSO discharges at 15 minutes for solutions A_{T1} to H_{T1}. Solution A_{T1} that corresponds to the minimum pollution load to receiving water has the smallest CSO discharges while Solution H_{T1} that corresponds to the minimum wastewater treatment cost has the largest discharges. Table 2 reveals a consistent pattern that suggests the proposed optimization model yields satisfactory results.

Table 3 exhibits the wastewater depths at CSO chambers and storage tanks for Solutions A_{T1} to H_{T1}. Wastewater depths highlighted in grey in Table 3 correspond to the respective maximum depths for the tanks and CSO chambers. These highlighted wastewater depths are entirely consistent with the CSO discharges seen previously in Table 2. For example, chamber T3is full for all solutions (A_{T1} to H_{T1}). Accordingly, Table 2 shows that discharges to the receiving water occur at chamber T3 for all solutions. Another interesting observation can be made for chamber T5 for solutions F_{T1} to H_{T1}. Table 2 shows solutions

Table 2 Combined sewer overflows at time *t*=15 minutes for selected solutions.

Solutions	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
A_{T1}	0	0	2.25	0	0	0	0
B_{T1}	0	0	3.72	0	0	0	0
C_{T1}	0	0	2.42	4.26	0	0	0
D_{T1}	0	0	3.44	4.36	0	0	0
E_{T1}	0	0	3.92	4.36	0	0	0
F_{T1}	0	0	4.54	4.37	0	0	0
G_{T1}	1.98	0	4.74	4.36	0	0	0
H_{T1}	4.49	0	4.74	4.36	0	0	0

Table 3 Wastewater depths in the chambers at time *t*=15 minutes for selected solutions.

Solution	Wastewater depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
A_{T1}	5.26	6.26	7.95	5.53	8.17	7.17	7.63	1.75	7.42
B_{T1}	5.19	6.26	7.95	7.98	8.17	7.18	7.63	1.71	7.4
C_{T1}	5.34	6.26	7.95	8.04	8.17	7.18	7.63	1.76	7.42
D_{T1}	5.29	6.26	7.95	8.04	8.17	7.18	7.63	1.71	7.43
E_{T1}	5.22	6.26	7.95	8.04	8.17	7.18	7.63	1.71	7.34
F_{T1}	5.16	6.26	7.95	8.04	8.18	7.18	7.63	1.76	7.4
G_{T1}	5.42	6.26	7.95	8.04	8.18	7.18	7.62	1.78	7.24
H_{T1}	5.42	6.26	7.95	8.04	8.18	7.18	7.62	1.79	7.24

F_{T1} to H_{T1}have no CSO discharges at chamber T5 that is full as Table T3 shows. In fact, this is due to the T9 on-line storage tank that is not full. In addition, it can be seen in Table 3 that there is wastewater in theT8storage tank and, moreover, tank T2 is not full. Consequently there are no CSO discharges at the T2 chamber as Table 2 shows, for all solutions A_{T1} to H_{T1}.

5. Conclustions

The work presented in this paper shows a considerable improvement in controlling urban wastewater systems compared to the previous work by the same authors [5-6] and the hydraulic simulations post optimization show that the solutions are feasible. The proposed model gives optimal control settings for the combined sewer overflows where a single set of static controls is used throughout the storm duration of 15 minutes. Further research is required to develop a holistic dynamic optimization procedure for the full duration of a storm.

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