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Design of Forebay and Micropool for Highway Stormwater Detention Basins

Prepared
by

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16. Abstract

A new procedure was developed to design a sediment forebay at the entrance of a stormwater detention basin. It was found that the geometry of the forebay depends on the size of the target particle for settlement in the forebay. The fall velocity of the target particle and the horizontal inflow velocity provide the basis to size the length and depth of the forebay pool. From the gradation curve derived from the soil laboratory, the target particle is recommended to have a diameter of 0.1 to 0.2 mm. It is expected that a forebay will intercept 60 to 70% of sediment load in stormwater. A micropool is designed to provide a continuous suction flow to drain the remaining water in the detention basin after the outlet screen become clogged. In this study, dry samples of float deposits were collected from the screens in detention basins, and they were analyzed in the soil laboratory to construct the gradation curves. It was found that D90=0.3 mm and the saturated specific gravity is 0.8 to 0.9. The new concept of float velocity was derived in this study. Using the float velocity as the basis, the micropool surface area and flow depth can be determined. As recommended in this study, the flow depth in micropool shall include evaporation loss through the period of inter event time, dead storage for solids, and suction head to sustain the flow. The screens covering WQCV outlet plates have long been a maintenance issues due to clogging and UDFCD now recommends only three or four orifices to maximize the orifice size and minimize clogging of the orifice plate.

Implementation

1. New design procedures for forebay sizing and micropool design will be incorporated into CDOT‘s Hydraulic Design Manual. Technical papers will be prepared to translate the findings into engineering design charts. A condensed version of this report will be presented in conferences.
2. Sediment gradation samples collected at locations upstream and downstream of the EDB 502L will be submitted to the CDOT's maintenance. EDB 502L is conservatively built and does satisfy the design criteria for both water quality and flood mitigation purposes.

17. Keywords Forebay, Micropool, Highway Detention, Siphon Flow, Urban Trash, WQCV.	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service www.ntis.gov or CDOT's Research Report website http://www.coloradodot.info/programs/research/pdfs

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Design of Forebay and Micropool for Highway Stormwater Detention Basins

CONTENTS

LIST OF FIGURES

LIST OF TABLES

Design of Forebay and Micropool for Highway Stormwater Detention Basins

RESEARCH STATEMENT

A storage facility is portrayed by its stage-storage-outflow curve. Design of a detention basin involves a *volume-based approach* to construct the stage-storage curve, and another *flow-based approach* to define the stage-outflow curve. The former requires the information of basin geometry, while the latter involves the details of forebay, micropool, and outlet structure. There are two popular volume-based methods recommended to quantify the required detention volume, including the *water quality capture volume (WQCV) approach* to intercept frequent runoff events, and *excess urban runoff volume (EURV) method* to reduce post-development peak flows. Both empirical formulas for WQCV and EURV were derived from the assumption that the basin is completely empty before receiving the next event. In fact, the operation of a basin is always dependent on the remaining water depth from the previous event. During a rainy season, the accumulation of continuous rain-runoff volumes into the basin can lead to a false alarm of extreme event. Secondly, designs of forebay and micropool should be related to the gradation distribution of solid particles in stormwater. Empirical recommendations may lead to oversized micropool and undersized forebay or vice versa. Therefore, in this study, it is proposed to investigate: (1) *how to design forebay and micropool based on the on-site sediment characteristics*, and (2) *how to add a freeboard to a basin according to the risk of residual water depth*. For this project, the Urban Drainage and Flood Control District is responsible to monitor the extended detention basin located at S Knox Court and State Highway 285. The collected data are used to verify the new methods developed in this study. Findings from this study provide significant improvements to current design methods.

Key Words: Water Quality Capture Volume, Excess Urban Runoff Volume, Micropool, Forebay, Drain Time, Detention, Residual Water Depth.

1. INTRODUCTION

Due to the *Clean Water Act*, local governments in the US require urban stormwater runoff to be treated in order to improve water quality in the downstream receiving water bodies. In conjunction with water quality treatment, current design criteria also require the attenuation of peak flows under the post-development condition. As more areas are urbanized, pollutant loads in stormwater are increased. Without mitigation, these pollutants would negatively impact waterway ecology and wildlife habitat when travelling downstream into the receiving waters. The concept of *Stormwater Quality Capture Volume* (WQCV) was developed to define the required stormwater volume to be treated for reducing total suspended solids (TSS) and metal pollutants prior to being released downstream (Urbonas et al. in 1989, Guo and Urbonas 1996). Based on field observations performed in the EPA Reports in 1983 and 1986, an average extended detention time of 12 hours provides a removal rate of 80-90% to reduce the annual TSS load generated from the tributary watershed (Driscoll et al. 1989).

In the last decade, the concept of *Excess Urban Runoff Volume* (EURV) expressed in inch/catchment was also developed and recommended as a new design standard intended to replace the extended detention basin (EDB) standard in the metro Denver area (UDFCD 2011). A EURV represents the on-site increased runoff volume directly due to the increase of impervious surface area within the tributary catchment. A full spectrum detention basin (FSDB) is constructed with its WQCV as the bottom layer for water quality enhancement up to the 6-month event, and EURV as the upper layer for peak flow reduction up to the 10-yr event. Detention basins are often equipped with a *forebay* to trap particles ≥1 to 2 mm in diameter, and a *micropool* to sustain continuous flow released by inverted Siphon effects in case of outlet clogging. In current practice, both EDBs and FSDBs are designed under the assumption that all runoff events coming into the basin are independent of previous storm events. In fact, the operation of a basin is certainly dependent on the residual water depth from the previous storm events. During a rainy season, the accumulation of runoff volumes from a series of storm events may fill up the basin as if the major storm occurred. According to the drainage manual (UDFCD 2011), a drain time of 40 hours is recommended for WQCV, and less than 72 hours for EURV (per Colorado Revised Statute 37-92-602(8), 97% of the 5-year storm must drain in 72 hours or less). In practice, the longer the drain time is, the higher the overflow risk is. Therefore, it is a challenge as to how to reduce the risk of residual water depth by selecting a proper freeboard for a basin.

The goals of this report are twofold: (1) to develop consistent procedures to size forebay and micropool, and (2) to develop a risk-based guidance to evaluate the residual water depth in a basin.

1.1 Basic Concept of Watershed Depression Storage

A rural watershed is characterized with its hydrologic losses including interception, infiltration and depression losses. In comparison, interception losses due to bushes and trees are negligible in an urban area. Depression loss depends on the storage volume associated with the depressed area. Infiltration loss depends on the type of soils; and it occurs to the overland flows as soon as the rainfall excess exceeds the depression losses. Developments of an urban area result in more pavements, impervious surfaces, and fills of depressed areas. As illustrated in Figure 1.1, an urban drainage system often includes the underground storm sewers sized to carry the minor event, and the street gutter designed to deliver the major event. Such a double-decker flow system is to mimic the natural waterway that consists of a low-flow main channel and overbank floodplains.

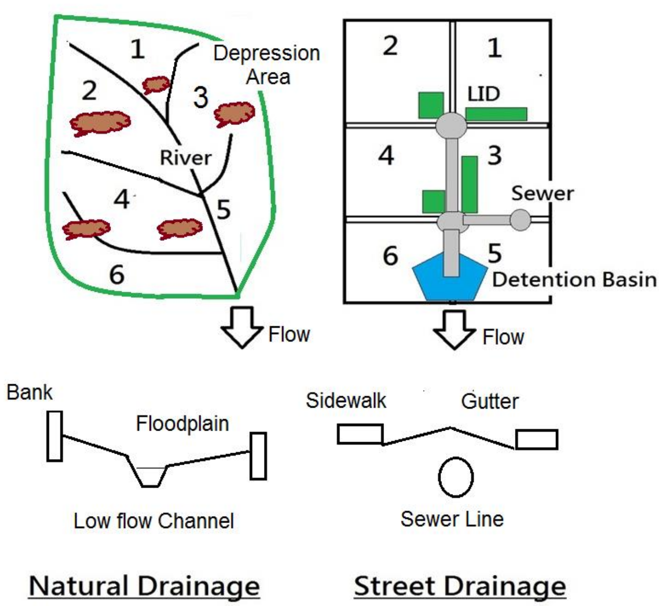


Figure 1.1 Comparison between Natural and Street Drainage Networks

In an urban catchment, the source of storm runoff is the impervious areas. Before the overland flows become concentrated, the *increased runoff volume per unit area* (V-problem) is the cause of water quality problems. After the overland flows are collected into street gutters, sewers, and channels, the *increased runoff flow* (Q-problem) is the cause of flooding problems.

Conventional stormwater management has focused on how to reduce peak flows using stormwater detention, while the latest development is to integrate the low-impact designs into the stormwater management to enhance both stormwater quality and quantity controls. A low-impact design is to apply a filtering process to better stormwater quality and an infiltration process to reduce stormwater volume. Since the low-impact designs are aimed at the runoff source control, therefore they are only applicable to a small tributary area. The latest developments on low-impact designs include infiltration beds, rain gardens, bio-swales, and porous pavements. As shown in Figure 1.2, stormwater low-impact designs are classified into: (1) *conveyance type* such as porous pavements using infiltration bed, and (2) *storage type* such as rain garden using an infiltration basin. Obviously, the effectiveness of a low-impact design depends on how to intercept the surface runoff volume. To differentiate from the stormwater detention storage volume (WDSV) for extreme events, the intercepted stormwater volume for low-impact designs is termed water quality capture volume (WQCV). A WQCV shall be in the same magnitude (i.e., 0.5 to 3 times) the natural depression volume that was obliterated during the urbanization process.

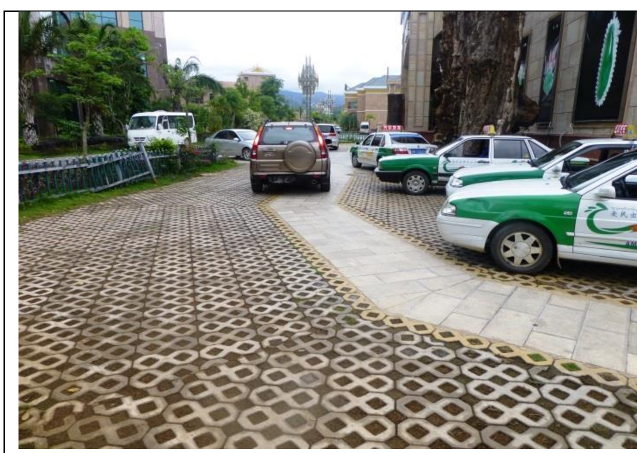
	
Conveyance Type -- Porous Pavement	Storage Type – Rain Garden

Figure 1.2 Conveyance and Storage Low-Impact Designs

Over 30 years of learning, in the year 2010, the effort of stormwater Best Management Practices (BMP) has concluded that the *low-impact-development* (LID) concept is the best approach to integrate both flood mitigation and water quality enhancement together.

2. WATER QUALITY CAPATURE VOLUME

2.1 Rainfall and Runoff Distributions

The conventional criteria developed for the purpose of flood mitigation are not suitable for sizing stormwater quality basins. It is because the goal of stormwater quality basin (WQB) is to capture frequent runoff events, not the extreme. Frequent rainfall events have to be delimited from a continuous record by a user-defined minimum inter-event time (Guo and Urbonas 1996). Such a minimum inter-event time of no rain is termed *event separation time*. As illustrated in Figure 2.1, the continuous record is divided into 3 events using an event separation time of six hours. After individual events are identified, the *event rainfall depth* and *duration* can be further calculated for statistical analyses.

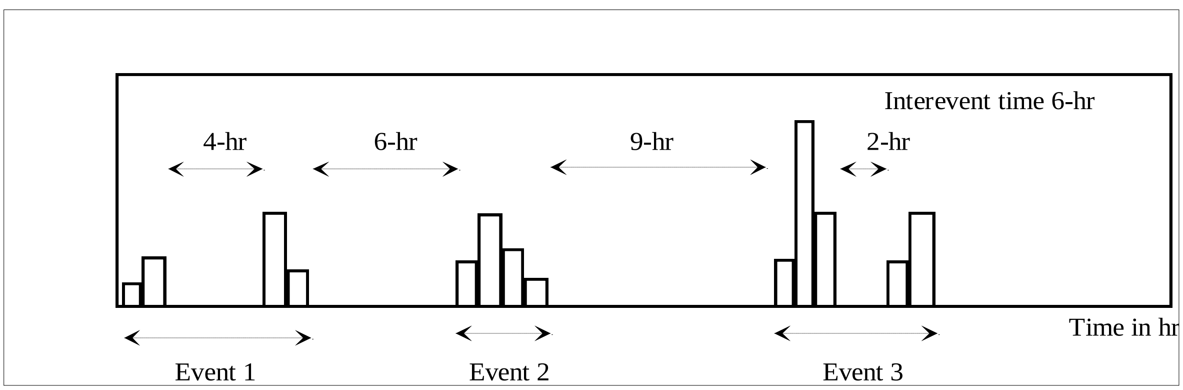


Figure 2.1 Event Separation by Inter-event Time.

In practice, the event separation time should be selected based on the watershed characteristics such as sediment resident time or basin's drain time. The 1986 EPA study reported that about *80* to *90* percent solids were removed if a 12-hour drain time is applied to a wet pond or a 24-hour drain time is applied to a dry pond. Therefore, it is recommended that the event separation time be the drain time of a basin. After a continuous rainfall record is divided into individual events, Figure 2.2 is the distribution of rainfall depths observed at the City of Denver, Colorado. Although a two-year storm event is often considered a small event for flood control projects, a 2-yr event, in fact, has a rainfall depth greater than *95* percent of the rainfall population. Figure 2.3 shows the distribution of rainfall depth by storm numbers observed over a period of 30 years in the City of San Diego, California. It shows that *97* percent of the events having a depth less than the local two-year rainfall depth. Although the skewness of event-rainfall depth distribution varies with meteorological region, it is generally true that the number of smaller rainfall events dominate the rainfall population.

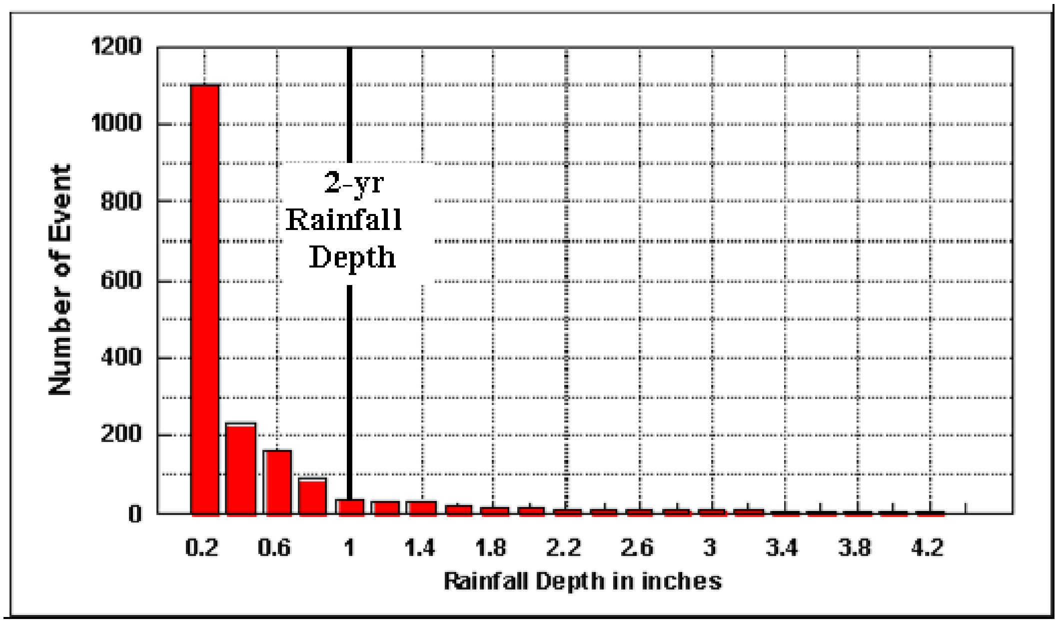


Figure 2.2 Rainfall Depth Distribution at Denver, Colorado

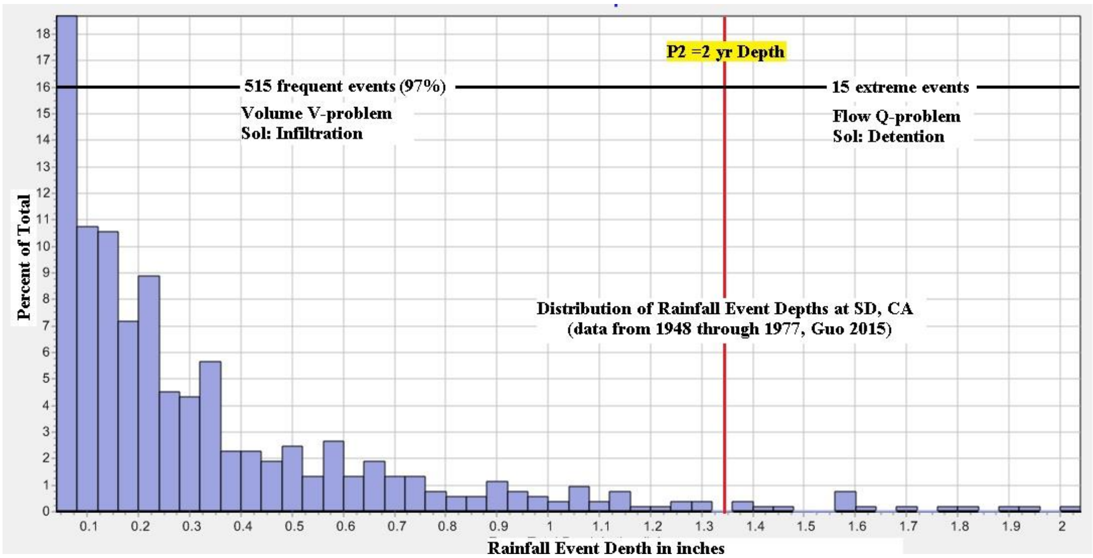


Figure 2.3 Rainfall Depth Distribution at San Diego, California

Not every raindrop can join surface runoff unless the event rainfall depth is greater than the interception loss. The runoff-producing rainfall depth is the difference between the recorded rainfall depth and the interception loss as:

$d_i = D_i - I_s$ in which d_i = runoff-producing rainfall depth in [L] for the *i-th* event, D_i = recorded rainfall depth in [L] at rain gage, and I_s = interception loss in [L] such as 0.05 to 0.1 inch, depending on ground slope and impervious cover. Having the continuous rainfall record divided into individual storms, the statistics for event-depth, duration, and inter-event time can further be calculated as:

$$D_m = \frac{1}{N}\sum_{i=1}^{i=N} d_i \tag{2.2}$$

$$S_D = \frac{1}{(N-1)}\left[\sum_{i=1}^{i=N}(d_i - D_m)^2\right]^{\frac{1}{2}} \tag{2.3}$$

$$C_s = \frac{1}{S_D{}^3 N(N-1)(N-2)}\left[\sum_{i=1}^{i=N}(d_i - D_m)^3\right] \tag{2.4}$$

$$T_{I\,m} = \frac{1}{N}\sum_{i=1}^{i=N} T_{I\,i} \tag{2.5}$$

in which D_m = average event rainfall depth, N = total number of events in the record, S_D = standard deviation, C_s = skewness coefficient, T_{Ii} = time interval to the next event, and $T_{I\,m}$ = average inter-event time. The above approach was employed to analyze the continuous rainfall records observed in eight metropolitan areas (Guo and Urbonas in 1996). Approximately *1,000* to *1,500* individual events were identified from each continuous rainfall record using an inter-event separation time of 6, 12, or 24 hours. The rainfall statistics in inches and average inter-event time in hours are summarized in Table 2.1. As indicated in Table 2.1, the average inter-event time is *9 to 57* times the event separation time. The distributions of rainfall depth are skewed in all cities used in Table

2.1. Figure 2.4 presents the rainfall event depth for the continent of the US derived from the study using a 6-hour event separation time and *0.1* inch as the interception loss (Driscoll et al in 1989).

Table 2.1 Rainfall Statistics Using 6-, 12-, and 24-hr Event Separation Times

City	6-hr				12-				24-			
	D_m	*S.D.*	C_s	$T_{I\,m}$	D_m	*S.D.*	C_s	$T_{I\,m}$	D_m	*S.D.*	C_s	$T_{I\,m}$
	Inch	*inch*		*Hour*	*Inch*	*inch*		*hour*	*inch*	*inch*		*Hour*
Seattle, WA	0.48	0.49	2.75	53.5	0.60	0.64	2.67	72.7	0.78	0.90	3.06	98.1
Sacramento,	0.61	0.62	2.96	166.7	0.72	0.76	3.50	208.8	0.82	0.92	3.44	251.6
Phoenix, AZ	0.42	0.36	2.59	261.3	0.45	0.40	2.41	300.1	0.48	0.44	2.57	341.8
Denver, CO	0.44	0.48	3.59	106.4	0.46	0.51	3.47	121.4	0.51	0.56	3.30	144.2
Cincinnati,	0.58	0.55	3.03	65.2	0.66	0.64	2.76	81.1	0.73	0.71	2.51	97.8
Tampa,	0.66	0.78	4.40	71.4	0.71	0.83	4.46	79.6	1.01	1.10	2.89	114.7
Boston, Ma	0.70	0.79	4.98	70.7	0.73	0.81	4.60	82.1	0.78	0.84	4.28	94.8

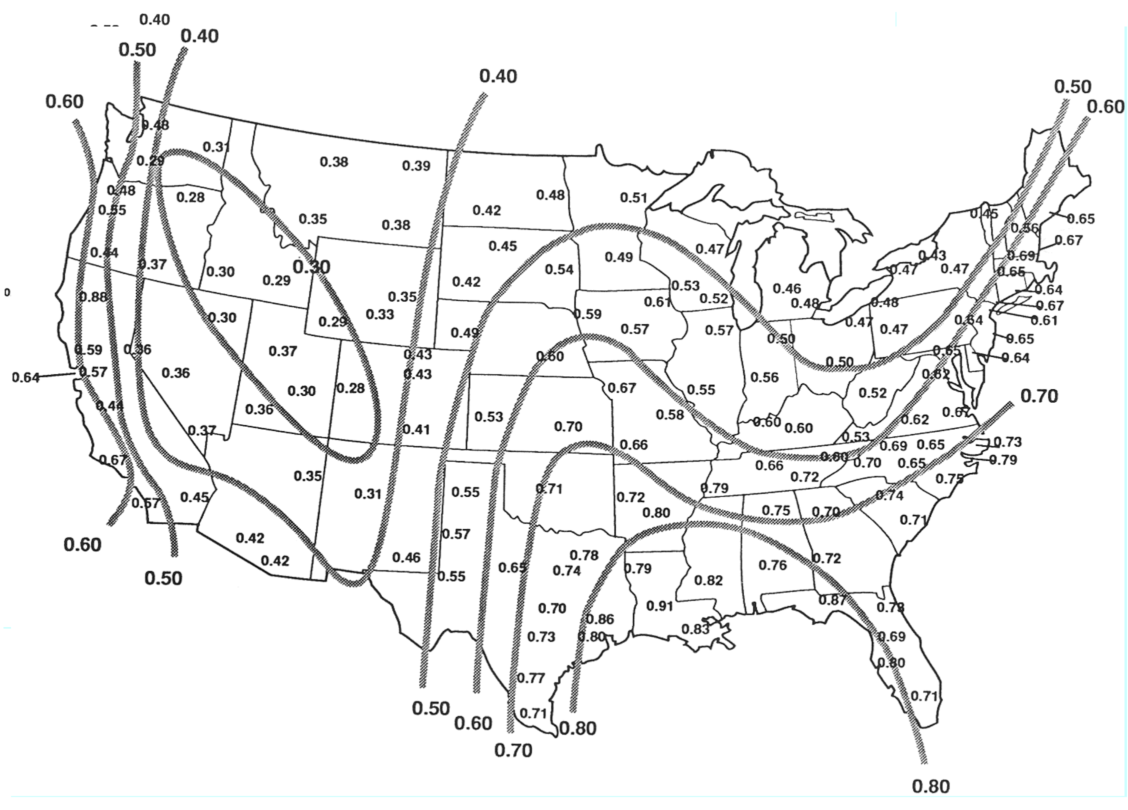


Figure 2.4 Average Rainfall Event Depth in Inches for the Continent of the US

2.2. Regression Model for WQCV

Based on the long-term data bases collected at *Seattle WA, Sacramento CA, Cincinnati OH, Boston MA, Phoenix AZ, Denver CO, and Tampa FL,* the regression formula was derived using the best fitted approach as (Guo and Urbonas 1996):

$$\frac{D_o}{D_m} = aC + b \tag{2.6}$$

$$C = 0.858 I_a^{\ 3} - 0.780 I_a^{\ 2} + 0.774 I_a + 0.04 \tag{2.7}$$

in which D_o= WQCV in inch per watershed area, D_m = average rainfall event depth in inches, C= runoff coefficient, I_a = watershed impervious ratio, and *a* and *b* = coefficients derived from regression analysis and listed in Table 2.2. The values for variable, b, are numerically negligible for practice or b=0 is acceptable. For the seven metropolitan cities, the regression equations show excellent correlation coefficients, r^2, ranging from *0.80* to *0.97*, depending on drain time.

Table 2.2 Coefficients for Determining Empirical WQCV

Drain Time	Volume	Ratio	
	A	*b*	*r-square*
12-hr	1.36	-0.034	0.80
24-hr	1.62	-0.027	0.93
48-hr	1.98	-0.021	0.84

The WQCV defined by Eq 2.6 would provide a runoff capture volume capture rate between 78.*0* and *85.0* percent (UDFCD 2011).

2.3. Exponential Distribution for WQCV

As aforementioned, the WQCV is in the same magnitude as the natural depression loss. With the average rainfall event depth in Figure 2.4, the probabilistic density function (PDF) for the rainfall depth distribution is described as (Guo and Urbonas 2002):

$$f(d) = \frac{1}{D_m} e^{\frac{-d}{D_m}} \quad (2.8)$$

in which d = a random rainfall depth in inch or mm , and D_m = average rainfall event depth in inch or mm in Figure 2.4. Considering hydrologic losses and incipient depth, the storage volume of WQB is treated as a runoff depth per watershed as:

$$D_o = WQCV \quad (2.9)$$

$$d = D_0 - I_s \quad (2.10)$$

in which D_o = WQCV as runoff depth in mm or inch per watershed, C = runoff coefficient, and I_s = incipient runoff depth such as *0.1* inch. Substituting Eq's 2.8 and 2.9 into Eq 2.8, the runoff capture percentage is integrated as:

$$C_v = P(0 \le d \le D_o) = 1 - ke^{\frac{-D_0}{CD_m}} \quad (2.11)$$

$$k = e^{\frac{-I_s}{D_m}}$$

(2.12)

in which C_v = runoff capture percentage and $P(0 \le d \le D_o)$=probability function. Any event that produces a runoff volume more than D_o will overload the basin. Therefore, the overflow risk is calculated as:

$$R_v = 1 - C_v = ke^{\frac{-D_o}{CD_m}}$$

(2.13)

The plot of C_v versus D_o using Eq 2.13 is termed *Runoff Capture Curve.* Figure 2.5 presents a set of generalized runoff capture curves produced using Eq 2.13 for runoff coefficients of *0.2, 0.4, 0.6, 0.8* and *1.0*. It is noticed that the curvature of runoff capture curve increases when the runoff coefficient decreases. The runoff capture curve becomes almost a linear response between rainfall and runoff amount when $C = 1.0$. This tendency reflects the fact that the higher the imperviousness in a catchment, the less the surface detention. As a result, the response of a highly urbanized catchment to rainfall is quick and direct.

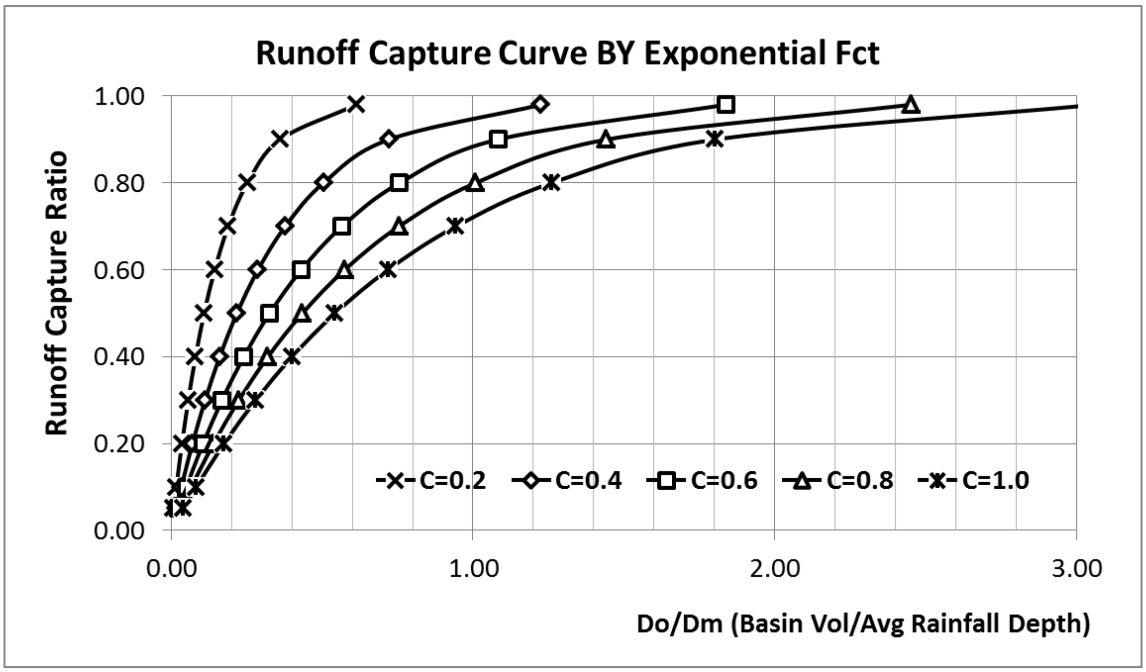


Figure 2.5 Normalized Runoff Capture Curves

Both Eq's 2.6 and 2.13 are derived based on the assumption that the basin is always emptied out before the next event. In case that the basin is operated through a rainy season, frequent small events will continually fill up the basin. As a result, the residual water depth from the previous event is a serious concern in the operation of a detention basin. When the basin is overtopped with a series of small events, a false alarm of the extreme event is triggered and the sediment removal function is impaired. In this study, an investigation of residual water depth is further conducted at the selected site for stormwater detention operations.

2.4 Field Investigation

The extended detention basin located at the intersection of South Knox Court and US Highway 285, designated as *EDB502L*, was designed and constructed in the year of 2011 as part of the US285 Reconstruction Project performed by the Colorado Department of Transportation (CDOT) (Harris, Kocher & Smith, 2011). This was a multi-phase project involving major improvements to US Highway 285 from Federal Boulevard to Kipling Street. The portion of the project which includes the EDB502L is designated as Area 4/Maintenance Yard Outfall and is located in portions of Sections 31 and 32, T. 4 S., R. 68 W. and Section 5, T. 5 S., R. 68 W., in Arapahoe County. The site location is shown in Figures 2.6 and 2.7.

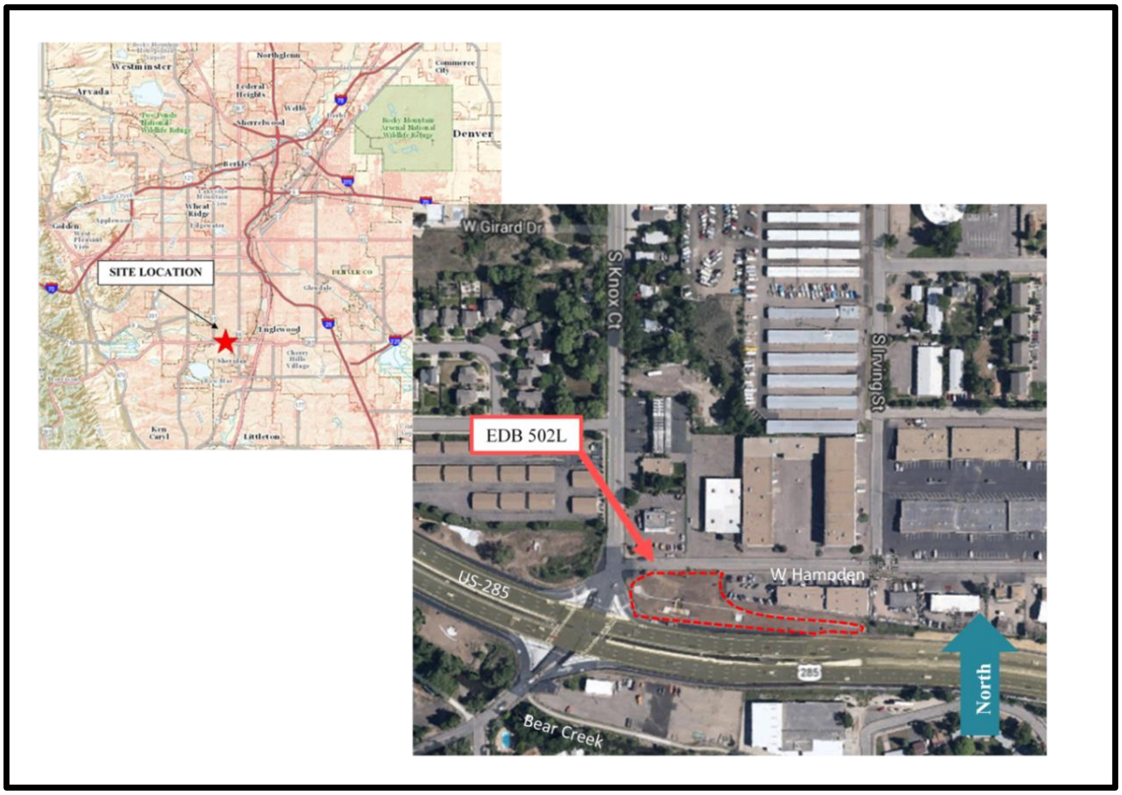


Figure 2.6 Site Location for Sample Extended Detention Basin (EDB 520L)

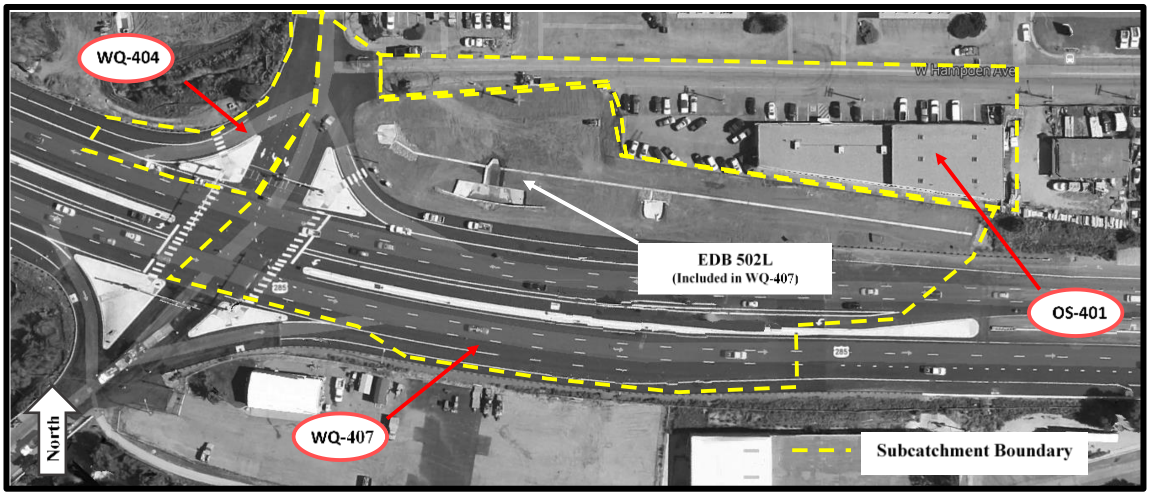


Figure 2.7 Tributary Sub-catchments to WQB in EDB 502L

EDB 502L was designed to comply with the water quality requirements of CDOTs Municipal Separate Storm Sewer Systems (MS4) permit, which mandates a permanent BMP treatment of runoff from all new and disturbed pavement. Because the US Highway 285 Reconstruction Project involved disturbing and laying miles of new pavement this extended detention basin and several others similar in nature were designed to catch and treat runoff from these areas for water quality storms. As summarized in Table 2.2, the tributary area during a water quality storm event to EDB 502L is limited to OS-401, WQ-404, and WQ-407 or a total of 4.90 acres since the smaller storms are carried offsite in the storm drain system. The off-site tributary area upstream of EDB 502L includes sub-catchments, OS-1, OS-2, and OS-6 as shown in Figure 2.8.

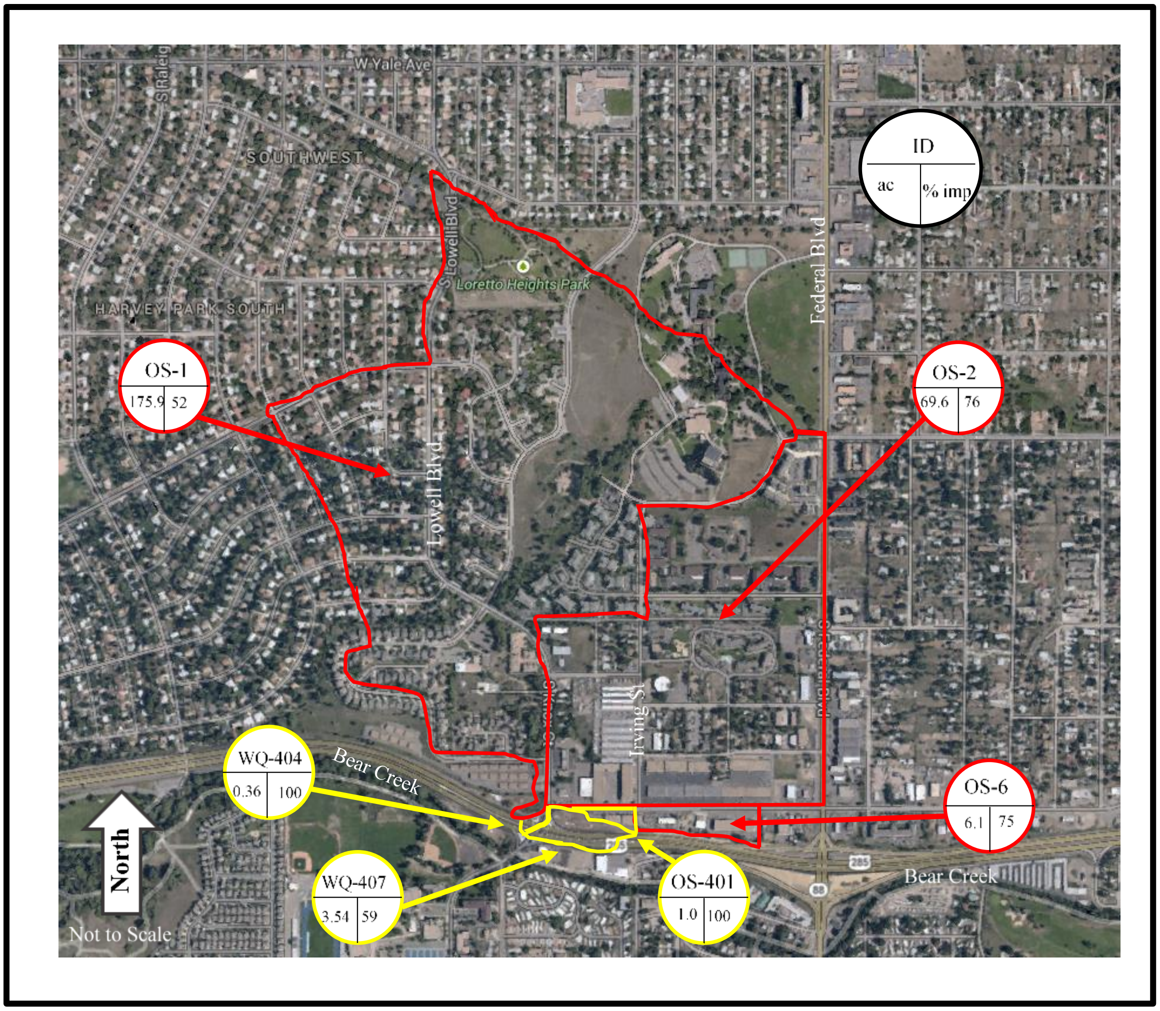


Figure 2.8 Off-site Sub-catchments Tributary into EDB 502L

Layout of Basin System

Micropool and Perforated Plate

Jet flows through the Perforated Plate

10-ft by 7-ft Outfall Culvert to Bear Creek

Figure 2.9 As Built EDB 502L Located at S Knox Court and SH 285.

Table 2-3 Water Quality Capture Volume For EDB 502L

WQCV Calculations			Drain	40 hr
Sub- ID	Area acres	Imperviousness %	WQCV Inch	WQCV acre-ft
WQ-404	0.36	100.00	0.500	0.042
WQ-407	3.54	59.00	0.233	0.069
OS-401	1.00	100.00	0.500	0.042
Sum	4.90		1.2330	0.1530

Table 2.4 Hydrologic Parameters for Off-site Catchments

Off-site catchment	Area	Impiousness
ID	Ac	%
OS-1	176	52
OS-2	69.6	76
OS-6	6.1	75

These off-site areas are fully developed neighborhoods equipped with curb-gutters and storm sewers to drain the minor events up to the 2 to 5-yr storm into the dedicated four outfall systems along Bear Creek. Excess stormwater from major events will overflow into EDB 502L from the off-site areas. As shown in Figure 2.9, EDB 502L is composed of a forebay for sediment settlement, a trickle channel system for low flows, a WQ basin with three layers, including micropool, WQCV, and an additional storage volume above the overtopping crest. As summarized in Table 2.3, the WQCV of 0.15 acre-ft is determined based on the site imperviousness and drain time of 40 hours. In practice, we shall add 20% the site WQCV as the dead storage volume for potential sediment deposit.

3. EXCESS URBAN RUNOFF VOLUME (EURV)

Both WQCV and EURV are derived based on the runoff volume in [L^3] per unit area in [L^2] or expressed as runoff depth in [L] per catchment. As the catchment is developed, its runoff volume is increased as:

$$EURV = P_{Tr}(C_a - C_o) \tag{3.1}$$

Where P_{Tr}= design rainfall depth in inches for the selected return period of T_r in years, C_a = post-development runoff coefficient as a function of catchment's post-development imperviousness ratio of I_a, and $C_{o}=$ pre-development runoff coefficient corresponding to catchment's pre-development imperviousness ratio of I_o. As indicated in Eq 3.1, a EURV is determined with the design event and catchment's imperviousness. Relatively, EURV is much more sensitive to catchment's imperviousness than rainfall's frequency. Several empirical equations were derived to determine the EURV as (MacKenzie 2015):

$EURV_A = 1.68 I_a^{1.28}$ for hydrologic Type A soils (3.2)

$EURV_B = 1.36 I_a^{1.08}$ for hydrologic Type B soils (3.3)

$EURV_{C/D} = 1.06 I_a^{1.08}$ for hydrologic Type C/D soils (3.4)

The EURV for a given site is greater than the WQCV and close to the 10-yr detention volume. To design an extending detention basin, the bottom volume is reserved for WQCV that is to be drained over 40 hours, while the upper volume is to accommodate the EURV minus the WQCV that is to be drained over 12 to 32 hours so that the WQCV and the EURV together are drained in 52 to 72 hours (UDFCD 2016). It is important to note that Colorado law requires 97% of the 5-year event to drain within 72 hours. Using Eq 3.3, the EURV at the EDB 502 L site is determined in Table 3.1 as:

Table 3.1 EURV Determined for EDB 502L Site

EURV Calculations			Drain time	< 72 hr
Sub-catchment ID	Area acres	Imperviousness %	EURV Inch	EURV acre-ft
WQ-404	0.36	100.00	1.20	0.36
WQ-407	3.54	59.00	0.68	0.20
OS-401	1.00	100.00	1.20	0.10
Sum	4.90		3.08	0.66

During an extreme event, the minor system through the off-site areas will be overwhelmed, and the excess stormwater overflows into EDB502L. As illustrated in Figure 3.1, the outlet of EDB502L is equipped with a perforated plate to drain the WQCV over 40 hours and the EURV over 72 hours. Extreme events would flow over the overtopping weir that drains into a 10-ft by 7-ft culvert.

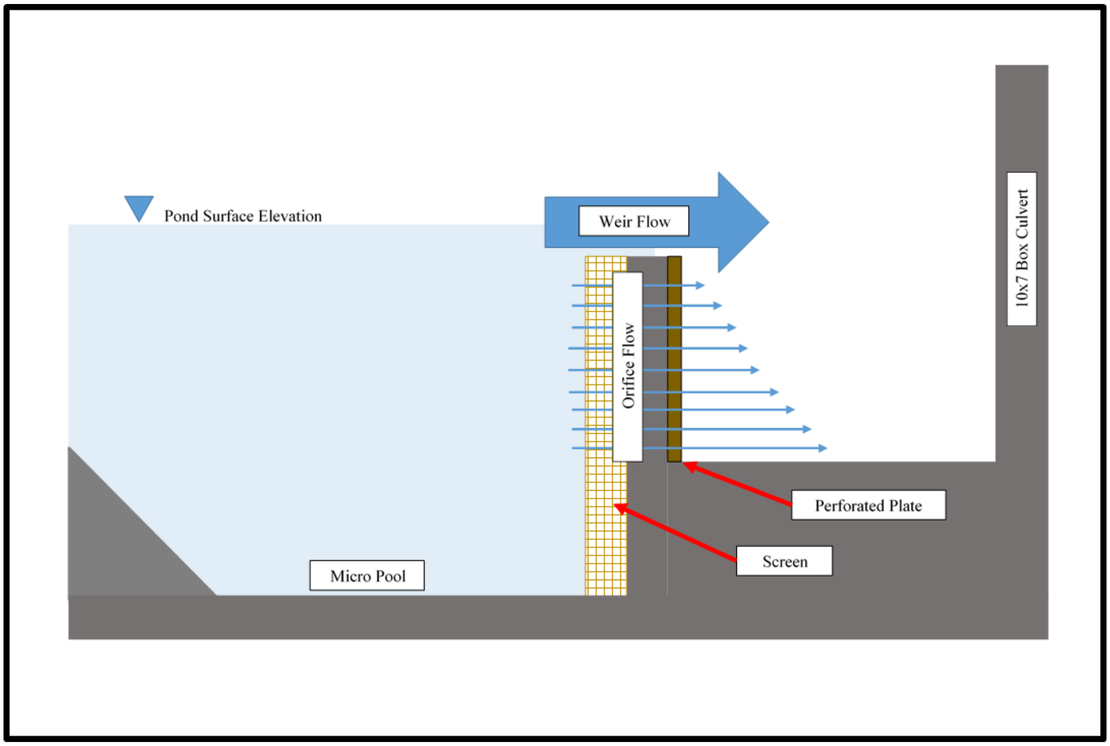


Figure 3.1 Vertical Profile for Outlet Structure in EDB 502L.

Table 3.2 As-built Stage-Area-Storage Curve for EDB 502L

Relative Elevation Ft	Depth Ft	Area ft^2	Total Volume ac-ft	Total Volume ft^3	Facilities	Remarks
5295.03	0				Bottom of Micropool	
5297.53	0	295	0	0	Bottom of Plate	
5298	0.3	330	0	93.62		
5299	1.3	1712	0.02	1024.65		
5300	2.3	4506	0.09	4023.26		
5301	3.3	8955	0.24	10627.88	Top of Plate 5300.7 ft	WQCV
5302	4.3	12160	0.49	21144.87	Overtopping Crest at 5301 ft	EURV
5303	5.3	15964	0.81	35163.98		
5304	6.3	22678	1.25	54386.85		
5305	7.3	31471	1.87	81341.55		
5306	8.3	37392	2.66	115730.52	Brim Full Elevation	
5306.2	8.5	38415	2.83	123310.99	Street Crown Elevation	

According to the Final Drainage Report on '*US Highway 285 Reconstruction CDOT Project No. BR 2854-113 Area 4 – Maintenance Yard Outfall*", the cross sectional areas for as-built EDB 502L are summarized in Table 3.2. Obviously, EDB 502L was conservatively constructed with an ample brim-full storage volume of 2.83 acre-ft which is much more than the required WQCV and EURV (Harris Kocher Smith, 2011).

4. DESIGN OF FOREBAY FOR SEDIMENT SETTLEMENT

A sediment forebay in Figure 4.1 is a small hardened basin located at the entrance of a detention basin. A forebay is designed to diffuse the inflow into a shallow, wide bay area which is usually rimmed with a riprap or/and concrete berm. A forebay acts as a pre-treatment structure shaped to settle down heavy or/and coarse particles in stormwater. A forebay can significantly reduce the regular maintenance work in the micropool. A forebay needs seasonal clean-up work to remove the sediment deposit. After a severe event, it is necessary to have field inspections and timely repairs. Therefore, on top of hydraulic efficiency, the design of forebay has to emphasize on local maintenance access and trash removal.

Forebay Pool	Forebay with Energy Dissipater

Figure 4.1 Forebay in Stormwater Basin

As shown in Figure 4.1, a forebay consists of a shallow bay area to settle solids, an overtopping wall to pass high flows, and a small floor pipe to release low flows. During an event, as soon as the bay area is filled up, excess water overtops the wall. The low flow outlet can be constructed using a floor pipe or a vertical slot on the wall. During the period of recession, the water depth in the bay area acts as the headwater to drain the stored water through the floor pipe or slot. The low flow outlet should be sized to release 1 to 2% of the 100-year peak discharge. The overtopping wall is built as a riprap/concrete berm. As expected, a forebay will be overtopped frequently. As a result, it is necessary to protect the downstream side of the berm with riprap blankets. It is preferable to have a concrete floor to cover the bay area because of frequent sediment removals. A riprap bay area in Figure 4.2 may be covered and seeded with soils. Reinforcement of fabric protection should be placed at high speed flow areas.

Vertical slot for low flow outlet

Floor pipe for low flow outlet

Figure 4.2 Low Flow Outlet and Trapped Sediment in Forebay

To improve the sediment trap efficiency, a forebay should be designed to maximize the flow length through the bay area, and minimize the floor slope to encourage particle's settlement. As illustrated in Figure 4.3, the bay area is confined with an overtopping weir that has a width, W_s, and height of H_s. For a given design flow, the overtopping weir produces a headwater depth, H_w, on top of the weir crest. As the incoming flow is diffused into the bay area, particles in stormwater begin to settle along the flow length, L_s. In general, the larger the particle is, the earlier the settlement will occur. During the resident time, if the particle can flow through the forebay before it is settled on the floor, it is a case of wash; otherwise it is a case of trap.

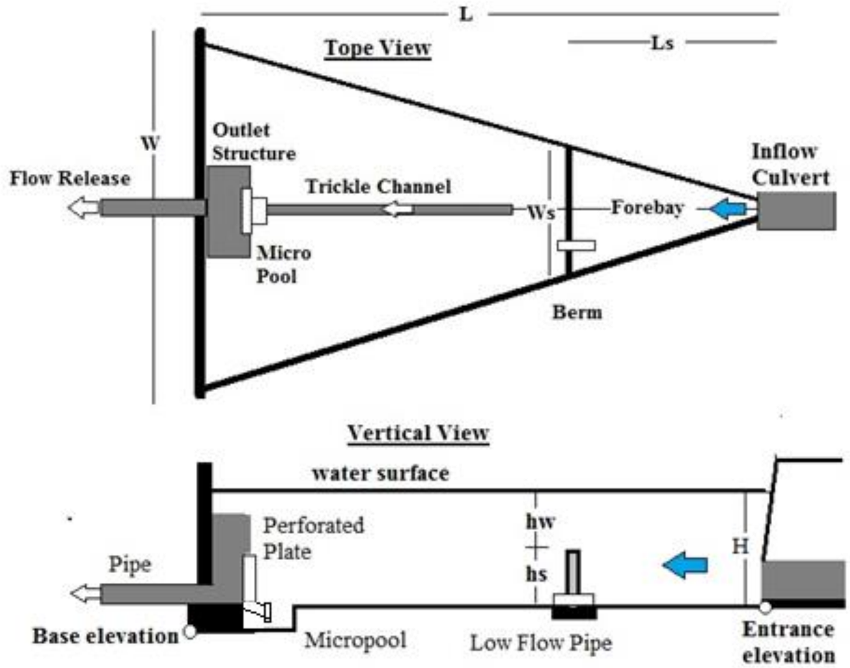


Figure 4.3 Illustration of Forebay Layout

As illustrated in Figure 4.3, particle's fall velocity is an important factor that determines the chance for the particle to be trapped or washed during the resident time.

4.1 Fall Velocity for Solid Particle in Forebay

As shown in Figure 4.4, the vertical movement of a particle in water column is dominated by the balance of the buoyance force due to the displacement of water volume, the weight due to the gravity, and the drag force due to particle's movement (Pemberton and Lara in 1971). Assuming that the particle is in spherical shape, the body weight is defined as:

$$F_g = \rho_s g \, \frac{\pi {D_s}^3}{6} \tag{4.1}$$

Where F_g= body weight in [pounds or newton], ρ_s= density of particle in [M/L^3], g = gravitational acceleration in [L/T^2], and D_s= target diameter of particle in [L] such as 1 mm. The density of soils ranges from 2.4 to 2.8 (g/cm^3). Buoyance force is equal to the weight of water volume displaced with the particle, and it can be calculated as:

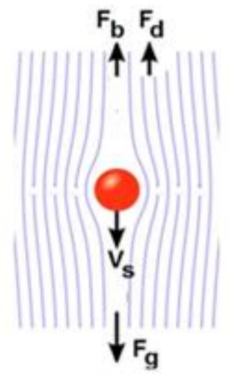


Figure 4.4 Fall Velocity

$$F_b = \rho_w g \ \frac{\pi {D_s}^3}{6}$$

(4.2)

Where F_b=buoyancy force in [pounds or newtons], ρ_w=density of water in [M/L^3]. The drag force acts in the opposite direction of the particle's movement. When the particle is settling, its drag force acts in opposition to the gravitational force, but in concurrence to the buoyancy force. The drag force is computed as:

$$F_d = C_d \ \frac{\pi {D_s}^2}{4} \frac{V^2 \rho_w}{2} \qquad (4.3)$$

Where F_d= drag force, C_d= drag coefficient, and Vs=solid particle's fall velocity. Referring to Figure 4.4, the above forces shall be balanced as:

$$F_g = F_b + F_d \qquad (4.4)$$

Aided by Eq.'s 4.1 through 4.4, the fall velocity is solved as:

$$V_s = \sqrt{\frac{4gD}{3C_d}(S_g - 1)} \ \text{in m/sec. (for metric units)} \qquad (4.5)$$

Where S_g= specific gravity of solid particle such as 2.4 to 2.8. The drag coefficient depends on particle's flow Reynolds number. For a spherical particle, the empirical formula for C_d is:

$$C_d = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + \ 0.34 \approx 0.4 \, when \ \ Re > 10{,}000 \qquad (4.6)$$

$$R_e = \frac{V_s D_s}{\upsilon} \qquad (4.7)$$

Where R_e =Reynolds number as the ratio of particle's momentum force to viscous force of water, and υ= kinematic viscosity of water such as 1.2 x 10^{-5} ft^2/sec. Using an iterative process, one can start with an estimate value, such as R_e=5,000, to calculate C_d in Eq 4.6, V_s in Eq 4.5, and then R_e in Eq. 4.7. Repeat this process until the estimated Reynolds number closely agrees with the computed. As illustrated in Figure 4.5, a particle would be captured in the forebay if the following condition is satisfied:

$$T_s = \frac{H}{V_s} \tag{4.8}$$

$$L_s \geq U_s T_s \tag{4.9}$$

Where H= water depth in forebay in [L], T_s= travel time or residence time in [T], L_s= flow length through forebay in [L], and U_s= horizontal flow velocity in [L/T]. Eq. 4.9 implies that particle's horizontal travel distance is shorter than the basin length, L_s, before the particle has landed on the floor (Simon and Senturk in 1990).

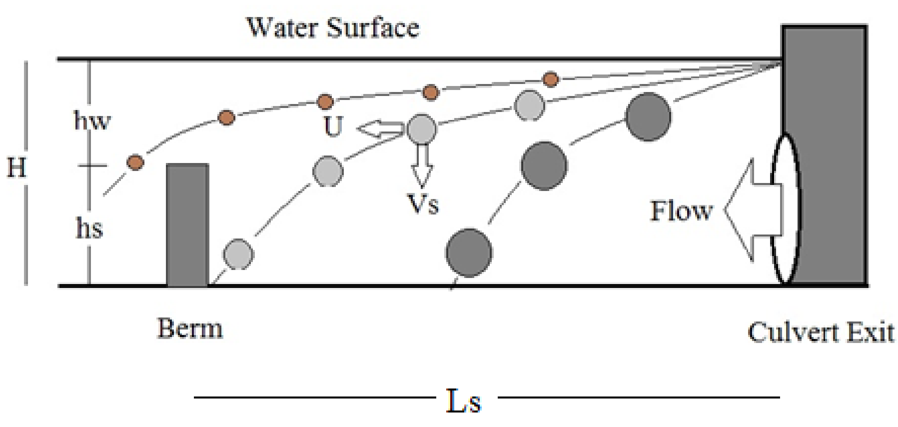


Figure 4.5 Particle Settlement in Forebay

The flow horizontal velocity, U_s, is the cross sectional average velocity calculated as:

$$U_s = \frac{Q}{HW} \tag{4.10}$$

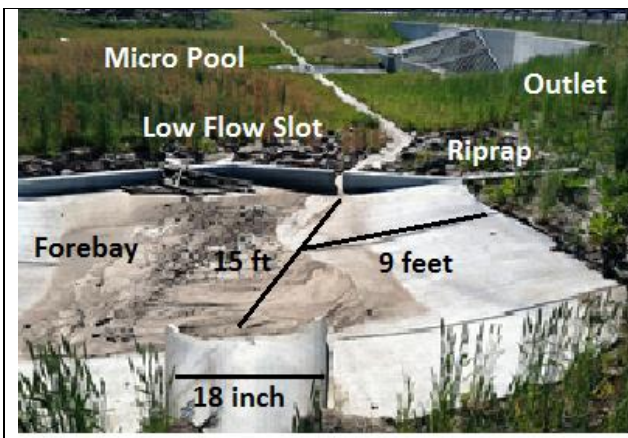
$$Q = \frac{2}{3} C_o \sqrt{2g} W\, h_w^{1.5} \tag{4.11}$$

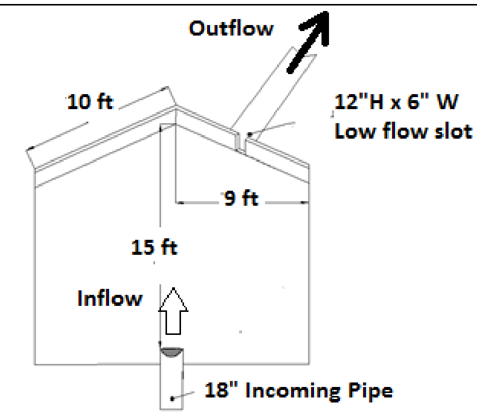
$$H = h_w + h_s \tag{4.12}$$

Where Q= design discharge in [L^3/T], W= width of berm in [L], h_w= headwater depth in [L] for weir flow on top of berm, and h_s= height of berm in [L]. In practice, the sediment load is depicted with a gradation curve. Therefore, we have to apply Eq.'s 4.1 through 4.12 to a selected particle size, D_s, to determine if any particles $\leq D_s$ would be settled in the basin.

4.2 Field Investigation

The existing EDB 502L built at S Knox Court and HW 285 is selected to investigate the aforementioned new procedure developed for the design of forebay. As shown in Figure 4.6, the as-built forebay is located at the west entrance into EDB 502L. The incoming circular pipe is a circular 18-inch concrete pipe. The concrete forebay is approximately 15-ft long, 18-ft wide, and 12-inch deep. A vertical cut through the overtopping wall serves as the low flow pass. The overtopping apron is protected with grouted riprap.





Inflow Pipe to Forebay at West Entrance	Outflow from Forebay to Micropool

Figure 4.6 As-built Forebay at EDB 502L

As illustrated in Figure 4.7, two (2) sets of sediment samples were collected downstream of the forebay, and another three (3) sets of sediment samples were collected upstream of the forebay. To produce particle gradation curves from the samples, standard sieve analyses were conducted in the Soil Engineering Laboratory University of Colorado Denver using eight different sieve sizes, including #4 (4.75mm), #10 (2.0 mm), #16 (1.18 mm), #30 (0.6 mm), #40 (0.425 mm), #50 (0.3mm), #100 (0.15mm), #200 (0.075mm), and the sieve pan. Each empty sieve was first weighed and recorded prior to the test. All chunks and clots within the sample were ground into individual constituent parts. The entire sample was poured into the top sieve, and the sieve stack was continually shaken for 15 minutes. Each sieve was then weighed and recorded to measure the weight of the retained sediment sample. Figure 4.8 presents gradation curves derived from the above procedure. The average was then taken to produce the representative upstream and downstream sediment particle graduation curves.

The incoming pipe to the forebay is an 18-inch circular concrete pipe laid on a slope of 1%. The flow full capacity through this 18-inch circular pipe is determined to be 9.1 cubic ft per second (cfs). This design flow will overtop the downstream weir. The horizontal length, L_s, in the forebay is 15 feet. The overtopping weir has a width, W_s, of 20 feet and height, H_s, of 12 inches. With these as-built parameters, the sediment load collected upstream of the entrance was processed through the forebay. As shown in Table 4.1, the sediment trap efficiency is determined to be 90% or any particles≥0.2mm will be settled in the forebay pool.

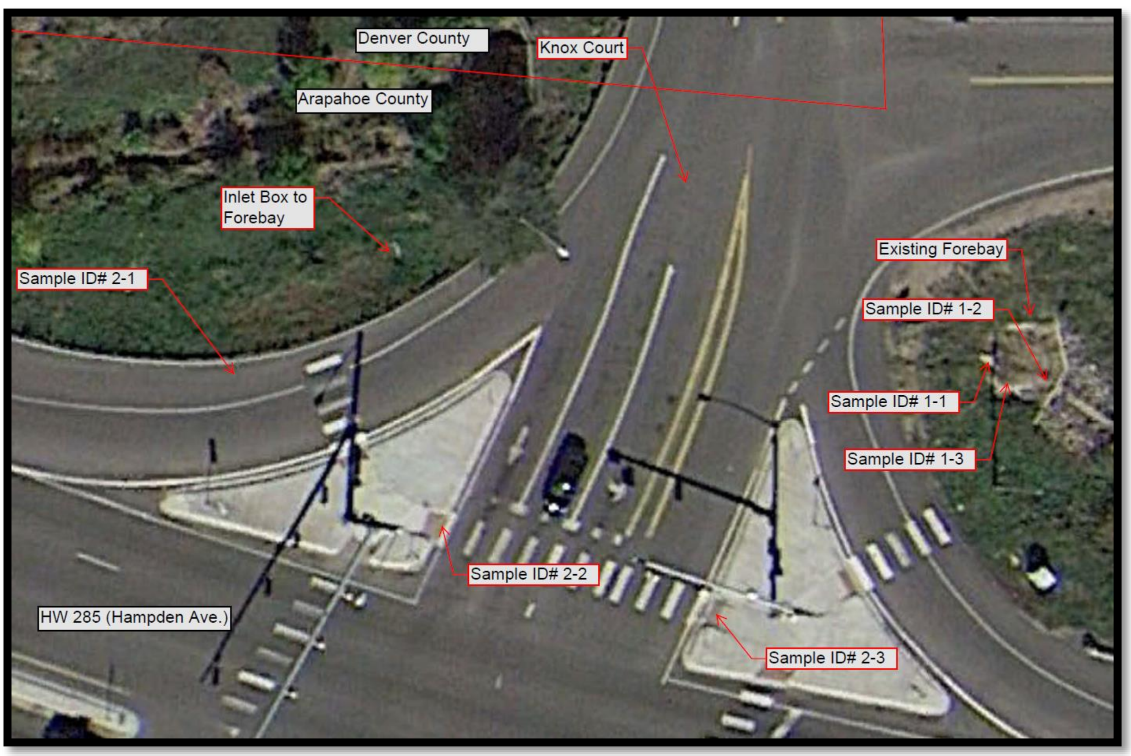


Figure 4.7 Locations of Sediment Samples at EDB 502L

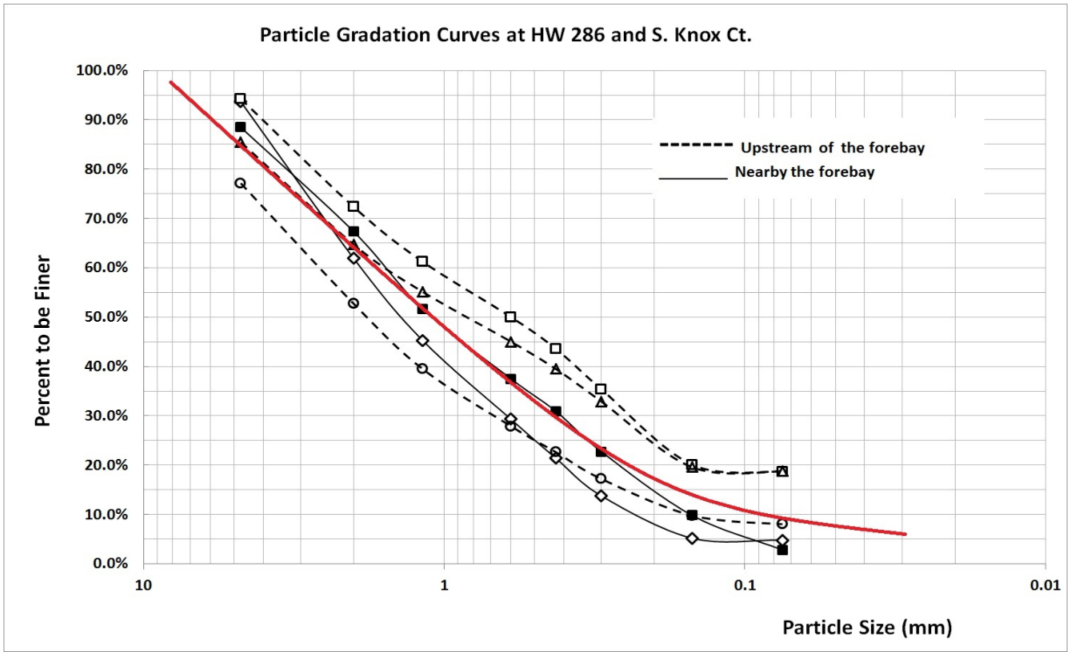


Figure 4.8 Gradation Curves for Sediment Samples at EDB 502L Site

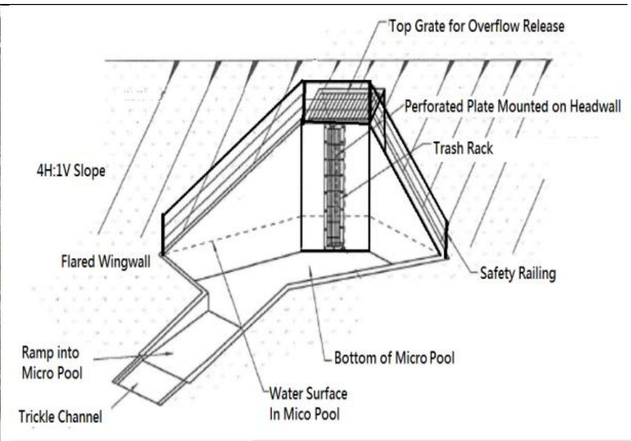
Table 4.1 Analysis of Sediment Trap Efficiency at Forebay EDB 502L

Incmoing Flow through Incoming Pipe	Q =	9.11	cfs
Discharge Coefficient for Overtopping Weir	Cw =	0.65	
Width of Overtopping Weir	Ws=	20.00	ft
Height of Overtopping Weir	Hs =	12.00	inch
Headwater Depth on Top of Weir Crest	Hw=	3.09	inch
Water Depth in Forebay	H=	15.09	inch
Average Specific Gravity of Sediment Particles	Sg =	2.56	
Width of Forebay	W =	18.00	ft
Cross Sectional Flow Area in Forebay	A =	16.98	ft^2
Design Peak Flow Through Forebay	Q =	9.11	ft^3/s
Water Flow Velocity Through Forebay	U =	0.54	ft/s
Horizontal Length of Forebay	Ls =	15.00	ft

Particle Size	Percent Finer %	Reynolds No.	Drag Coeff	Fall Velocity V_{Fall}	Settlement Analysis	% of Particles Trapped in Forebay	Diff in Re
mm	%			m/s		%	Check
10.000	100.00%	6084.169	0.382	0.730	settled	0.0%	0.000
5.000	85.00%	2057.966	0.418	0.494	settled	15.0%	0.000
2.000	65.00%	461.527	0.532	0.277	settled	35.0%	0.001
1.000	50.00%	135.080	0.776	0.162	settled	50.0%	0.000
0.500	35.00%	33.530	1.574	0.080	settled	65.0%	0.000
0.300	25.00%	10.295	3.606	0.041	settled	75.0%	0.000
0.200	18.00%	3.657	8.472	0.022	settled	82.0%	-0.001
0.100	10.00%	0.537	49.129	0.006	settled	90.0%	0.000
0.080	9.00%	0.249	96.399	0.004	washed	90.0%	0.025
0.030	5.00%	0.017	1417.100	0.001	washed	90.0%	-0.001

5. MICROPOOL FOR INVERTED SIPHON FLOW

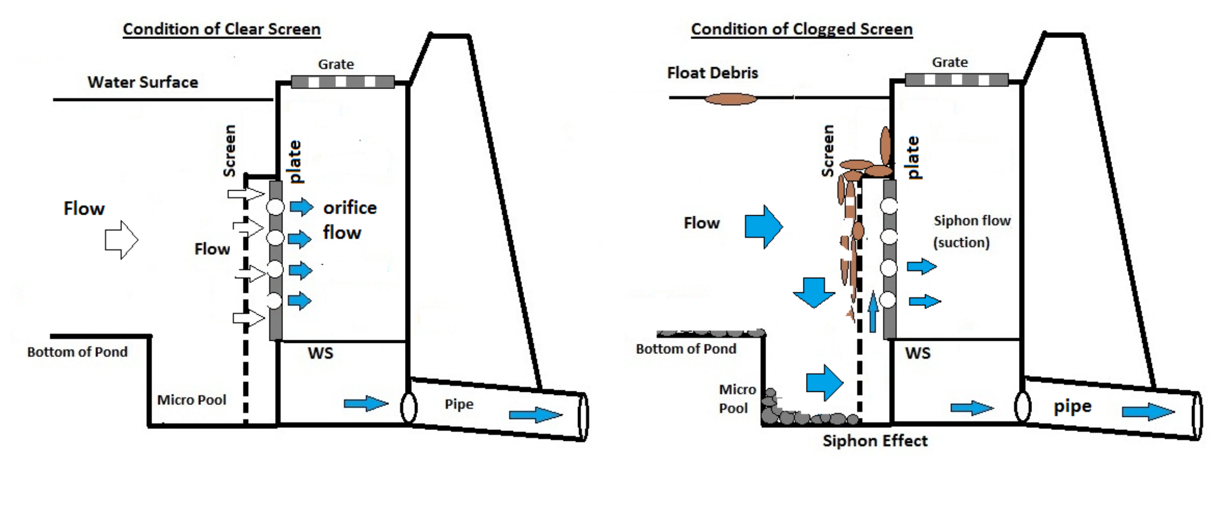
Between two adjacent storm events, the basin remains dry. Urban debris and dry leaves are built up in the basin. As soon as flood water enters the basin, light debris float up with the rising water. As shown in Figure 5.1, floating debris tends to flow around the outlet structure. As soon as the outflow devices become clogged, standing water will be built up in the basin. Often a screen or a trash rack is installed in front of the perforated plate to reduce the clogging potential, and the micropool acts as an inverted Siphon to provide a suction flow in case of clogging.



Micropool in front of Outflow Structure	Micropool and Siphon Flow

Figure 5.1 Micropool in As built EDB 502L.

As shown in Figure 5.2, a micropool is a sunken wet pool that is built in front of the outflow structure that can act as a submerged inverted siphon device. The submerged perforated plate may act as an inverted Siphon that will continually drain water as soon as the hydraulic head is developed due to clogging around the outlet structure. The standing water in the basin produces a sufficient headwater depth to lift water through the gap between the screen and the plate. Such Siphon flow continues until the basin is emptied out (Guo, Shih and MacKenzie 2012).



Clear Screen	Clogged Screen

Figure 5.2 Clear and Clogging Drainage Conditions

5.1 Size Gradation Curve for Float Debris

Figure 5.3 shows the evidence of clogging on the screen in a micropool. Algae may grow in the micropool. In this study, the distribution of particle sizes in float debris was analyzed based on the dried cakes scraped from the screens in front of the perforated plates in several urban detention basins near the site of EDB 502L (Mendi 2012).

Debris Cake Intercepted by Screen	Sediment Captured in Forebay

Figure 5.3 Sediment Removal at Forebay and Micropool

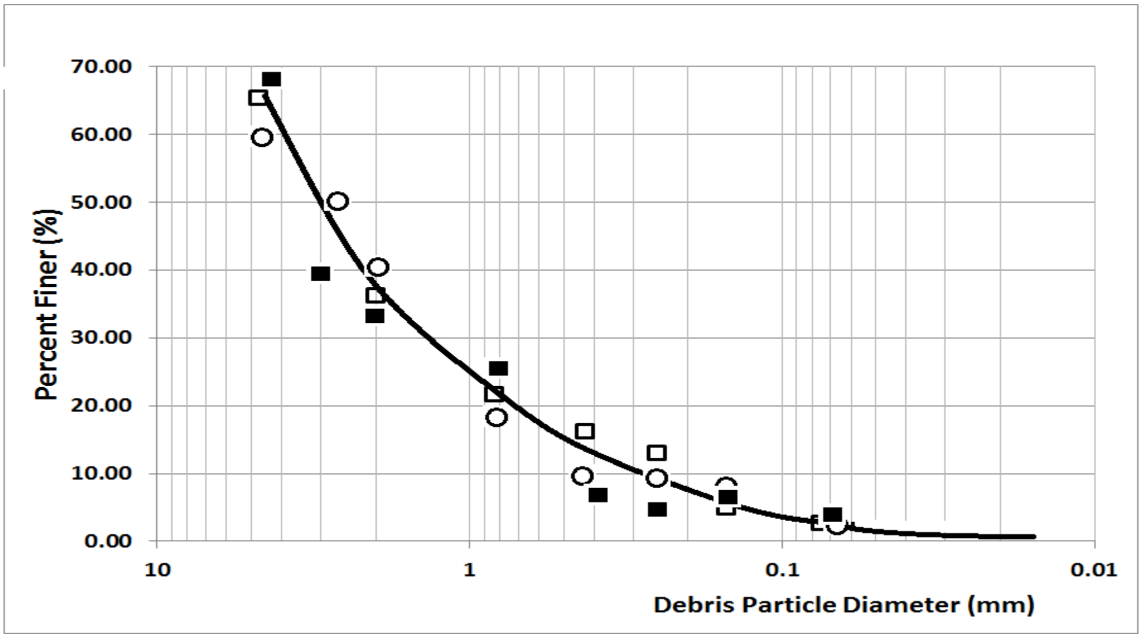


Figure 5.4 Distribution of Particle Size for Float Debris

Several sets of samples were analyzed and plotted in Figure 5.4. Particles in the samples are ranged from 0.01 to 5 mm. Considering that the screen shall intercept 90% of floating particles, the threshold particle size is determined to be D_b =0.3mm based on Figure 5.4. Any particles $\geq D_b$ would be intercepted by the screen in front of the perforated plate. The field sample was also investigated in the laboratory for determining its specific gravity. It was found that the specific gravity of float debris collected in detention basins highly depends on its moisture content. For saturated float debris, the specific density ranges 0.8 to 0.9.

5.2 Float Velocity for Debris Particle

The forces acting on a floating particle as illustrated in Figure 5.5 include the downward body weight and drag forces that are balanced with the upward buoyance lift. The water flow around a floating particle moving upward at a velocity, V_b, is unsteady. To convert such an unsteady flow into a steady flow, we need to "freeze" the moving particle. Adding a downward velocity, V_b, to the entire flow field as illustrated in Figure 5.5, the steady flow pattern is a downward water flow around the stagnant particle. This downward velocity is equivalent to the siphon flow that goes through the micropool surface.

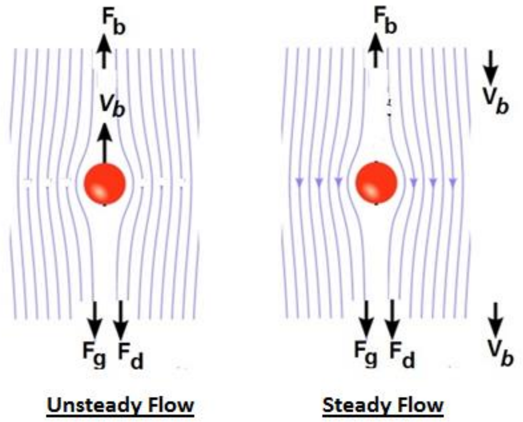


Figure 5.5 Flow Field around Floating Particle in Water

Aided with Eq.'s 4.1, 4.2, 4.3, the upward floating velocity for debris particle is derived as:

$$F_b = F_g + F_d \quad (5.1)$$

$$V_b = \sqrt{\frac{4gD_b}{3C_d}(1 - S_b)} \quad (5.2)$$

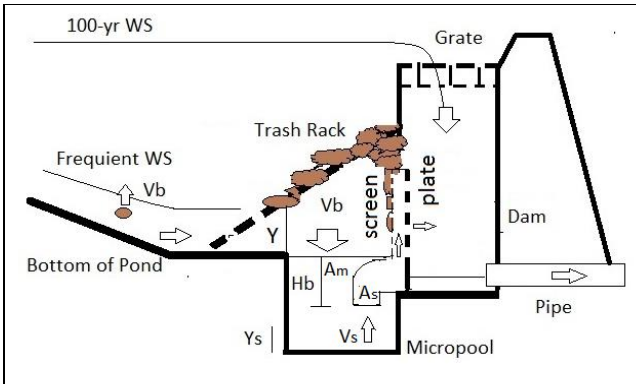
Where V_b = float velocity in [L/T], D_b= minimum particle size in [L] that is allowable to flow through the perforated plate, such as 1.0 mm, and S_b= specific gravity in [L/L] of saturated debris float such as 0.8 to 0.9.

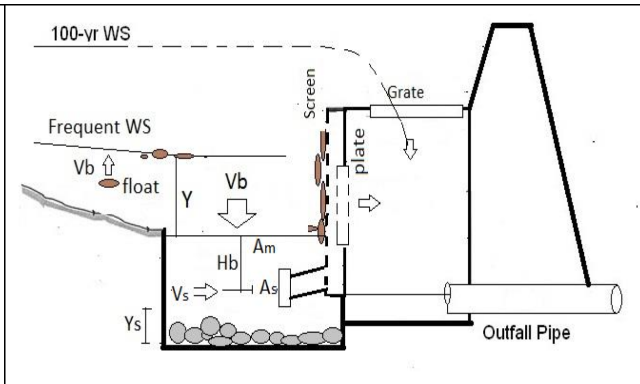
5.3 Design of Micropool

A micropool is a back-up system in case of clogging around the outflow structure. The main purpose of a micropool is to warrant the drain time and flow release rate under the design condition. Usually, a detention basin is not clogged during an extreme event because of its huge flow volume with diffused debris loads. An outflow structure is, in fact, more vulnerable to debris clogging during a series of small events in the magnitude of 3 to 6-month events. The trickle flow continually carries debris and trash into a basin. As soon as the outlet plate is getting clogged, the basin will accumulate standing water that tends to become a mosquito bed. A micropool is designed to provide a suction head to produce a continuous siphon flow to drain the accumulated runoff volume. Consider the WQCV as the target volume for micropool design. Usually the drain time for WQCV is 40 hours. Therefore, the average release rate is defined as:

$$q = \frac{WQCV}{T} \tag{5.3}$$

In which q= average release in [L^3/T], WQCV=water quality capture volume in [L^3], and T=drain time in [T].





Up-sloped Siphon Pipe	Reversed Siphon Pipe

Figure 5.6 Types of Siphon Pipes to Lift Water Flow

The flow release, q, has to go through the surface area of the micropool. Thus, the surface area, A_m in [L^2], of the micropool is determined as:

$$A_M = \frac{q}{V_b(1 - C\log)}$$

(5.4)

(5.4)

Where A_M= micropool surface area in [L^2], Clog= area clogging ratio due to algae in micropool such as 0.5. As soon as it rains, the micropool is the first low spot to be filled up and the siphon device in Figure 5.6 is submerged and primed with its surcharge depth, H_b. As the water surface rises in the detention basin, floating debris will be built up on the screen in front of the perforated plate. After the screen becomes plugged, a standing water depth, Y in [L], is developed in the basin. The suction flow is calculated as:

$$q_S = C_o(1 - C\log)A_S\sqrt{2g(H_b + Y)} \tag{5.5}$$

Where q_S= suction flow in [L^3/T] equal to the flow release rate, q in Eq 5.3, C_o= discharge coefficient, A_S= opening area of siphon flow in [L^2], g=gravitational acceleration in [L/T^2] such as 32.2 ft/sec^2 or 9.81 m/sec^2, Y= standing water depth in [L], and H_b= specified surcharge depth in [L]. To be conservative, the cross sectional area, A_S, of the siphon device is determined with Y≈ 0 as:

$$A_S = \frac{q}{C_o(1 - C\log)\sqrt{2gH_b}} \tag{5.6}$$

Considering the dead storage for settled solids, and evaporation loss, the depth of the micropool shall be not less than:

$$H_M = H_b + Y_S + Y_V \tag{5.7}$$

Where H_M= depth of micropool in [L], Y_S= dead storage depth in [L] for sediment deposit in [L], Y_V=evaporation depth in [L]. Evaporation rate, E_V in [L/T], is local and seasonal. The proper evaporation depth shall be estimated for the wet months in a year. From the local hydrology, the average inter-event time, T_I, in Table 2.1, provides a basis to estimate the required evaporation depth as:

$$Y_V = E_V T_I \tag{5.8}$$

Evaporation rates can be found in **http://www.wrcc.dri.edu/htmlfiles/westevap.final.html** for states in the US.

5.4 Field Investigation

EDB 502L in Figure 2.9 provides the required WQCV for 3 sub-catchments, including WQ404, WQ 407, and OS401. In fact, and Sub-catchment, OS6, also directly flows into EDB 502L. Therefore, a total of 11 acres with the weighted imperviousness percentage of 75% is taken into consideration for the design of micropool. The WQCV is determined to be 0.27 inch. Adding 20% of sediment deposit volume, the WQB shall have a storage volume of 0.33 acre-ft. Set the drain time to be 40 hours. The average release discharge is 0.1 cfs. The screen of the perforated plate is designed to intercept up to 90% of floating particles or only float particles<0.3 mm may pass through the screen. According to Figure 5.4, D90=0.3 mm. Consider other design information, including Co=0.6, Clog=0.5, S_b=0.83, T_I=7 days, E_V=0.35 inch/day, and H_b=1.0 foot. The design of micropool is summarized in Table 5.1.

Table 5.1 Design of Micropool for EDB 502 L

WATERSHED INFORMATION					
Catchment Tributary Area			A=	11.00	acre
Catchment Imperviousness %			*Ia=*	75.00	
Drain Time			Td=	40.00	hr
WQCV from the WQCV Calculator			1.2*WQCV=	0.33	ac-ft
Average Design Flow Rate			Qo=WQCV/Td	0.100	cfs
FLOAT VELOCITY					
Float Specific Gravity			Sb=	0.83	
Size of Float Particle			D90=	0.30	mm
Water Viscosity			V=	0.0000012	m^2/s
Analysis of Float Velocity					
Float Size	Reynolds No.	Drag Coeff	Float Velocity	Diff in Re	
mm	Guess		m/s	Check	
0.300	1.481	19.013	0.006	0.00	
	Float Velocity for particle		Vb=	0.019	ft/sec
SYPHON CAPCITY					
Orifice Coefficient Applied to Screen holes			Co=	0.60	
Clogging factor due to Algae			Clog=	0.50	
Surcharge depth			Hb=	1.00	foot
Suction Flow Velocity			Vs=	8.02	fps
Flow Area for Syphon Flow			As=	0.041	sq ft
GEOMETRY OF MICRO POOL					
Evaporation Rate			E=	0.35	in/day
Interevent Time			Inter Time=	7.00	days
Sediment Dead Storage			Dsedi=	1.00	ft
Surface Area			Am=	10.26	sq ft
Depth for Micro Pool			Ym=	2.20	ft

For this case, the micropool should have a surface area of 10.26 sq feet, and a depth of 2.20 feet. The flow movement driven by the inverted Siphon effect is as slow as 0.019 ft/second through the micropool surface area. The Siphon opening area should be 0.041 sq feet (2 sq inches). The as-built micropool has a surface area of 6 by 12 sq feet and a depth of 2.5 feet. In fact, 15% of the as-built micropool is sufficient for this case.

6. UNCERTAINTY OF RESIDUAL WATER DEPTH

The effectiveness of trapping solid settlement in a WQB depends on the resident time or the drain time of the basin. In general, the longer the resident time is, the higher the particle trap ratio will be. On the other hand, a longer drain time imposes a higher risk of overtopping the basin due to subsequent storm events. Many local governments in the Denver Metro area have adopted freeboard guidelines to add more storage volume to accommodate the potential residual water depth. Currently, freeboard guidelines vary greatly due to the uncertainty to quantify the residual water depth, including, but not limited to, the following:

> *City and County of Denver*
>
> "For sites greater than or equal to 5 acres, the elevation of the top of the embankment shall be a minimum of 1 foot above the water surface elevation when the emergency spillway is conveying the maximum design or emergency flow. For sites less than 5 acres, the minimum required freeboard is 1.0 foot above the computed 100-year water surface elevation in the detention facility." (City and County of Denver, 2006)
>
> *Adams County*
>
> "Adams County also requires that 50% of the WQCV be added to the calculated 100-year Volume. An additional one-foot of depth must be added to the overall volume to accommodate for freeboard. Administrative relief for exemptions or reductions in freeboard requirements may be granted by the Director of Public Works." (Adams County, 2011)
>
> *City of Aurora*
>
> "All detention facilities for new developments or redevelopments that disturb greater than or equal to five acres shall be designed to include the UDFCD's Full Spectrum Detention volume and the 100-year detention volumes in the following

manner. Up to one half of the Full Spectrum Detention may be included within the 100-year detention volume flood pool." The minimum freeboard for open space detention facilities is one-foot (1.0') above the computed 100-year water surface elevation. The emergency overflow weir sill shall be set at the freeboard elevation." (City of Aurora, 2010)

6.1 Numerical Algorithm to Reproduce Residual Water Depths

In this study, the hourly rainfall data base recorded at the Stapleton Airport, Denver, Colorado was used to analyze the residual water depths. Consider each hourly rainfall depth as an individual and independent event. Each rainfall event produces its own runoff volume that drains into the detention basin. If the basin is empty, the residual water depth is none; otherwise the initial water depth at the basin is recorded for further statistical analysis. To analyze the continuous rainfall events, set the counter I to cover I=1 for the first event to I=last event number in the data base. The following numerical algorithm is derived to reveal the residual water depth at the beginning of each hourly rainfall event.

Select the drain time, T_D, for the WQCV determined at the site:

$$q = WQCV / T_D$$

Set i=1 to i=last event for a loop calculations

$$T_I(i+1) = T(i+1) - T(i)$$

$$V_R(i) = \max[0, V_{SI} - (q + E_v / 24) T_I(i)]$$

$$V_I(i) = V_R(i) + C[D(i) - I_s]$$

$$V_q(i) = \max[V_I(i), qT_d]$$

$$If\,[V_I(i) > V_q(i)] > WQCV, V_{OV}(i) = V_I(i) - V_q(i) - WQCV, \text{ } otherwise \text{ } V_{OV}(i) = 0$$

$$V_S(i) = \min[WQCV, V_I(i) - V_q(i)]$$

$Let\ \ V_{SI} = V_S(i)$; repeat the loop for the next hourly rainfall event.

Where T = the beginning hour for the event I, T_I = inter-event time in hours, V_R= residual water depth in inches, V_{SI} = stored runoff volume in inches at the end of previous hourly rainfall event, q= average release rate from WQB at inch/hr, E_v= daily evaporation rate in inch/day, C= runoff coefficient for the tributary catchment, D= hourly rainfall depth in inches, I_s = initial loss for each hourly event ranging from 0.05 to 0.1 inch, T_d= duration for each event or one hour in this study, V_q= volume in inches flowing through the basin during the hourly event, V_{OV}= overtopping volume in inches, and V_S= stored runoff volume in inch at the end of hourly rainfall event. As soon as it rained, the initial water depth is recorded as the residual water depth associated with the rainfall event. The total incoming loading to the basin is the rain-runoff volume from the current event plus the residual volume from the previous event. The total runoff volume is then released at the average rate, q, for one hour. The remaining volume in the basin is continuously accumulated to the next hourly rainfall-runoff depth. During the inter-event time, the last hourly rainfall-runoff volume in the basin is continuously released to become empty or mixed with the incoming rainfall event if the inter-event time is too short. This algorithm is repeated through a long-term hourly rainfall record to construct the population of residual water depth at a given basin site.

6.2 CASE STUDIES

From August 1949 through December 1979, the hourly rainfall depths were recorded at the Stapleton Airport, Denver, Colorado. This continuous rainfall record of 30 years long was employed to reproduce the distribution of residual water depths at a WQB with a drain time of 40 hours and an imperviousness of 75% in the tributary area. Table 6.1 is the summary of the input parameters for this case study. The data base covers a total of 258,888 hours in which there were 10,835 rainy hours. Set I= 1 to I=10,835 or a total of 10,835 independent hourly events were analyzed. From 1948 through 1978, the inter-event time varied from 1 to 1179 hours with an average of 23 hours. The longest continuous rainy period lasted 66 hours in the year of 1965.

Table 6.1 Input Parameters for Analyses of Residual Water Depth

Watershed imperviousness	75.00	%	Total hours in Data Base	258888	hours
Runoff Coefficient	0.54		Total rainy hours	10835	hours
Initial loss	0.00	inch	Average Release Rate	0.007	inch/hr
Watershed WQCV	0.289	inches	Average Interevent	23	hours
Event Separation Time	6.00	hours	Average Residual Vol	0.052	inch
Pond Drain Time	40.00	hours	Max Residual Vol	0.282	inch

Figure 6.1 presents the distribution of residual water depths normalized by the basin depth. For the case with imperviousness of 75% and drain time of 40 hours, the WQB in Denver would have a 5% chance of being full when needed for the next storm due to continuous accumulation of rainfall-runoff volumes into the basin. Similarly, there is a 15% chance of a WQB being half full for the next hourly rainfall event.

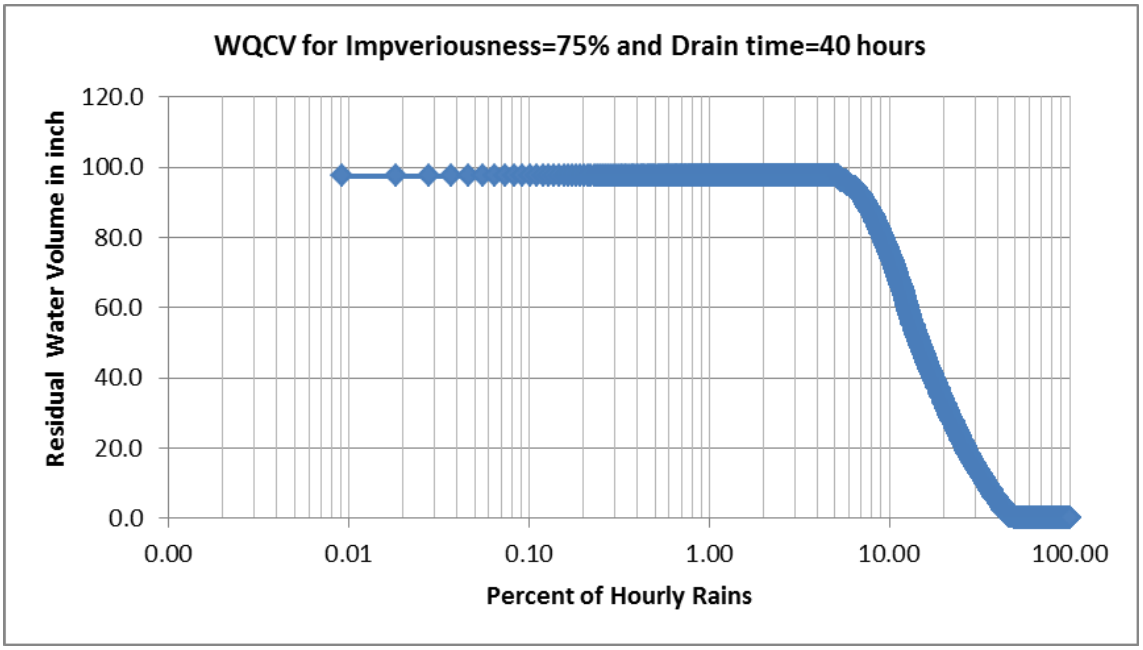


Figure 6.1 Normalized Residual Water Depth for WQCV with Imp=75% Drain Time=40 hr

Figure 6.2 presents the study on the distribution of normalized residual water depths for EURV basins. For the case with imperviousness of 75% and drain time of 72 hours, a EURV basin in Denver would have a chance of 0.1% to become 90% full due to continuous accumulation of rainfall-runoff volumes in the basin. Similarly, a chance of 1% is expected to have a EURV basin being half full before the next hourly rainfall event.

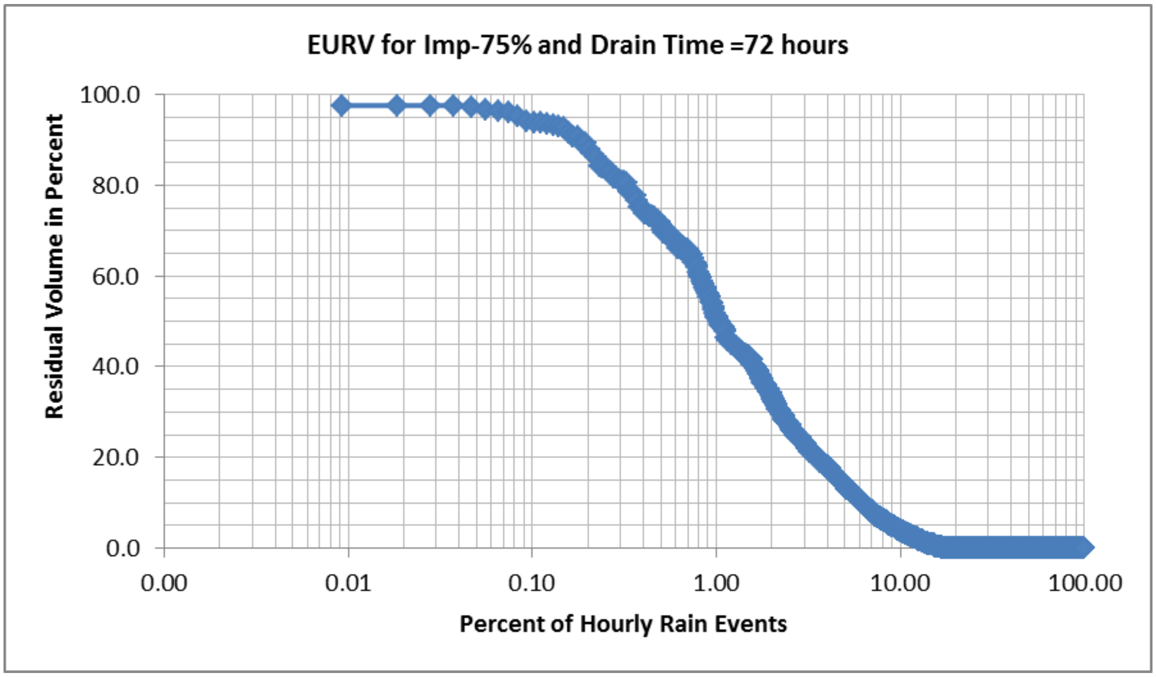


Figure 6.2 Normalized Residual Water Depths for EURV with Imp=75% Drain Time=72 hr

6.3 FIELD INVESTIGATION

The Urban Drainage and Flood Control District (UDFCD) installed pressure gages at EDB 502L to record the water depths during the summer in 2015. From the UDFCD's ALERT system, the rainfall record was retrieved from the Westwood Rain Gage Station to match with the runoff depths recorded at the site. The observed rainfall distribution was coded into EPASWMM to produce corresponding runoff hydrographs from all sub-catchments. The link-node system developed for the entire watershed as a tributary area to EDB 502L is illustrated in Figure 6.3. There are 4 outfall pipes from the site to Bear Creek. Each outfall pipe is analyzed with culvert hydraulics to establish the stage-outflow curve. EDB 502L is equipped with a perforated plate as shown in Figure 6.4. The stage-outflow curve derived for EDB 502L is plotted in Figure 6.5. This curve consists of three (3) segments, including no flow release from the micropool, orifice flow from the as-built perforated plate, and weir flow overtopping the wall into the 10-ft by 7-ft culvert.

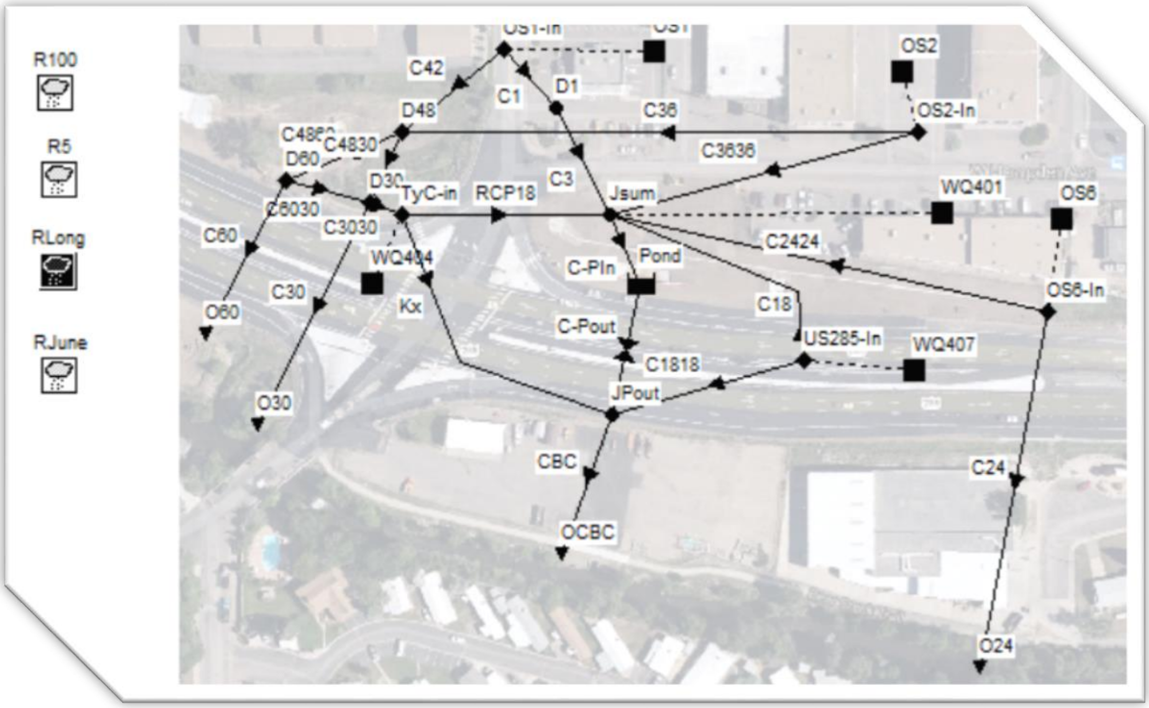


Figure 6.3 Link-Node Model for EDB 502L Test Site.

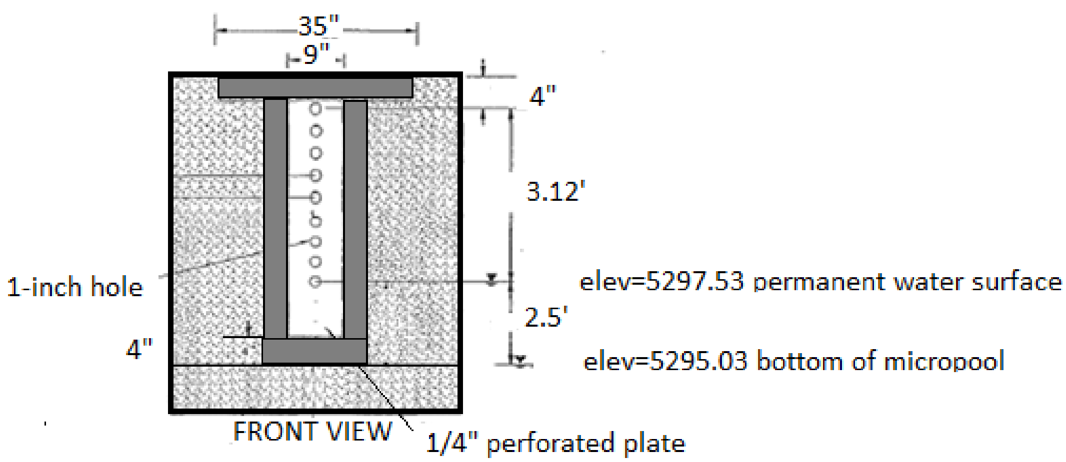


Figure 6.4 As built Perforated Plate at EDB 502L.

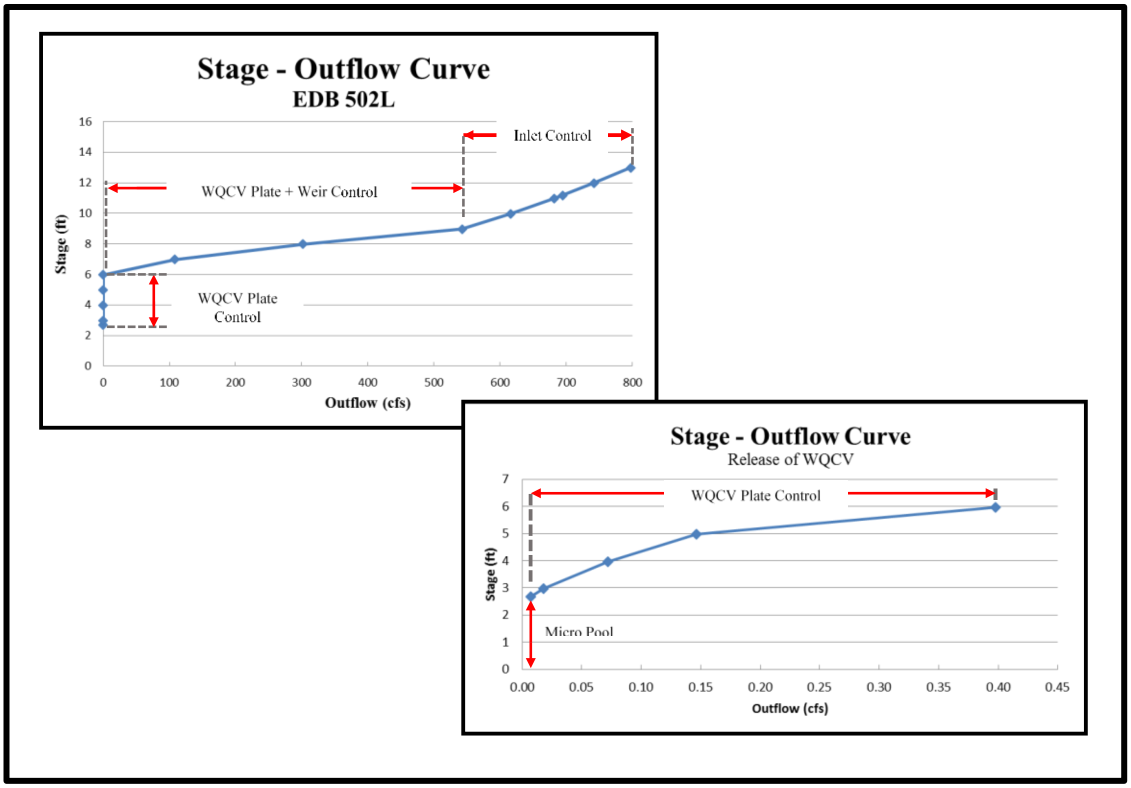


Figure 6.5 Stage-Outflow Curve Developed for EDB 502L

Figure 6.6 presents the predicted water depths at EDB502L during the continuous storm events from 7/6/2015 8:00 to 7/9/2015 20:00 using EPA SWMM5. In general, predicted water depths in the basin follow the same pattern as the observed rainfall distribution. However, between events, the numerical predictions tend to give higher residual water depths. This fact may be due to inadequate representations of field evaporation rates which should vary with respect to daily temperature, namely higher rates during the day time and lower rates during the night time. A single daily evaporation rate in EPA SWMM5 is insufficient to represent the field evaporation loss.

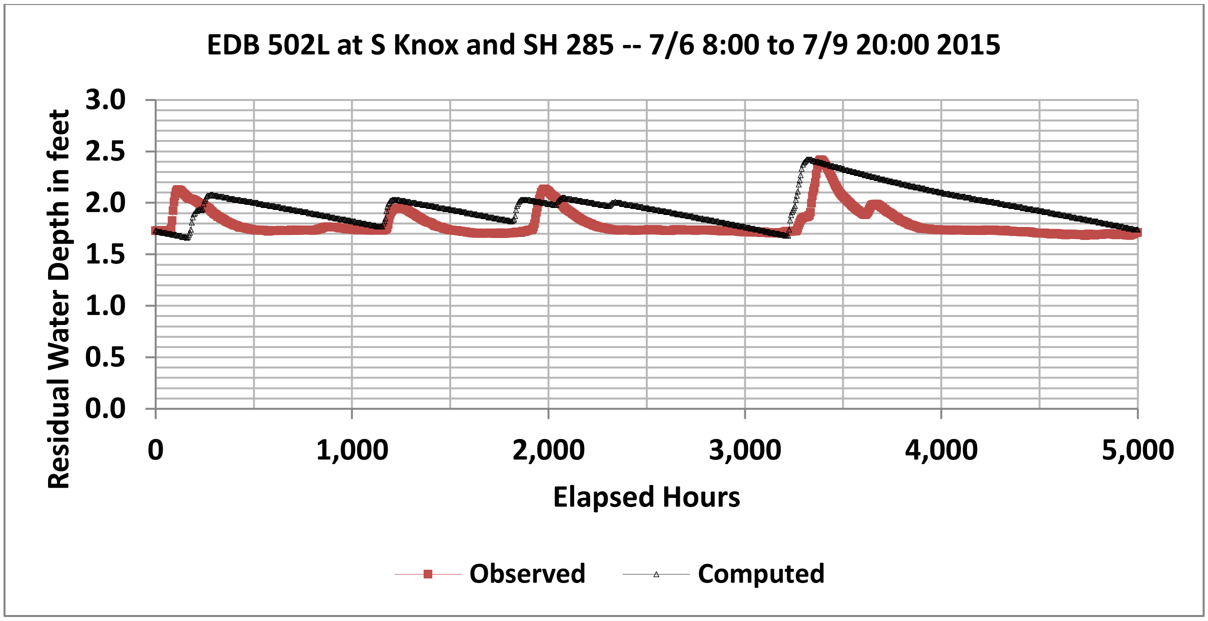


Figure 6.6 Continuous Rainfall-Runoff Simulations at EDB 502L Test Site.

Next, a long-term continuous simulation is conducted to reproduce the residual water depths at EDB 502L from 1948 through 1978. A daily evaporation rate of 0.25 inch/day was applied to the water surface at EDB 502L. The operation of the detention basin in the model is prescribed by the user-defined storage-outflow curve. Again, on an hourly basis, the array of continuous hourly runoff depths was produced from EPASWMM5 and plotted in Figure 6.7. From 1948 to 1978, the chance for EDB 502L to become 90% full is approximately 3% while the chance for being half full is 20%.

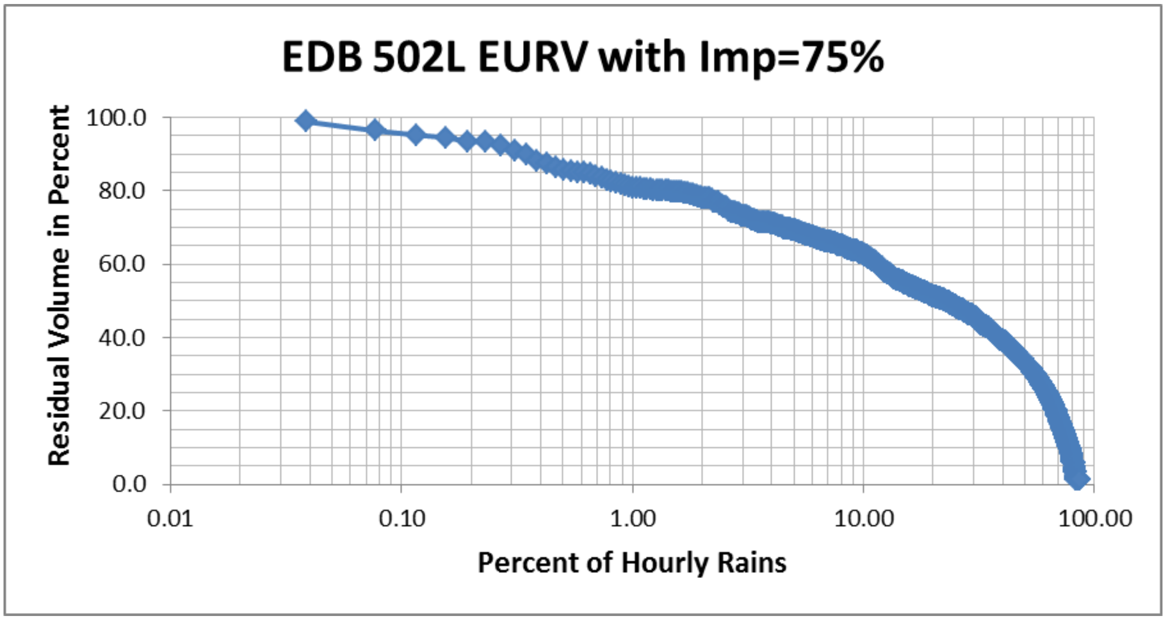


Figure 6.7 Normalized Residual Water Depths for EURV at EDB 502L

Both Figures 6.6 and 6.7 define the exceeding probability for various magnitudes of residual water depths. They offer a risk-based approach to define the required freeboard.

7. CONCLUSION

1. A new procedure was developed to design a sediment forebay at the entrance of a stormwater detention basin. It was found that the geometry of the forebay depends on the size of the target particle for settlement in the forebay. The fall velocity of the target particle and the horizontal inflow velocity provide the basis to size the length and depth of the forebay pool. The required pre-knowledge is the on-site sediment gradation curve. In this study, samples were collected at locations upstream and downstream of the forebay in EDB 502L. From the gradation curve derived from the soil laboratory, the target particle is recommended to have a diameter of 0.1 to 0.2 mm. It is expected that a forebay will intercept 60 to 70% of sediment load in stormwater.

2. A micropool is designed to provide a continuous suction flow to drain the remaining water in the detention basin after the outlet screen become clogged. In this study, dry samples of float deposits were collected from the screens in detention basins, and they were analyzed in the soil laboratory to construct the gradation curves. It was found that

D90=0.3 mm and the saturated specific gravity is 0.8 to 0.9. The new concept of float velocity was derived in this study. Using the float velocity as the basis, the micropool surface area and flow depth can be determined. As recommended in this study, the flow depth in micropool shall include evaporation loss through the period of inter event time, dead storage for solids, and suction head to sustain the flow. The screens covering WQCV outlet plates have long been a maintenance issues due to clogging and UDFCD now recommends only three or four orifices to maximize the orifice size and minimize clogging of the orifice plate.

3. In this study, a residual water depth in a detention basin is defined as the remaining water depth from the previous hourly event. The fluctuation of residual water depths depends on the storage volume, WQCV or EURV, and their drain times. In this study, a total of 10835 hourly rain events were identified from a total of 258,888 hours from 1949 through 1979. Considering that we had 10835 independent rainfall events over 30 years, the average chance of having a rainfall event per day is 4% in the Metro Denver area. From 1948 through 1978, the inter-event time varied from 1 to 1179 hours with an average of 23 hours. The longest continuous rainy period lasted 66 hours in the year of 1965.

Sample tests on residual water depth were conducted for WQCV with a 40-hr drain time and EURV with a 72-hr drain time for a watershed with imperviousness of 75%. The chances for various residual water conditions is summarized in Table 7.1

Table 7.1 Exceeding Probability of Residual Water Conditions in Metro Denver Area

	WQCV Basin	EURV Basin
100% full	5%	0.01%
75% full	10%	0.2%
50% full	15%	1%
25% full	20%	2%

Obviously, in operation, a EURV basin has much less risk to become full than a WQCV basin because EURV >WQCV at any site. Table 7.1 was produced by the continuous rainfall-runoff flows through a basin for a period of 30 years. A WQB is expected to have a chance of 5% to become full when needed for the next storm in a year or once every 20 years. Table 7.1 provides a risk-based approach to design the freeboard for a basin.

References

Adams County. (2011). "*Chapter 9 - Storm Drainage Design and Stormwater Quality Regulations"*. Adams County, Colorado

ASCE Manual of Practice No. 87 and WEF Manual of Practice No. 23, (1998). "Urban Runoff Quality Management", American Society of Civil Engineers, New York.

City and County of Denver. (2006). "*City and Count of Denver Storm Drainage Design & Technical Criteria"*. Denver, Colorado.

City of Aurora. (2010). "*City of Aurora Storm Drainage Design and Technical Criteria*". Aurora, Colorado.

CWA (1977). Clean Water Act, http://www.epa.gov/owow_keep/NPS/cwact.html

Driscoll, E.D., Palhegyi, G.E., Strecker, E.W. and Shelley, P.E. (1989). Analysis of Storm Events Characteristics for Selected Rainfall Gauges Throughout the United States. U.S. Environmental Protection Agency, Washington, D.C.

EPA Report (1983). Results of the Nationwide Urban Runoff Program, Final Report, U.S. Environmental Protection Agency, NTIS No. PB84-185545, Washington, DC, 1983.

EPA Report (1986). Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality, U.S Environmental Protection Agency, EPA440/5-87-001, September.

EPA SWMM (2011), Stormwater Management Model supported by EPA. http://www.epa.gov/nrmrl/wswrd/wq/models/swmm/

Guo, James C.Y. and Mackenzie, K (2014). "Modeling Consistency for small to large watershed studies", ASCE J. of Hydrologic Engineering, V 19, N8, Aug , 04014009-1 to -7.

Guo, James C. Y. Urbonas, B. and MacKenzie K. (2014) "Water Quality Capture Volume for LID and BMP Designs", ASCE J of Hydrologic Engineering, Vol 19, No 4, April, 682-686

Guo, James C.Y. and Urbonas B. (2014) " Volume-based Runoff Coefficient", ASCE Journal of Irrigation and Drainage Engineering, V 140, N 2, Feb, 04013013-1 to -5.

Guo, James C.Y. (2013). "Green Concept in Stormwater Management", Journal of Irrigation and Drainage Systems Engineering, Vol 2, Issue 3, ISBM 2168-9768.

Guo, James C.Y. (2012). "*Off-stream Detention Design for Stormwater Management*", ASCE Journal of Irrigation and Drainage Engineering, Vol. 138, No. 4, April 1.

Guo, James C. Y. Shih, H. M. and MacKenzie K. (2012) "Stormwater Quality Control LID Basin with Micropool", ASCE Journal of Irrigation and Drainage Engineering, Vol. 138, No. 5, May 1.

Guo, James, C.Y., Urbonas, B., and K. MacKenzie (2011). "*The Case for a Water Quality Capture Volume for Stormwater BMP*", J. of Stormwater, Oct.

Guo, James C.Y. (2010)"Preservation of Watershed Regime for Low Impact Development using (LID) Detention", ASCE J. of Engineering Hydrology, Vol 15, No 1., January.

Guo, James C.Y.(2009) "*Retrofitting Detention Basin for LID Design with a Water Quality Control Pool*", ASCE J. of Irrigation and Drainage Engineering, Vol 135, No 6, October.

Guo, James C.Y. and Urbonas, B. (2009) "*Conversion of Natural Watershed to Kinematic Wave Cascading Plane*", ASCE J. of Hydrologic Engineering, Vol 14, No. 8, August.

Guo, James C.Y. and Cheng, J. Y.C.(2008) "*Retrofit Stormwater Retention Volume for Low Impact Development (LID)*", ASCE J. of Irrigation and Drainage Engineering, Vol 134, No 6, December.

Guo, James C.Y. (2004). "*Hydrology-Based Approach to Storm Water Detention Design Using New Routing Schemes*," ASCE J. of Hydrologic Engineering, Vol 9, No. 4, July/August

Guo, James C.Y. (2002). "*Overflow Risk of Storm Water BMP Basin Design,*" ASCE J. of Hydrologic Engineering, Vol 7, No. 6, Nov.

Guo, James C.Y. and Urbonas, Ben. (2002). *"Runoff Capture and Delivery Curves for Storm Water Quality Control Designs,*" ASCE J. of Water Resources Planning and Management, Vol 128, Vo. 3, May/June.

Guo, James C.Y. (1999). "*Detention Basin Sizing for Small Urban Catchments,*" ASCE J. of Water Resources Planning and Management, Vol 125, No.6, Nov.

Guo, James C.Y. and Urbonas, Ben (1996). "*Maximized Detention Volume Determined by Runoff Capture Rate*," ASCE J. of Water Resources Planning and Management, Vol 122, No 1, Jan.

Harris Kocher Smith (2011). "*Final Drainage Report US Highway 285 Reconstruction CDOT Project No. BR 2854-113 Area 4 – Knox/Lowell Intersection*", published by Urban Drainage and Flood Control District, Denver, Colorado. http://www.udfcd.org

MacKenzie, K. A. (2010). "*Full-Spectrum Detention for Stormwater Quality Improvement and Mitigation of the Hydrologic Impact of Development: A Regionally Calibrated Empirical Design Approach*", Master Degree Study Report, Civil Engineering, U of Colorado Denver, Colorado

MacKenzie, K.A. (2015), "Revision on UDFCD EURV Determination Memorandum", http://udfcd.org/wp-content/uploads/uploads/resources/technical%20memos/UDFCD_EURV_Determination_Memorandum.pdf
Medi, Saman (2012). "*Design Verification of Micropool at EDB502L*", Master-Degree Study, Civil Engineering, U of Colorado Denver, Colorado.

Simon, D. and Senturk, F. (1990). "Sediment Transport Technology", Water Resources Publication, WWW.Amazon.Com.

UDFCD (1995). "*Improvements to Knox Court and Irving Outfall*", As-Built, published by Urban Drainage and Flood Control District, Denver, Colorado. http://www.udfcd.org

UDFCD (1996). "*Floyd Ave and Knox Court Storm Sewer*", As-Built, published by Urban Drainage and Flood Control District, Denver, Colorado. http://www.udfcd.org

UDFCD (2001). "*Urban Stormwater Design Drainage Manuals*", Vol 1 and 2, published by Urban Drainage and Flood Control District, Denver, Colorado. http://www.udfcd.org

UDFCD (2011). "*Urban Stormwater Design Drainage Manuals*", Vol 3, published by Urban Drainage and Flood Control District, Denver, Colorado. http://www.udfcd.org

Urbonas, B., Guo, J. C., & MacKenzie, K. (2001). "*Description of the WQ-COSM Computer Model to Generate a Water Quality Capture Volume for Stormwater BMPs"*. Denver.