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Study on Estimation of Inflow Discharge to Underground Storage System for Mitigation of Urban Inundation Damage

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Abstract

Underground storage systems are installed to mitigate damage of urban flooding. Estimation of overflowing discharge into those storage systems is significant for evaluation of their mitigation effects. Such overflow happens between pressurized sewer pipe and storage system over a side weir. The equation proposed by De Marchi to estimate overflow discharge over a side weir has been verified through several experimental researches only for open channel flow. Therefore in this study, overflow discharge coefficient is evaluated for pressurized flow of circular pipe through experiments with different side weir length. From obtained results, peculiar water head profile over the side weir can be observed. Discharge coefficient derived from experimental results implies applicability of constant value to estimate overflow discharge if mean value of integrated water head along the side weir is used, which should be verified through numerical simulation in the future.

Key words：Underground storage system, Side weir, Discharge coefficient

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1. INTRODUCTION

Even though a large number of researches related to reduction of the urban inundation damage have been conducted, the damage is still serious problems every year. Urban inundation has caused immense property damage and personal injury due to local heavy rainfall during a short-time and extreme climate in worldwide. To mitigate this problem, underground storage systems as an effective countermeasure have been implemented especially in highly urbanized area. However, there are no criteria how much mitigation effect can be expected from installation of such underground storage systems. In many cases, those storage systems are attached to sewerage systems, and some part of stormwater within a sewerage pipe is diverted over the side weir into the storage system. Therefore, evaluation of mitigation effect of storage system requires appropriate estimation of overflow discharge from sewerage system over the side weir. So, the researches in terms of overflow discharge over the side weir related to underground storage systems are essential.

Side weirs are hydraulic structures that are usually used as flow dividers in urban drainage systems, irrigation channels and flood protection works (Granata et. al. 2013). The main contributor to the understanding of hydraulic behavior of side weir is De Marchi(1934). He presented theory based on the assumption of constant energy head along the side weir and the overflow discharge being calculated by classical weir formula which overlooks the effect of lateral outflow direction, local velocity and type of flow (pressurized or non-pressurized) in the system. The equation is as follows,

$$q = \frac{dQ_{out}}{dL} = \frac{2}{3} C_d \sqrt{2g} (h - p)^{\frac{3}{2}}$$

where q is discharge per unit length of side weir, Q_{out} is overflow discharge, L is distance along the side weir measured from upstream end of side weir, g is the acceleration of gravity, p is the height of the side weir, h is the flow depth at the section L, C_d is the discharge coefficient of side weir. The discharge coefficient is influenced by the following parameters,

$$C_d = f(v, D, g, h, p, L, S \cdots)$$

where D is diameter of main pipe, S is slope of main channel.

Some previous theoretical analyses and experimental researches have been reported in terms of flow over rectangular side weirs in circular open channel (Allen, 1957 ; Uyumaz and Muslu, 1985 ; Vatankhah, 2012 ; Granata, 2013). Generally, The method assumes one-dimensional flow conditions, thus neglecting the variations of overflow direction and the velocity distribution (Willi H. Hager 1987). Besides, as seen in Table 1, a lot of researches obtain the equations for discharge coefficients for rectangular side weirs based on experimental results, where F is Froude number for the upstream end of the side weir on the main channel. Those side weir discharge coefficients are determined by varying experimental conditions like flow state, weir length and height.

All of the above researches have been conducted under open channel flow conditions, and the De Marchi's approach seems appropriate for the open channel flow condition. However, there is no study that verify the De Marchi equation's suitability in the pressurized flow condition. Thinking of overflow from sewerage system during urban flooding, pressurized flow condition would often happen and De March equation's applicability must be also verified in that condition. Hence in this research, the experimental set up is proposed with pressurized circular channel with different side weir length so that the effectiveness of the use of

Table 1 Side weir discharge coefficient equations

Researcher	**Equation**
Subramanya et al. (1972)	$C_d = 0.611\sqrt{1-(\frac{3F^2}{F^2+2})} = 0.864(\frac{1-F^2}{2+F^2})^{0.5}$
Yu-Tech (1972)	$C_d = 0.623 - 0.222F$
Hager (1987)	$C_d = 0.485(\frac{2+F^2}{2+3F^2})^2$
Cheong (1991)	$C_d = 0.45 - 0.221F^2$
Borghei et al. (1999)	2 $C_d = 0.7 - 0.48F - 0.3\left(\frac{p}{h}\right) + 0.06\frac{L}{D}$

the above equation can be discussed.

The aim of this experiment is to determine the discharge coefficient for pressurized flow in circular channel with different side weir length so that effective conclusions can be made regarding suitability of the De Marchi equation also for these cases and such experimental data can be extensively used to validate future numerical model estimating effects of underground storage systems.

Fig. 1 Photo of the experimental setup

2. EXPERIMENTAL SETUP

The experiments were carried out in the Ujigawa Open Laboratory of the Disaster Prevention Research Institute (DPRI), Kyoto University. Experimental setup, side weir and sectional view of a rectangular side weir are shown in Fig. 1, Fig. 2 and Fig. 3.

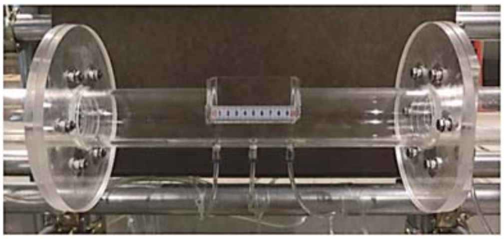

Fig. 2 Photo of the side weir

The model and proto type scale are assumed to be 1/22. Experimental setup consisting of side weir with the two circular acrylic pipes of 4m long and 0.05m internal diameter. There are upstream supply tank with the recirculation pump system and downstream collecting tank with a movable gate to adjust the downstream water level. The recirculation system can be controlled by the RPM controller. A flowmeter was used to measure the upstream input discharge. The flow heads have been measured by total seventeen piezometer tubes placed along the lowest bottom of pipe, as shown in Fig. 4. All of the experiments were carried out by the horizontal pipe. As seen in Fig. 5, the three different lengths of side weir, 10cm, 15cm and 20cm, have been used. Experiments were conducted for steady flow condition. The weir height was set as 4cm in all the cases. The length

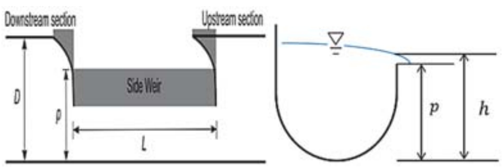

Fig. 3 Sketch of the rectangular side weir in a circular pipe

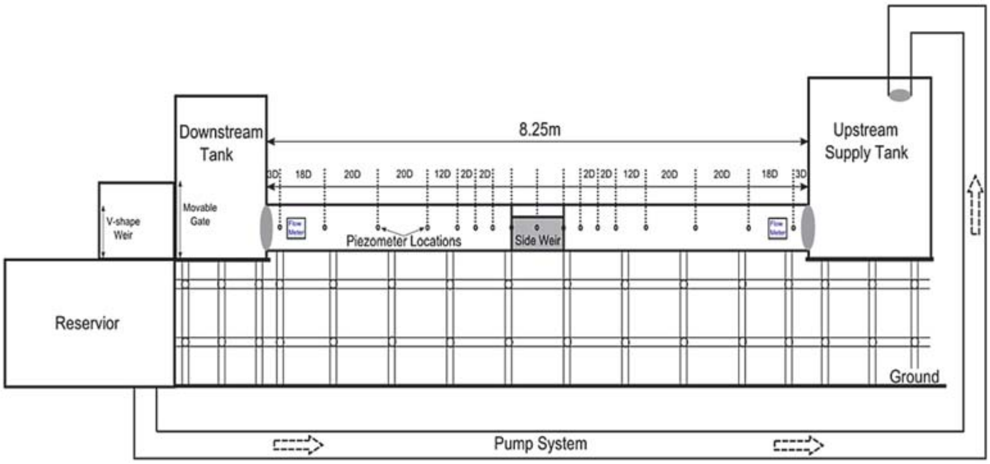

Fig. 4 Experimental arrangement

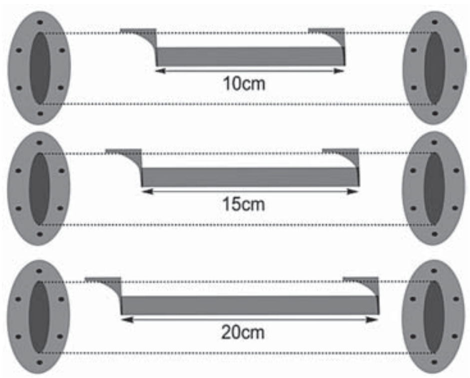

Fig. 5 The three different length of side weir

and height of side weir model were determined according to real size of the pipe diameter and the side weir in Moriguchi city, which can be regarded as a typical overflow system of side weir.

3. EXPERIMENTAL CONDITIONS

The experimental condition here is considered steady. Although, in reality, the phenomenon is unsteady, but for simplification of the problem, the steady-state condition is chosen and unsteady also will be analyzed as an advancement of the research. The experiments of 33 cases in total were conducted to determine overflow discharge coefficient keeping steady-state condition with different side weir length. The 11 cases are carried out for each different side weir, and supplied discharge at upstream tank differs from 0.5 l/s to 1.5 l/s. The downstream movable gate level is set to the bottom of main pipe. The detailed hydraulic conditions were summarized in Table 2, which contains the observed water level at the downstream tank. Each case of experiment was repeated three times to consider consistency of overflow discharge rate.

4. EXPERIMENTAL RESULTS AND DISCUSSION

(1) WATER HEAD PROFILE

The water surface profiles of whole pipe system of the case with 10cm weir length and those around the side weir of all the cases are described in Fig. 6. Fig. 7 shows the overflowing water of experiment of the case with 10cm weir length. On the whole, the water head at the upstream end of the side weir is lower than that at the downstream end of the side weir, as shown in Fig. 6. The same results were observed in all previous experimental studies. The water head slightly decreases at the middle of side weir for small discharge cases. These results are influenced by entrance effect at the upstream end of the side weir and lateral flow on the side weir. As the upstream discharge

Table 2 Experimental conditions (10cm, 15cm, 20cm)

Weir length (cm)	Case1	Case 2	Case3	Case4	Case5	Case6	Case7	Case8	Case9	Case10	Case11
Upstream discharge (l/s)											
10, 15, 20	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5
Froude number											
10, 15, 20	0.36	0.44	0.51	0.58	0.65	0.73	0.80	0.87	0.95	1.02	1.09
Water level at the downstream tank (cm)											
10	1.1	1.1	1.2	1.3	1.4	1.4	1.5	1.6	1.7	1.8	1.8
15	1.1	1.1	1.2	1.2	1.3	1.4	1.4	1.5	1.5	1.6	1.7
20	1.1	1.1	1.2	1.2	1.3	1.4	1.4	1.4	1.5	1.6	1.7

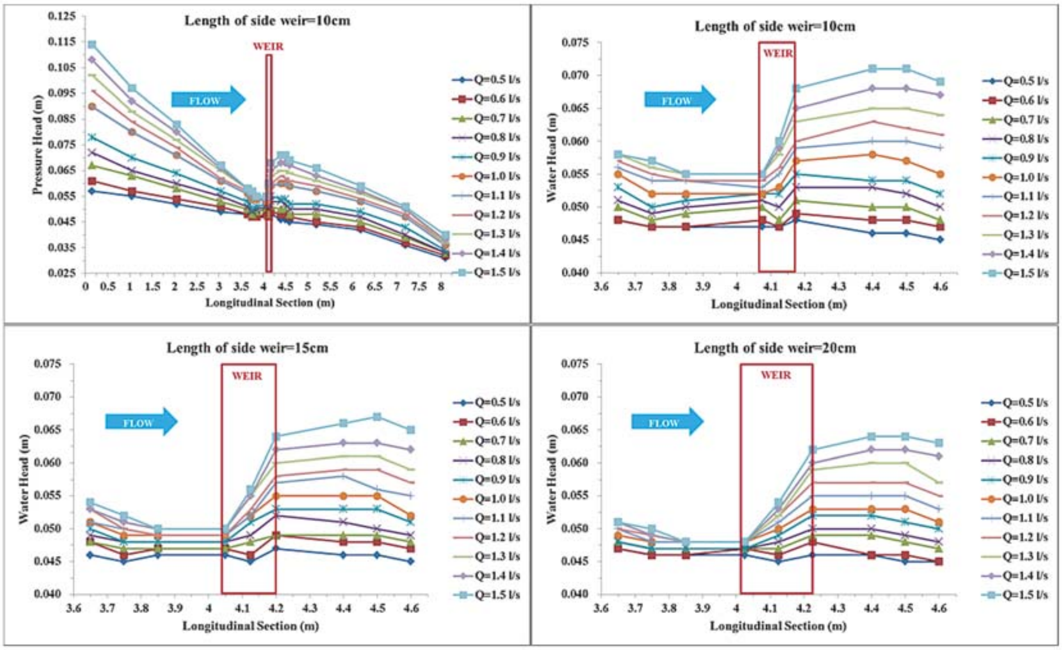

Fig. 6 The water surface profile

increases, the hydraulic gradient becomes steeper in comparison with the results under smaller discharge cases. The high water head at the downstream end of the side weir have an effect on water heads of the downstream pipe. The reason for this maybe the strength of lateral flow under the large discharge conditions. This result shows the difference between this study and previous studies.


Fig. 7 Photo of overflowing water of experiment

(2) DISCHARGE COEFFICIENT

The discharge coefficient was calculated by De

Marchi's equation as follows,

$$C_d = \frac{\frac{3}{2}Q_{out}}{\sqrt{2g}\,L(h-p)^{\frac{3}{2}}}$$

This discharge coefficient depends on the hydraulic conditions that are weir length, weir height, water head on the side weir and overflow discharge. Under the low discharge conditions, water heads are nearly constant but lateral flow is not constant. Under the large discharge conditions, both water head and lateral flow are not constant. As the upstream discharge increases, the lateral flow rate also increases in comparison with results under the low discharge conditions. According to some previous studies, water head at the upstream end of the side weir is used for parameter of discharge coefficient. However, as seen in the surface profile of Fig. 6, there is lager variation in water heads on the side weir excessively compared with those of previous studies. With this point of view, the averaged water head along the side weir adopted, which was derived from the side weir length and water head integrated along the side weir.

Uyumaz (1985) also reported that water head was not constant on the side weir. Using the mean of upstream and downstream water heads on the side weir did not produce satisfactory solutions. Calculating the mean of several intermediate heads proved more satisfactory results. Fig. 8 shows the coefficient value increases gradually if the water head at the upstream end of the side weir was used. On the other hand, the discharge coefficient remains almost constant if the mean value of integral heads was adopted instead of water head at the upstream end of the side weir. In particular, these coefficient values become close to constant value although upstream discharge increases, which implies applicability of constant coefficient value. Table 3 shows the experimental results of overflow discharge rate and calculated mean of

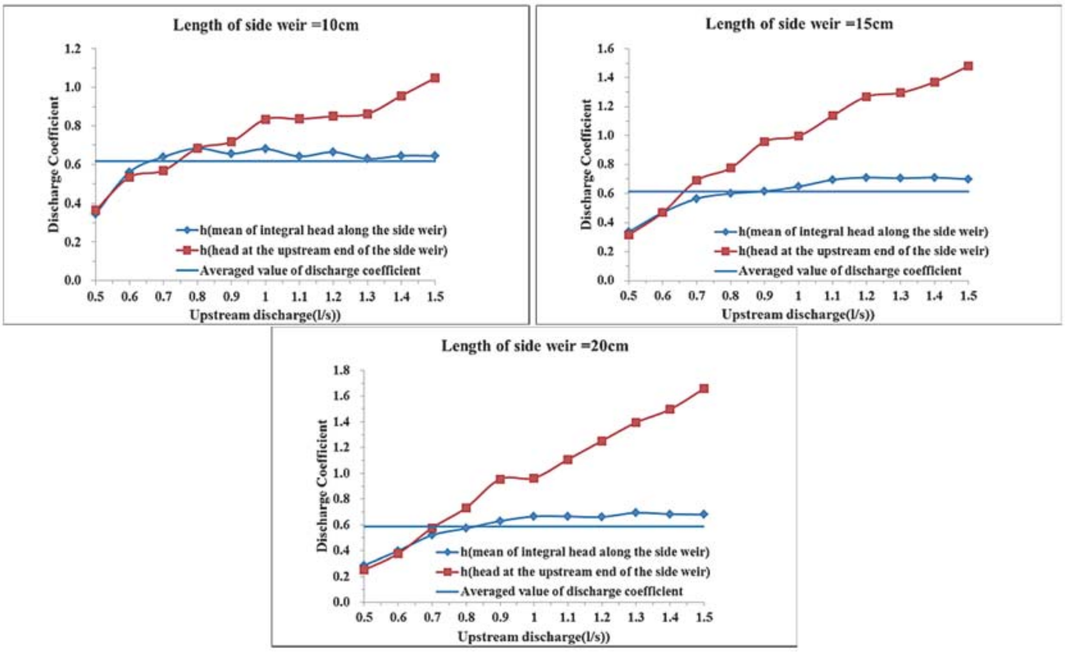

Fig. 8 The discharge coefficient distribution along the different water head on the side weir

Table 3 Experimental results (10cm, 15cm, 20cm)

Overflow discharge (l/s)											
Weir length(cm)	**Case1**	**Case 2**	**Case3**	**Case4**	**Case5**	**Case6**	**Case7**	**Case8**	**Case9**	**Case10**	**Case11**
10	0.063	0.113	0.168	0.233	0.279	0.324	0.366	0.416	0.468	0.519	0.569
15	0.065	0.122	0.179	0.246	0.304	0.377	0.430	0.480	0.574	0.607	0.656
20	0.069	0.130	0.199	0.253	0.330	0.407	0.468	0.529	0.589	0.632	0.700

Mean of integral heads on the side weir (cm)											
Weir length(cm)	**Case1**	**Case 2**	**Case3**	**Case4**	**Case5**	**Case6**	**Case7**	**Case8**	**Case9**	**Case10**	**Case11**
10	4.73	4.78	4.93	5.10	5.28	5.38	5.55	5.65	5.85	5.95	6.08
15	4.58	4.7	4.80	4.95	5.08	5.20	5.25	5.33	5.50	5.55	5.65
20	4.55	4.68	4.75	4.82	4.93	5.03	5.13	5.23	5.28	5.35	5.45

Discharge coefficient												
Weir length(cm)	**Case1**	**Case 2**	**Case3**	**Case4**	**Case5**	**Case6**	**Case7**	**Case8**	**Case9**	**Case10**	**Case11**	**Average value of Cd**
10	0.346	0.561	0.639	0.684	0.656	0.681	0.642	0.665	0.630	0.645	0.645	0.618
15	0.337	0.470	0.565	0.600	0.616	0.647	0.695	0.711	0.705	0.710	0.699	0.614
20	0.286	0.397	0.519	0.572	0.628	0.664	0.664	0.661	0.693	0.682	0.679	0.586

integral heads on the side weir. The discharge coefficients for each case are also shown in Table 3, which are derived from experimental data. As weir length increases, the mean of coefficient values decreases.

In application to actual estimation of overflow discharge, the constant coefficient value would be easy to handle. Therefore, as the next step, applicability of constant value derived from these experiments will be investigated by using numerical model. And this time, my study is limited to steady condition only for simplicity. In future work, I will consider the unsteady condition also.

5. SUMMARY

The present study has investigated the variation of discharge coefficient for pressurized flow in circular channel with different side weir length. A detailed work is also presented for flow surface profile along the side weir and the discharge coefficient of the side weir. The suitable discharge coefficient is essential to estimate the model validation through the experimental data and the water head on the side weir.

Finally, the next step will be to find out the suitable discharge coefficient within the limit of above experimental results for each weir length and experimental condition and to reproduce the overflow discharge obtained from experiment by using numerical model. That numerical model verified in that way would enable to estimate overflow discharge from sewerage to storage system and mitigation effect of those storage systems.

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