Study on purification mechanism in soil penetration facility for effluents from urban area and control strategies

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Abstract In this study, demonstration experiments for removal of pollutants from road surface runoff during storm events were carried out under natural conditions in an outdoor pilot-scale soil penetration facility. In general, soil retains suspended matter and removes dissolved matter by adsorption. However, issues such as reduced purification capacity resulting from clogging and recovery of purification capacity during periods of intermittent supply of the storm water affect the removal efficiency of pollutants. Therefore, this study aimed at clarifying purification mechanisms during storm events and understanding how the structural characteristics of the soil penetration facility affect purification capacity based on long-term continuous measurements. In addition, modeling the purification mechanism under changing characteristics of rainfall in the long-term was undertaken.

Keywords Infiltration capacity; mass balance model; road surface runoff; soil penetration facility; soil purification mechanisms; time-series model

Introduction

Lake Biwa is an important source of water for about 14 million people in the Kansai Metropolitan Region. The lake has undergone tremendous pollution since the period of high economic growth in Japan that started in the 1950s. The water quality of the lake improved to some extent in the 1980s due to a number of measures taken such as formulation of the Basic Law for Environmental Pollution Control and the setting of Environmental and Effluent Standards. However, the water quality has not significantly improved and still does not meet the Environmental Standards (Figure 1).

The pollutant load entering the lake as estimated by Shiga Prefectural Government is shown in Table 1. The pollutant load from urban and industrial point sources decreased between 1995 and 2000. However, the pollutant load from the non-point sources shows an increasing trend. The non-point pollutant load from land, especially from urban areas, has increased due to increased development of urban areas. Therefore, measures for reduction of pollutant load in road surface runoff during storm events have recently been emphasized. As one of the specific measures, treatment of road surface runoff by soil penetration was investigated in this study. The characteristics of road surface runoff and soil penetration purification during storm events are shown in Table 2.

In this study, continuous pilot-scale demonstration experiments were carried out to investigate the removal of pollutant load from road surface runoff using a soil penetration facility. In earlier studies it was shown that characteristics of pollutant runoff from road surface were strongly influenced by amount of precipitation, rainfall intensity and antecedent days (Sugihara *et al.*, 2000; Nishikawa *et al.*, 2001; Nishikawa *et al.*, 2002). These studies reported average pollutant removal efficiencies of 80%, 50% and 40% for COD,

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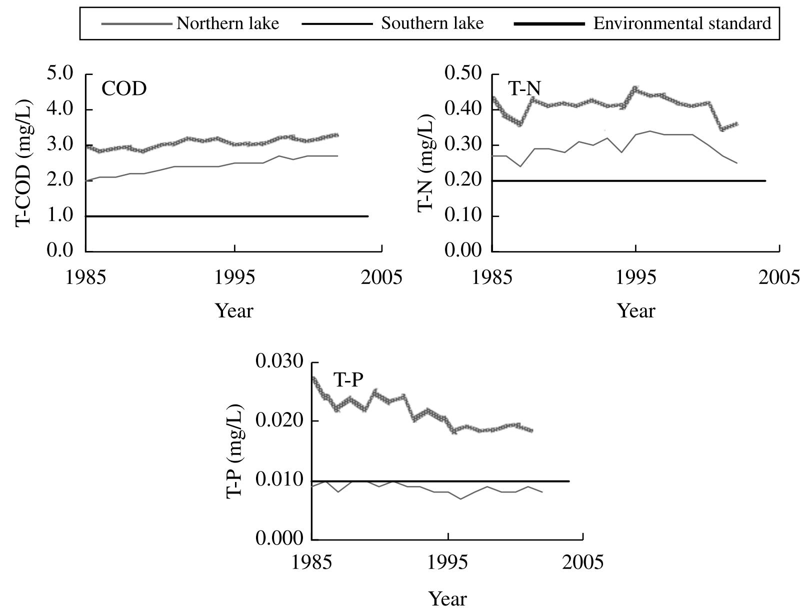

Figure 1 Trend of water quality of Lake Biwa, Shiga Prefectural Government (2004)

T-N and T-P, respectively. The studies further reported that clogging occurs in the 5 cm surface layer of soil and that the removal efficiencies of pollutants could be drastically increased by using an improved type of soil. As a continuation of the studies referred to above, the current study focused on understanding the mechanism of purification of soil and the effect of structural characteristics of the soil penetration facility on the purification capacity. The specific objectives of the study were: 1) to examine the purification mechanism of soil in relation to precipitation characteristics, 2) to model the purification mechanism of soil in the long term.

Outline of survey

Outline of soil penetration facility

A series of penetration experiments were continuously carried out at an experimental field located in Kusatsu City over a period of two years and two months. The outline of the soil penetration facility is shown in Figure 2 and Table 3. Runoff collected from 750

Table 1 Pollutant load going into Lake Biwa

		Point source		Non-Point source			
		Urban	Industrial	Agricultural	Natural	Land	Total
COD (t/day)	1995	16.0	11.5	7.5	15.8	5.4	56.2
	2000	11.5	7.1	7.0	15.8	5.8	47.2
	2005	9.3	6.6	7.0	15.7	6.1	44.7
T-N (t/day)	1995	5.7	3.4	3.4	7.7	1.4	21.6
	2000	5.5	2.5	3.2	7.7	1.5	20.4
	2005	5.4	2.3	3.1	7.7	1.6	20.1
T-P (t/day)	1995	0.52	0.37	0.18	0.21	0.07	1.35
	2000	0.43	0.25	0.16	0.21	0.08	1.13
	2005	0.38	0.23	0.16	0.20	0.09	1.06

Source: Shiga Prefectural Government (2004)

Table 2 Characteristics of purification by soil penetration for road surface runoff

Road surface runoff	Soil penetration
Irregular drainage	**Merit**
Intermittent drainage	Recovery of soil condition
Variation of flow	Pollutant removal
High concentration	Adsorption, Air exchange
SS, COD, T-P, T-N,	Microbe decompositon
Heavy metals and PAHs	Economical efficiency
	Low cost
Irregular pollutant load	**Demerit**
First flush	Sustainability
	Clogging

m^2 of road surface area was divided into two equal flows after which it flowed into each of the tanks in parallel. This water was called inflow water. Some of the inflow water penetrates the soil and is gathered by the drainage pipe at the bottom of the tank and then drained. This is called drained water. Some of the inflow water is retained in the soil and is referred to here as water held in soil. As the inflow water increases, some of the water flowing on the surface of the tank (surface flow) goes over the weir and is drained. This is called overflow water.

Outline of measurement

The facility was used continuously from September 2001 to November 2003. Over this period there were 253 rainfall events with a total precipitation of 3,500 mm. Measurement items were precipitation, flow rate and water quality. Precipitation was measured using a rain gauge installed near the experimental field. Runoff from the road surface was supplied to the field for all storm events. Effluent from the field was composed of surface flow and infiltrated flow. Water flow was gauged and water sampled for water quality analysis at three points for inflow, infiltrated flow and overflow. Precipitation was measured for all the 253 rainfall events that occurred over the period of the experiments. Water quality analyses were done for 14 of these rainfall events. The sampling interval for water quality analyses was 5 minutes during the early stage of runoff and peak discharge while a longer interval of about 30 minutes was used for other periods. Water quality was analyzed for SS, T-COD, D-COD, T-N, D-N, T-P, D-P and heavy metals.

Model

The basic concept of mass balance for runoff volume and pollutant load is shown in Figure 3. The mass balance of the facility was separated into two stages, namely, the

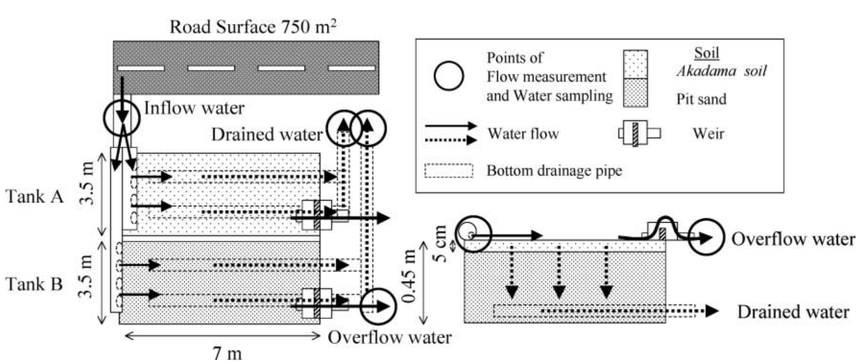

Figure 2 Soil penetration facility

Table 3 Outline of soil penetration facility

Tank	Surface area (m^2)	Depth (m)	Kind of soil (depth, m)		Height of weir
			Upper layer	Lower layer	(m)
A	25	0.45	*AKADAMA* soil (0.05)	pit sand (0.40)	0.03
B	25	0.45	pit sand (0.45)		0.03

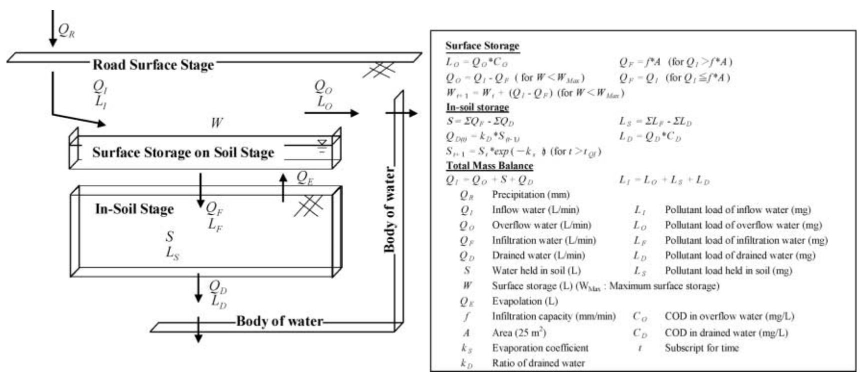

Figure 3 Basic concept of mass balance

surface storage stage representing storage of pollutants in water that flows on the surface of soil, and in-soil storage stage. The model parameters in Figure 3 were obtained statistically.

Results and discussion

Road surface stage

The characteristics of 50 rainfalls for which overflow occurred over the weir are shown in Table 4, while the characteristics of 14 rainfalls for which water quality was analyzed are given in Table 5.

Surface storage

Quantity of Water. The maximum amount of inflow water that penetrates through the soil corresponds to the infiltration capacity. When the inflow water exceeds the penetration capacity, the excess water is stored on the surface (this is referred to as surface storage). When the surface storage reaches the maximum surface storage capacity, excess water is discharged through the weir as overflow water.

The infiltration capacity was investigated during overflow conditions. Infiltration capacity was calculated by the following equation:

$$f = \Sigma Q_I / t / A \quad (1)$$

Table 4 Characteristics of rainfalls with overflow over the weir (50 rainfalls)

	Precipitation (mm)	Duration time (h)	Ave. rainfall intensity (mm/h)	Max. rainfall intensity (mm/h)	Antecedent days (day)
Ave.	25.5	9.47	4.69	24.48	2.83
Med.	20.0	8.42	3.00	18.00	1.34
Max.	131.0	41.83	24.00	111.00	17.49
Min.	3.5	0.17	0.41	3.00	0.25

Table 5 Characteristics of rainfalls with water quality analysis

Date	Precipitation (mm)	Duration time (h)	Ave. rainfall intensity (mm/h)	Max. rainfall intensity (mm/h)	Antecedent days (day)
2001					
9/10	24.5	18.55	1.32	15.00	2.39
10/9	52.5	14.17	3.71	39.00	5.25
11/29	7.5	10.33	0.73	12.00	22.43
2002					
3/5	52.0	15.67	3.32	18.00	5.18
7/13	6.0	3.00	2.00	21.00	2.32
9/6	14.5	8.83	1.64	15.00	8.64
9/26	5.5	3.50	1.57	6.00	9.28
10/15	5.5	0.50	11.00	18.00	7.17
11/1	33.5	10.50	3.19	6.00	8.06
2003					
5/30	19.5	8.00	2.44	9.00	4.10
6/16	20.0	6.00	3.33	18.00	0.61
7/29	31.5	17.00	1.85	18.00	5.91
10/13	15.5	8.67	1.79	15.00	17.49
11/3	20.5	20.00	1.03	21.00	11.80

The terms in Equation (1) are as defined in Figure 3. The variation of initial infiltration capacity (at 30 minutes from the start of overflow) and terminal infiltration capacity (at 30 minutes before overflow stopped) with time is shown in Figure 4.

From Figure 4, it was confirmed that the infiltration capacity decreased with time from the start of overflow and approached a constant value. This demonstrates the commencement of saturation of the soil. The relationship between the cumulative inflow water and infiltration capacity is shown in Figure 5. The following relationships were established:

$$\text{Tank A} \quad f = -1.82\ln(\Sigma Q_I) + 21.66\, R^2 = 0.52 \qquad (2)$$

$$\text{Tank B} \quad f = -1.96\ln(\Sigma Q_I) + 21.66\, R^2 = 0.73 \qquad (3)$$

The terms in Equations (2) and (3) are as defined in Figure 3.

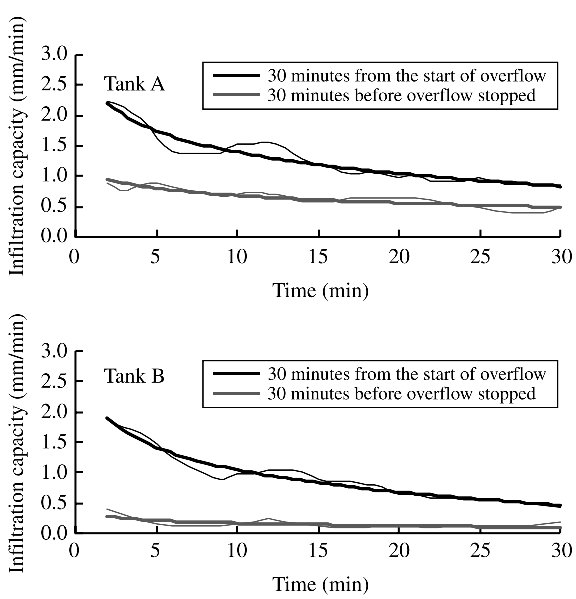

Figure 4 Variation of infiltration capacity with time

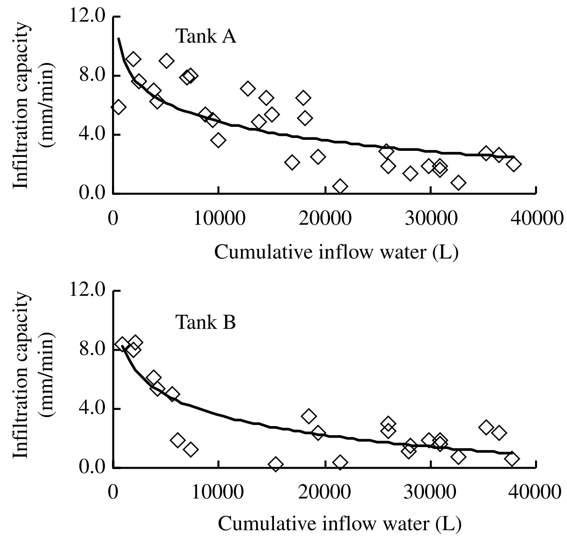

Figure 5 Variation of infiltration capacity with cumulative inflow water

Pollutant Load. Using results of actual measurement of the quality of overflow water, the pollutant load (expressed as COD) was simulated. The relationship between mean pollutant concentration of inflow water ($\Sigma L_I/\Sigma Q_I$) and the initial pollutant concentration in overflow water (C_{Fa}) and the terminal pollutant concentration in overflow water (C_{Ter}) is shown in Figure 6.

The following relationships were formulated:

Initial water quality

$$\text{Tank A} \quad C_{Fa} = 0.58x + 6.20 \; R^2 = 0.33 \tag{4}$$

$$\text{Tank B} \quad C_{Fa} = 0.63x + 6.34 \; R^2 = 0.38 \tag{5}$$

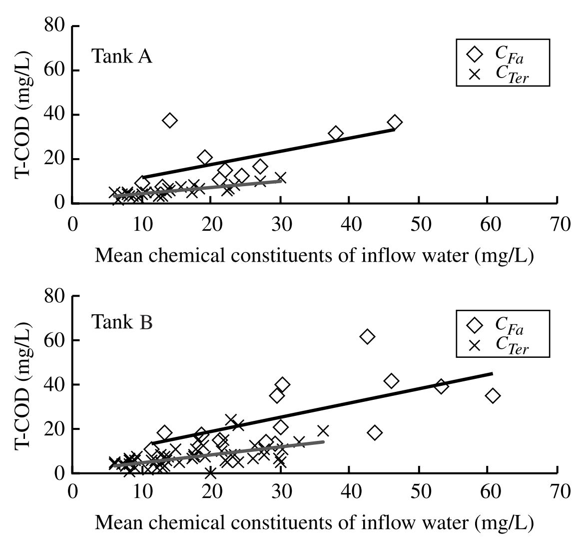

Figure 6 Concentration of pollutant in overflow water

Terminal water quality

Tank A $\quad C_{Ter} = 0.31x + 1.02\ R^2 = 0.69$ (6)

Tank B $\quad C_{Ter} = 0.36x + 1.47\ R^2 = 0.32$ (7)

In Equations (4)–(7), x is T-COD of inflow water (mg/L). Using the above results, the following relationship was established for COD in overflow water:

$$C_O = (C_{Fa} - C_{Ter}) * (1 - \exp(-k_{CO} * t) + C_{Ter} \quad (8)$$

In Equation (8), k_{CO} is a constant and other terms are as defined in Figure 3.

In-soil storage

Quantity of Water. As penetrated water infiltrates in the soil, drained water starts to appear depending on the amount of water held in the soil. Initially water held in soil increases with time. However, as penetrated water decreases, water held in the soil decreases. When inflow of water to the facility stops, water held in the soil continues to decrease due to water loss and drainage until water held in the soil reaches the amount of the water of adhesion on the soil. Using the mass balance for water volume (Figure 3), the maximum amount of water held in the soil for a given precipitation was determined. The relationship between cumulative penetrated water (at the time when maximum water held in soil occurs) and the maximum water held in soil is shown in Figure 7. The maximum water held in the soil (S_{Max}) for each of the tanks was determined as the maximum value of water held in the soil lying within the range of 2 standard deviations from the mean of the normal distribution curve. The following values of S_{Max} were obtained:

Tank A $\quad S_{Max} = 5,661\,(L)$ (9)

Tank B $\quad S_{Max} = 3,980\,(L)$ (10)

The soil condition in which water started to drain was determined from the relation between antecedent days (which affect the soil condition) and water held in the soil at the start of draining as shown in Figure 8.

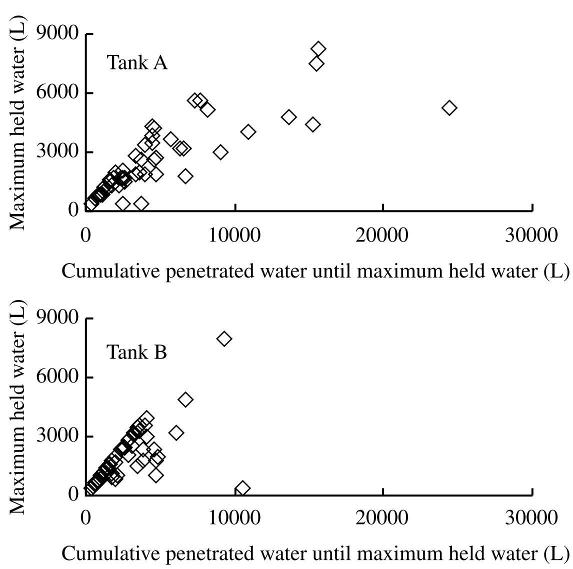

Figure 7 Maximum held water

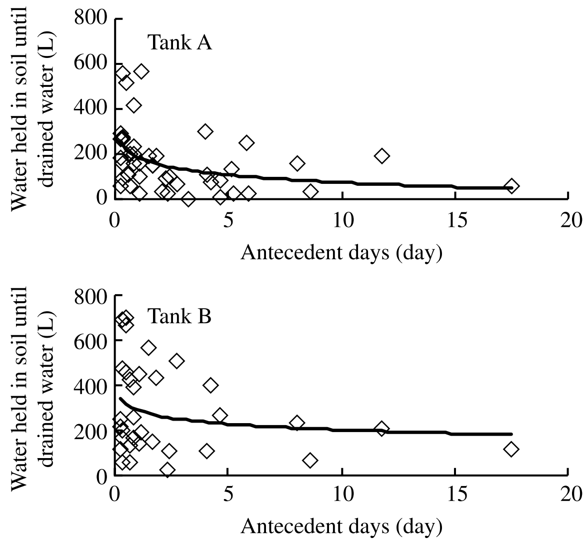

Figure 8 Water held in soil until drained water

Water held in soil at the start of draining decreased with increase in number of antecedent days. This is explained by the fact that the infiltration capacity recovers with increase in number of antecedent days due to increase in aeration of the soil. The following relationships were formulated:

$$\text{Tank A} \quad S_0 = -49.03 \ln(x) + 186.75 \; R^2 = 0.17 \tag{11}$$

$$\text{Tank B} \quad S_0 = -38.34 \ln(x) + 290.51 \; R^2 = 0.05 \tag{12}$$

In Equations (11) and (12), S_0 is water held in the soil at the stopping of water inflow and x is antecedent days (days).

There was seen to be a relation between drained water and water held in soil. At each survey, the relation between the ratio of drained water at maximum drained water (k_D) and water held in soil is shown in Figure 9.

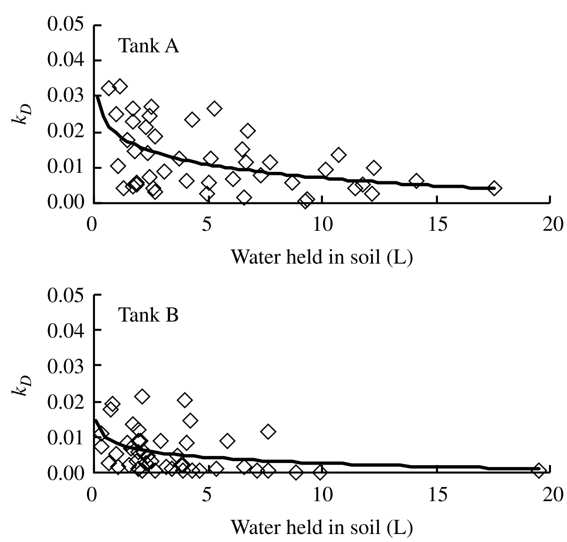

Figure 9 Ratio of drained water

The following relations were formulated:

Tank A $\quad k_D = -0.0054 \ln(S) + 0.052 \; R^2 = 0.23$ (13)

Tank B $\quad k_D = -0.0024 \ln(S) + 0.023 \; R^2 = 0.12$ (14)

In Equations (13) and (14), S is water held in soil. The above relations were used to determine k_D in time-series. From the stopping of water inflow, water held in the soil was lost through draining and evaporation. Therefore, the behavior of water held in the soil was investigated to determine drainage coefficient at the stopping of water inflow (ks_0). The relation between precipitation and ks_O is shown in Figure 10.

The following relationships were formulated:

Tank A $\quad ks_0 = 0.0018 \ln(x) - 0.0083 \; R^2 = 0.21$ (15)

Tank B $\quad ks_0 = 0.0008 \ln(x) - 0.0042 \; R^2 = 0.11$ (16)

In Equations (15) and (16), x is precipitation (mm). The evaporation coefficient (k_s) was determined to converge to 0 with time from the stopping of water inflow and the following relationship was formulated:

Tank A, B $\; ks = m * \ln(t) - ks_0$ (17)

In Equation (17), m is a constant.

Pollutant Load. Using results of actual measurement of the quality of drained water, the pollutant load (expressed as COD) was simulated. At each tank, the relationship between mean pollutant concentration of inflow water ($\Sigma L_I / \Sigma Q_I$) and the initial pollutant concentration in drained water (C_{Fa}) and the terminal pollutant concentration in drained water (C_{Ter}) is shown in Figure 11.

The following relationships were formulated:

Initial water quality

Tank A $\quad C_{Fa} = 0.84x + 18.96 \; R^2 = 0.90$ (18)

Tank B $\quad C_{Fa} = 1.30x + 31.34 \; R^2 = 0.42$ (19)

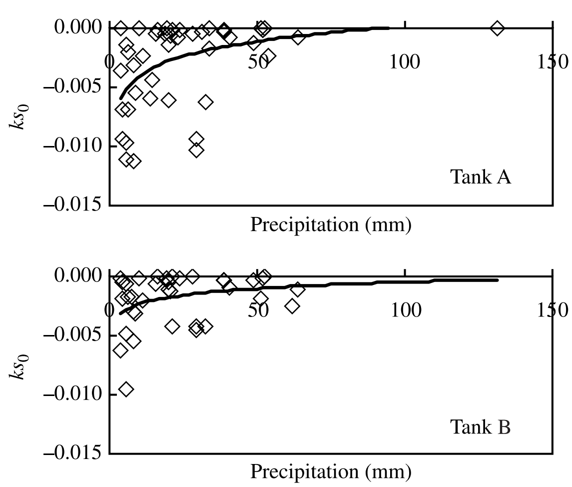

Figure 10 Variation of ks_O with precipitation

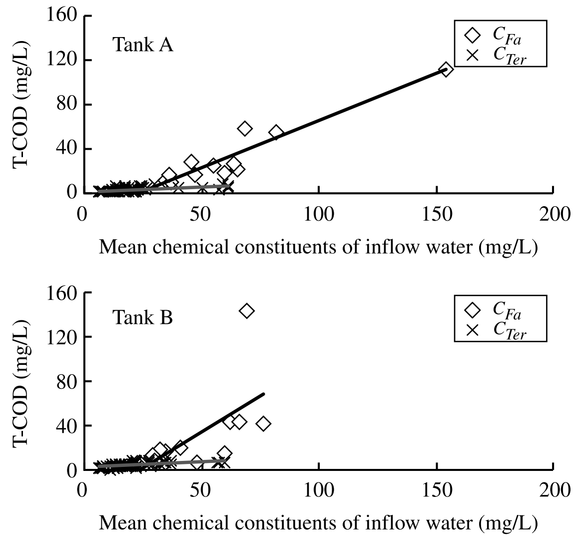

Figure 11 Quality of drained water

Terminal water quality

$$\text{Tank A} \quad C_{Ter} = 0.086x + 1.56 \; R^2 = 0.37 \tag{20}$$

$$\text{Tank B} \quad C_{Ter} = 0.106x + 1.96 \; R^2 = 0.44 \tag{21}$$

In Equations (18) – (21), x is T-COD of inflow water (mg/L). Using the above results, the following relationship was established:

$$C_D = (C_{Fa} - C_{Ter}) * (1 - \exp(-k_{CD} * t) + C_{Ter} \tag{22}$$

In Equation (22), k_{CD} is a constant.

Water flow simulation

Water flow simulation was done for six rainfalls from 2002/07/09 to 2002/07/19 among the 50 rainfalls that were analyzed in this study. Measured and estimated values for Tank

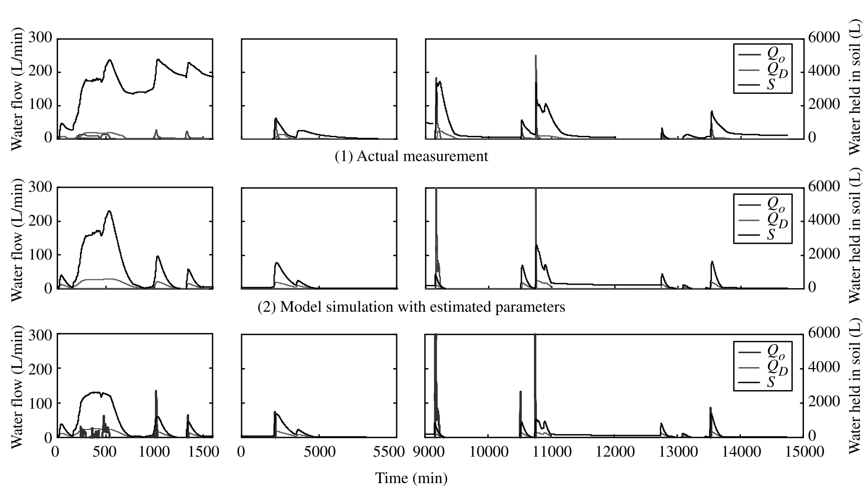

Figure 12 The result of water flow simulation

Table 6 The result of water flow simulation

Date	Actual measurement			Model simulation with estimated parameter						Model simulation with 20% of infiltration capacity					
				Estimate value			Relative error			Estimate value			Relative error		
	ΣQ_o (L)	ΣQ_D (L)	S(L)	ΣQ_o (L)	ΣQ_D (L)	S(L)	ΣQ_o (%)	ΣQ_D (%)	S(%)	ΣQ_o (L)	ΣQ_D (L)	S(L)	ΣQ_o (%)	ΣQ_D (%)	S(%)
2002/7/9	5,382	11,049	3,414	0	19,728	117	−100	79	−97	4,091	15,676	78	−24	42	−98
2002/7/13	738	1,621	–	0	2,390	48	−100	47	–	164	2,201	43	−78	36	–
2002/7/15	787	1,710	956	0	3,063	405	−100	79	−58	495	2,589	384	−37	51	−60
2002/7/16	5,372	4,605	126	9,365	927	8	74	−80	−94	9,365	917	7	74	−80	−94
2002/7/17	3,882	4,138	121	1,823	5,976	348	−53	44	187	4,035	3,923	188	4	−5	55
2002/7/18	1,610	1,850	232	0	3,839	78	−100	107	−66	278	3,481	54	−83	88	−77

A are shown in Figure 12 and the results of numerical analysis are shown in Table 6. The results show significant differences between the actual measurement and the estimated values of the amount of overflow. The discrepancy could be due to the fact that the estimated value of infiltration capacity was smaller than the actual measured value. The estimated values of infiltration capacity used in the model were reduced to 20%. As a result, the simulated value of flow approximated the actual measurement.

Conclusions

Time-series models were used to simulate the purification mechanism of road surface runoff in a soil penetration facility. The following results were obtained: 1) infiltration capacity was determined to be decreasing from the start of overflow, and to converge to a constant value at the end, 2) the start of flow of drained water was shown to be influenced by the recovery of infiltration capacity of the soil in antecedent days, 3) the decrease of water held in the soil from the time of stopping of inflow of water, due to drainage and water loss by evaporation, was illustrated, and 4) results of model simulation for flow did not fit well with measured values. Further analysis of the model parameters is needed.

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