OFF-STREAM DETENTION DESIGN FOR STORM WATER MANAGEMENT

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ABSTRACT
A detention basin is designed for storm water quantity and quality controls. An in-stream basin is often located on a widened floodplain, while an off-stream basin can be part of open space in a parking lot, park, or sport field. The inflow channel associated with an off-stream basin is allowed to carry its base flow not to exceed the downstream capacity. When the inflow channel is full, the excess flood water is diverted into the off-stream basin through the side weir. During the flood recession, the equalizer conduit drains the basin flow back to the channel. In this study, the rational volumetric method was expanded from in-stream to off-stream detention design. The required off-stream detention volume is maximized by the rainfall event longer than the time of concentration of the tributary area. In comparison, the off-stream detention volume can be significantly less than an in-stream, depending on the flow-through capacity in the downstream channel. This paper presents a design procedure to maximize the off-stream detention volume, to size the flow-diversion weir, and to set the flood-gate operation on the equalizer conduit. The simplified derivation on turning moment indicates that the operation of flood gate is dictated by the water depth differential and the weight of the gate.

Key Words: Off-stream Detention, Basin, Flood Gate, Culvert, Flow Diversion.

INTRODUCTION
Storm water management is one of the major tasks for preserving urban water environment (ASCE, 1994). A storm water drainage system consists of conveyance and storage facilities. Detention and retention basins are the major storage facilities designed for storm water quantity and quality controls (Guo 2007). Storage facilities in a drainage network should be placed at strategic locations in order to effectively attenuate peak flood flows (Jone et al. 2006). During the preliminary studies, it is necessary to evaluate all feasible combinations of basin locations, storage volumes, and allowable release rates (McCuen in 1998). Decision making relies on the impact assessments by numerical simulations for the entire watershed with and without the proposed detention basin. A detention basin may operate as an on-stream facility if the inflow channel directly drains into the basin or an off-stream facility if storm water is diverted into the basin from the inflow channel. Flow diversion is triggered when the flood flow exceeds the pre-determined channel capacity. In this study, the rational volumetric method using the Federal Aviation Administration’s approach (FAA 1970) was expanded from on-stream to off-stream storm water detention designs. An off-stream basin shall be equipped with a side weir installed on the low bank along the channel and an equalizer conduit installed on the basin floor. During the loading cycle, the side weir diverts the channel flow to fill up the basin, and then during the recession cycle, the equalized conduit drains the basin flow into the channel.

In this study, the design procedure was derived to maximize the off-stream detention volume, to size the side weir, and to set the flood-gate operation on the equalizer conduit. In comparison, an off-stream detention volume is smaller than the on-stream because the inflow channel is allowed to pass...
its base flow up to the downstream capacity. But, the straight-through base flow often flushes the debris and trash without quality control.

**OFF-STREAM DETENTION VOLUME**

Stormwater detention is an effective approach to reduce the after-development peak flows (ASCE, 1994). An on-stream detention is to drain a channel into a widened floodplain area, while off-stream detention is to divert stormwater from the channel into an adjacent basin. As illustrated in Figure 1, an off-stream detention allows the channel to carry its base flow, \( Q_1 \), and only diverts the peak volume into the basin. In practice, a diversion weir is installed along the channel bank to control the flow diversion, and a flood-gate conduit is laid on the basin floor to serve as an outlet when the flood water in the channel recedes. The basin may also have a separate outlet to release water, \( Q_2 \), out of the channel system. The sum of \( Q_1 \) and \( Q_2 \) must not exceed the total allowable release, \( Q_a \), from the watershed under the developed condition. The design criteria of allowable release flows can be found elsewhere (UDFCD 2001).

![Figure 1 Layout of Off-stream Detention Basin](image)

Small detention basins are usually installed as an on-site device to regulate the stormwater releases from urban catchments smaller than 100 acres. As a result, the rational method is suitable to predict the peak flow and volume. At the project site, the design rainfall intensity-duration-frequency formula (IDF) is described as:

\[
I = \frac{C_1}{(C_2 + T_d) C_3}
\]

in which \( I \) = rainfall intensity in \([L/T]\) such as mm/hr, \( T_d \) = rainfall duration in \([T]\) such as hour, and \( C_1 \), \( C_2 \), and \( C_3 \) are local IDF curve. Using the Rational method, the runoff volume can be represented by a trapezoidal hydrograph (Malcom 1982, McCuen 1998, Guo 1999). As illustrated in Figure 2, the inflow hydrograph has a linear rising limb over the time of concentration of the tributary watershed, and the
hydrograph plateau extends from the time of concentration, $T_c$, to the end of the rainfall event at $T_d$. In practice, the straight-thru capacity, $Q_1$, is the maximum allowable in the downstream channel. Flow diversion begins at the pre-set flow rate, $Q_1$. The outflow volume, the area of $abefg$ in Figure 2, can be calculated as two trapezoids, $bef$ and $abfg$, as:

$$S_o = \frac{Q_o}{2} (T_d + T_c - 2T_1) + \frac{Q_1}{2} [(T_d + T_c - 2T_1) + (T_d + T_c)]$$  \hspace{1cm} (2)$$

in which $S_o =$ outflow volume, $T_c =$ time of concentration of the tributary watershed, $T_d =$ rainfall duration, and $T_1 =$ time to begin flow diversion.

![Figure 2 Illustration of Detention Volume in Relation to Hydrographs](image)

On the linear rising limb, the peak inflow occurs at the time of concentration, $T_c$, and the diversion flow is triggered at:

$$T_1 = \frac{Q_1}{Q_p} T_c$$ \hspace{1cm} (3)$$

in which $Q_1 =$ downstream channel capacity equal to the allowable flow-through capacity, and $Q_p =$ peak inflow at $T_c$. The peak inflow is calculated as:

$$Q_p = CIA$$ \hspace{1cm} (4)$$

in which $C =$ runoff coefficient, and $A =$ watershed tributary area in $[L^2]$. For mathematical convenience, the outflow volume, $S_o$ in $[L^3]$, is expressed by the average release over the rainfall duration, $T_d$, as:

$$S_o = mQ_o T_d$$ \hspace{1cm} (5)$$

In which $m =$ volume adjustment factor. Equating Eq (2) to Eq (5) yields the value of $m$ to be:
\[
m = \frac{1}{2} \left[ 1 + \frac{(T_c - 2T_d)}{T_d} \right] + \frac{Q_1}{Q_2} \left[ 1 + \frac{(T_c - T_d)}{T_d} \right] \quad \text{for } T_d > T_c
\]  

(6)

For an on-line detention basin, \( Q_2 = 0 \) and \( T_1 = 0 \). As a result, Eq (6) is reduced to

\[
m = \frac{1}{2} \left( 1 + \frac{T_c}{T_d} \right) \quad \text{for } T_d > T_c
\]  

(7)

Eq (7) agrees with previous studies (Aron and Kibler in 1990 and Guo in 1999). According to the Rational method, the inflow volume in the channel is equal to the rainfall excess volume as:

\[
S_i = CIAT_d = Q_p T_d
\]  

(8)

Aided by Eq's (5) and (8), the detention volume for the selected rainfall duration, \( T_d \), is:

\[
S_d = S_i - S_o = (Q_p - mQ_d)T_d
\]  

(9)

In which \( S_i \) = inflow volume in \([\text{L}^3]\), and \( S_o \) = detention volume in \([\text{L}^3]\). As reported, a conveyance facility is designed to pass the peak flow estimated using \( T_d = T_c \). But, the design rainfall event for a storage facility should have a longer duration than \( T_c \) (Urbonas and Stahre in 1992). In this study, it is suggested that Eq (9) be tested for a range of \( T_d \), until the maximum detention volume is identified. Starting from \( T_d = T_c \), the detention volume is computed with an increment of 5 or 10 minutes in storm duration. Generally, the detention volume increases as the rainfall duration increases. After the maximal volume, the computed detention volume decreases as the rainfall duration increases (FAA 1970, DAAF 1977). Consequently, the design storage volume, \( S_s \), for the detention basin under design is determined as:

\[
S_s = \max(S_i - S_o)
\]  

(10)

This maximization procedure can be repeated for both 10- and 100-yr events if the off-stream basin is to be shaped for multiple events.

**DIVERSION FOR INFLOW**

Off-stream detention is an open channel flow system. The flow diversion system consists of a downstream culvert and a side weir. As illustrated in Figure 3, the side weir is built across the channel bank. The performance of a side weir depends on the headwater depth, \( h \), on top of its crest. As recommended, the downstream culvert is designed to pass the allowable flow-through capacity under inlet control condition. The inflow channel shall be laid on a mild slope. The headwater at the culvert entrance creates an M-1 backwater profile across the side weir. Usually an M-1 water surface profile tends to be long and stable when the culvert entrance becomes full. In this study, the continuity equation for this flow system is written as:

\[
Q_1 = C_o A_o \sqrt{2g(Y - D/2)}
\]  

(11)
\[ Q_w = C_o \sqrt[1.5]{2gB(Y - H)} \]  
\[ Q_p = Q_w + Q_i \]  

In which \( C_o \) = orifice coefficient, \( A_o \) = orifice opening area \([L^2]\), \( Y \) = water depth \([L]\), \( D \) = culvert diameter or height \([L]\), \( Q_w \) = weir flow \([L^3/T]\), \( Q_p \) = peak inflow \([L^3/T]\). These three equations assist the engineer to size the side weir, \( B \) and \( H \), and to determine the headwater, \( Y \), at the entrance of the culvert. Of course, the culvert is selected to be so small that the headwater is close to the flow depth in the incoming channel. This condition warrants that the side weir functions under a uniform headwater.

![Flow Diversion System](image)

**Figure 3 Flow Diversion System**

**FLOOD-GATE EQUALIZER CONDUIT FOR OUTFLOW**

A short conduit is laid on the basin floor to serve as an equalizer. As recommended, a trash rack shall be installed at the conduit entrance on the basin side (Guo et al. 2010a), while a flood gate is installed at the conduit exit on the channel side. For the reason of safety, the surface area of the trash rack shall be at least four times the conduit opening area at the entrance (Guo and Jones 2010b). During the basin’s loading cycle, the diversion flow occurs when the channel is full and the basin is empty. Therefore, the flood gate is automatically closed due to the pressure difference. When both the channel and basin are full, the hydraulic pressure is balanced on the both sides of the flood gate. During the flood flow recession, the water depth in the channel depletes before the basin. As a result, the positive force-moment about the top hinge would automatically open the gate to release the basin flow into the channel (Shipwith et al. 1990).
The hydraulic force acting on the flood gate is determined by the water pressure at the center of the
gate and the gate area, and then placed at the point of application. By gravity, the initial position of the
gate is vertical. The horizontal hydrostatic forces on the gate are calculated as:

\[ F_1 = \gamma(Y_1 - R)A_G \]  \hspace{1cm} (14)
\[ F_2 = \gamma(Y_2 - R)A_G \]  \hspace{1cm} (15)

In which \( F_1 \) = hydraulic force from channel, \( Y_1 \)= water depth in channel, \( A_G \)= gate area, \( R \)= radius for
circular gate, \( F_2 \)= hydraulic force from basin, \( Y_2 \)= water depth in basin, and \( \gamma \)= specific weight of wa-
ter. The points of application for these two horizontal forces are located as:

\[ Y_{p1} = Y_{c1} + \frac{I_c}{Y_{c1}A_G} = (Y_1 - R) + \frac{R^2}{4(Y_1 - R)} \] for circular gate \hspace{1cm} (17)
\[ Y_{p2} = Y_{c2} + \frac{I_c}{Y_{c2}A_G} = (Y_2 - R) + \frac{R^2}{4(Y_2 - R)} \] for circular gate \hspace{1cm} (18)

For convenience, the weight of the gate is expressed in its unit weight per area as:

\[ W = wA_G \]  \hspace{1cm} (16)

in which \( W \)= weight of gate, and \( w \)= unit weight per area of gate. As illustrated in Figure 4, the opera-
tion of the flood gate is dynamic, starting from its vertical position and then becoming open at an in-
clined angle, \( \theta \). To simplify, the turning moment about the hinge in Figure 4 is calculated as:
\[
\gamma (Y_2 - R) (R + \frac{R^2}{4(Y_2 - R)}) - \gamma (Y_1 - R) (R + \frac{R^2}{4(Y_1 - R)}) \geq - w \frac{(S - 1)}{S} \sin \theta \geq 0 \quad (19)
\]

When the basin is full, both \(Y_2\) and \(Y_1\) are numerically much greater than \(R\). As a result, Eq (19) is reduced to:

\[
\Delta Y = Y_2 - Y_1 \geq \frac{w \sin \theta (S - 1)}{\gamma} \quad (20)
\]

and \(\Delta Y\) = water depth difference, \(S\) = specific weight of gate such as 7 to 8 for iron gate, and \(\theta\) = minimal inclined angle to open gate. Eq (20) significantly simplifies a complicated force-moment calculation when determining the operation of a flood gate in numerical simulations. At each computing time step, the operation of the flood gate, open or close, can be easily identified by the water depth difference between the basin and the channel.

DESIGN EXAMPLE FOR OFF-STREAM BASIN

The example in Figure 5 involves a project site in City of Denver, Colorado. Because all design information in this case is prepared in English units, therefore pounds and feet are used in this example. The Denver’s IDF formula is prescribed with \(C_1 = 74.1\), \(C_2 = 10.0\), and \(C_3 = 0.786\) for the 100-year event. The inflow channel collects storm water generated from a tributary area of 62 acres. The runoff coefficient for the tributary area is \(C = 0.68\). The time of concentration of the catchment is \(T_c = 20\) minutes. The total allowable storm water release from the watershed is set to be 62 cfs. The channel illustrated in Figure 5 is designed to pass no more than 15 cfs into the downstream street inlet. As a result, the maximum release from the detention basin for this case is 47 cfs which is the difference between the total allowable release of 62 and the on-stream release of 15 cfs. A side weir, a downstream culvert, and equalizer conduit are installed to fill and to drain the detention basin. The task is to determine the off-stream detention storage volume, and dimensions of these hydraulic drains.

As indicated in Eq 10, the detention volume can be maximized by a test over a range of storm duration. For example, try \(T_d = 50\) minutes. The calculations are summarized as follows:

1. **Inflow volume**

\[
I = \frac{74.1}{(10 + 50)^{0.786}} = 2.97 \quad \text{inch/hr}
\]

\[
Q_p = CIA = 0.68 \times 2.97 \times 62 = 125.2 \quad \text{cfs}
\]

\[
S_i = CIAT_d = 0.68 \times 2.97 \times 62 \times 50 = 8.68 \quad \text{acre-ft}
\]
(2) Outflow volume

\[ T_1 = \frac{Q_1}{Q_p} T_e = \frac{15}{125.2} \times 20 = 2.40 \text{ minutes} \]

\[ m = \frac{1}{2} \left[ 1 + \frac{20 - 2 \times 2.4}{50} \right] + \frac{15}{47} \left( 1 + \frac{20 - 2.4}{50} \right) = 1.1 \]

\[ S_{o} = mQ_{o}T_{d} = 1.1 \times 47 \times 50 \times 60 / 43560 = 3.51 \text{ acre-ft} \]

(3) Storm water detention volume, \( S_d \), for the 50-minute rain storm is:

\[ S_d = 8.68 - 3.15 = 5.18 \text{ acre-ft} \]

Repeating this process for the range of rainfall duration from 40 to 80 minutes, Table 1 summarizes the variation of detention storage volumes. The maximum storage volume is identified to be 5.22 acre-ft with storm duration of 60.0 minutes.
<table>
<thead>
<tr>
<th>Duration (minutes)</th>
<th>Rainfall Intensity (inch/hr)</th>
<th>Inflow Volume (acre-ft)</th>
<th>Peak Runoff (cfs)</th>
<th>Diversion Time T₁ (minutes)</th>
<th>Coef m</th>
<th>Outflow Volume (acre-ft)</th>
<th>Storage Volume (acre-ft)</th>
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<td>8.02</td>
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<td>2.40</td>
<td>1.08</td>
<td>3.51</td>
<td>5.18</td>
</tr>
<tr>
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<td><strong>2.63</strong></td>
<td><strong>9.23</strong></td>
<td><strong>110.78</strong></td>
<td><strong>2.71</strong></td>
<td><strong>1.03</strong></td>
<td><strong>4.01</strong></td>
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<td>90.93</td>
<td>3.30</td>
<td>0.97</td>
<td>5.02</td>
<td>5.08</td>
</tr>
</tbody>
</table>

Note: 1 inch = 25.4 mm, 1 ft = 0.305 m, 1 acre = 0.4 hectare

Table 1 Maximal Detention Volume for Example Design

For this case, the culvert under the service road needs to pass a peak flow of 15 cfs. With D=15 inches and Cₒ=0.6, Eq (11) suggests a head water depth, Y, of 7 feet in the inflow channel and in the basin as well. Consider a peak water depth of 7 feet in the basin, the average cross area for the basin is approximately 180-ft by 180 ft.

As shown in Table 1, the peak flow is 110.78 cfs for the design event. As a result, the side weir has to divert the difference of 95.78 cfs into the basin. With C_w=3.0, W=32 ft, and Y=7 ft, Eq (12) suggests that the weir crest be set at H=6 ft above the channel invert or the weir flow is subject to one-ft headwater.

The weight of flood gate depends on its material. An 18-inch circular iron gate weighs 200 pounds with its specific gravity of 7.5 (Reference??). This gate is considered open when the inclined angle is 45° or greater. The unit weight per gate area is:

\[ w = \frac{W}{A} = \frac{200}{\pi(1.5)^2} = 113 \text{ pound per sq ft of iron gate} \]

For this case, the minimum depth differential for the gate to be open is approximately 1.1 feet of water as computed as:

\[ \Delta Y = (Y_2 - Y_1) \geq \frac{113 \sin 45°}{62.4} \frac{(7.5 - 1)}{7.5} = 1.1 \text{ ft} \]

For this case, the critical design rainfall duration for the off-stream detention design is 60 minutes, while the time of concentration of the watershed is 20 minutes. Since storm water detention is designed for peak flow reduction, the detention system sized under the 60-minute rainfall event must be incorporated into a computer model (HEC HMS 2010) to study the flood mitigation under the major event, i.e 100-yr event. This is a common practice in drainage design.

CONCLUSION

Over the years, the rational volumetric method for stormwater detention design has been used under the assumption that the peak outflow could be estimated by the gravity flow through the outfall pipe, i.e. Manning’s equation. As a result, the full-flow capacity has been used to size the required detention volume, i.e. m=1.0 for all cases. Recognizing that the value of m must reflect the on-stream or off-stream operation, the current approach significantly violates the principle of continuity in flow volume.
This paper modifies the FAA method for both on-stream and off-stream basin designs to satisfy the flow continuity. The adjustment factor, m, varies between 1.0 and 0.5 when sizing an on-stream basin, while the value of m can exceed unity for an off-stream basin, depending on the flow-through capacity in the channel.

The flow diversion system shall include a downstream culvert to control the flow-through capacity in the channel, a side weir to divert the excess storm water into the basin, and an equalizer conduit to release the stored water volume. The culvert and side weir must be sized to satisfy the flow continuity between the peak inflow and the downstream allowable flow release in the channel. The operation of the flood gate on the equalizer conduit depends on the hydraulic forces and turning moments about the hinge. The simplified method derived in this paper indicates that the water depth differential and the weight of the gate are the key factors on gate operation.

The rational method applies the uniform rainfall distribution to runoff predictions. Although the temporal variations on rainfall hyetograph may result in differently shaped storm hydrographs, the total water volume is conserved. Therefore, the simplified trapezoidal hydrograph is applicable for detention designs. As a common practice, all single elements in a drainage system shall be sized independently, and then linked together as a drainage system for performance evaluation under backwater effects. Similarly, a detention basin shall be sized using the maximization procedure developed in this paper, and then incorporated it into the drainage system for performance evaluation under the major flood events, i.e. 10- and 100-yr events. As expected, minor modifications may be needed until the entire drainage system satisfies the design constraints and criteria.

ACKNOWLEDGMENT

The methods presented in this paper have been included in the Urban Storm Water Design Criteria Manual (2001), recommended by Urban Drainage and Flood Control District (UDFCD) for storm water designs in the Denver metropolitan area. The computer model, UD-Detention, can be downloaded with many other EXCEL Spreadsheets from the web site: www.UDFCD.ORG.

REFERENCES


II. Notations:

\( A_o = \) watershed area in \([L^2]\),
\( A_G = \) flood gate surface area, in \([L^2]\),
\( A_o = \) orifice opening area at culvert entrance in \([L^2]\),
\( B = \) width of side weir in \([L]\),
\( C = \) runoff coefficient
\( C_1 = \) constant in IDF formula
\( C_2 = \) constant in IDF formula,
\( C_3 = \) constant in IDF formula
\( C_o = \) orifice coefficient,
\( D = \) culvert diameter or height in \([L]\)
\( F_1 = \) hydraulic force from channel depth,
\( F_2 = \) hydraulic force from basin depth,
\( I = \) rainfall intensity in \([L/T]\) such as mm/h
\( I_c = \) moment of inertial of gate’s area about its central axis in \([L^4]\)
\( H = \) distance of weir crest above the channel invert in \([L]\)
\( h = \) head water on weir in \([L]\)
\( m = \) adjustment factor
\( Q_1 = \) channel flow-thru capacity in \([L^3/T]\),
\( Q_2 = \) basin release in \([L^3/T]\)
\( Q_a = \) total allowable release from watershed in \([L^3/T]\)
\( Q_p = \) peak inflow at \( T_c \) on hydrograph and in channel in \([L^3/T]\),
\( Q_w = \) weir flow \([L^3/T]\)
\( R = \) radius of circular flood gate in \( [L] \)
S = specific gravity, 7.0 to 8.0, for iron or steel gate.
S_i = inflow volume in [L^3]
S_o = outflow volume in [L^3]
S_s = design storage volume in [L^3]
T_c = time of concentration of the tributary watershed in [L^2]
T_d = rainfall duration in [T] such as hour,
T_1 = time to begin flow diversion in [T]
Y = head water in front of downstream culvert in [L],
Y_1 = water depth in channel during flood flow recession in [L]
Y_2 = water depth in basin during the flood flow recession in [L]
Y_{P1} = distance to point of application for F_1 in [L]
Y_{P2} = distance to point of application F_2 in [L]
Y_{C1} = water depth in channel to the center of flood gate in [L]
Y_{C2} = water depth in basin to the center of flood gate in [L],
ΔY = difference in water depth in [L]
w = unit weight per flood gate area [force/L^2]
W = weight of flood gate [force]
y = water specific weight [force/L^2]
θ = incline angle of flood gate