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# INUDATION CAPACITY AT STREET INTERSECTION

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*ABSTRACT* The major function of a street is to maintain the movement of traffic. Under the concept that street drainage shall be designed to collect storm water as fast as possible, the street storm water capacity has been defined as its hydraulic conveyance as estimated by Manning's formula. This practice has resulted in a prevailing experience that street intersections are often flooded. This study presents an investigation on street hydraulic capacity. It is found that the street storm water capacity at a sump is in fact dictated by the storage capacity rather than the conveyance capacity. A new design methodology is developed in this study to consider the street depression storage as a criterion when sizing a sump inlet. Design parameters required by this method include the local IDF information, catchment area, runoff coefficient, street transverse slope, and the configuration of the sump area as a fraction of a circle.

*Key Words: street, sump, depression, storage, water spread, storm water, runoff*

## INTRODUCTION

A street drainage system includes storm water collection, conveyance, and storage facilities. Although street inlets and storm sewers are often designed as part of the minor drainage system to pass the 2- to 5-year peak runoff discharges, streets are in fact part of the major drainage system which shall be capable of passing the 100-year storm event under the specified design constraints. For instance, the water spread on a local street is limited to one-half traffic lane under the 2-year storm (design event), and one traffic lane in each direction free from water spread during the 100-year storm (checking event) (Anderson 1993). Although the street drainage concept is to remove storm water from streets as fast as possible, we in fact experience a common problem that street intersections are flooded. Inundation at a street intersection causes inconvenience and imposes potential hazards to traffic. If traffic safety has been an increasing concern, why do we still experience such a prevailing symptom that street intersections are often flooded?

In an urban area, the geometry of a street cross section plays a key role in storm water drainage designs. Figure 1 illustrates a typical street cross section. Transverse slopes move the water off the pavement quickly while curbs and gutters transport the water efficiently to inlets from which the water drains into storm sewers or natural waterways. In order to have a positive drainage by the gravity, a minimum longitudinal grade of a street is recommended to be 0.50% (AASHTO, 1990). However, street vertical curves connect segments of constant grades; and since these curves often represent a change between a positive and negative grade, a low point along a street exists.

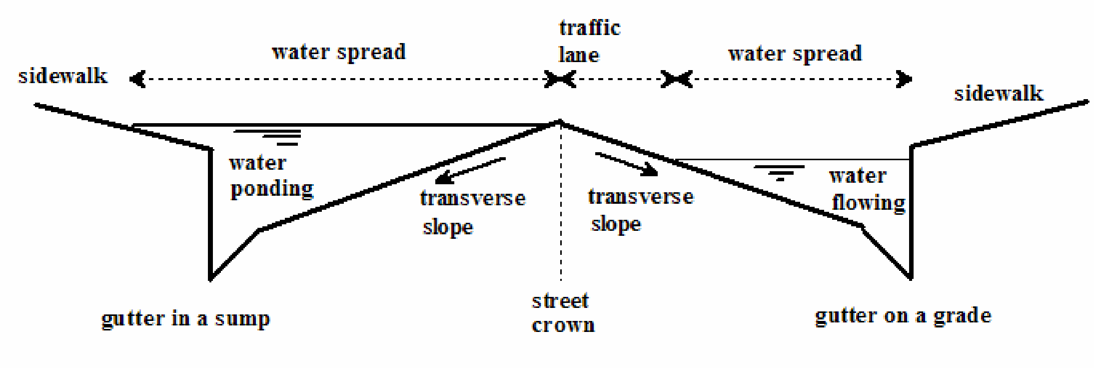

**Figure 1  Street Cross Section**

On a positive grade, an inlet shall be installed wherever the spread of storm water exceeds allowable limits. At a low point, an inlet must be provided to collect the storm water by efficient transverse slopes. The transverse slope of a street is a comprise among drainage, driver comfort, and vehicle stability. Anderson (1993) outlined various roadway cross sections and the necessary parameters involved in the determination of a transverse slope. The maximum transverse slope of 2.0 % is recommended for two-lane pavements. In the case that a higher transverse slope is required to reduce water on the pavement of a three or more lane pavements, it is suggested that the transverse slope be increased in each successive lane up to a maximum permissible slope of 4.0 percent (Gallaway 1979). Although the street transverse slope forms a triangular channel along the street curb and greatly increases street hydraulic capacity and inlet efficiency, it does create a sump at a street intersection as shown in Figure 2. At an intersection of major collector streets, any storm water crossing a street must be avoided. As a result, storm water entrained at the sump area must be drained by sump inlets.

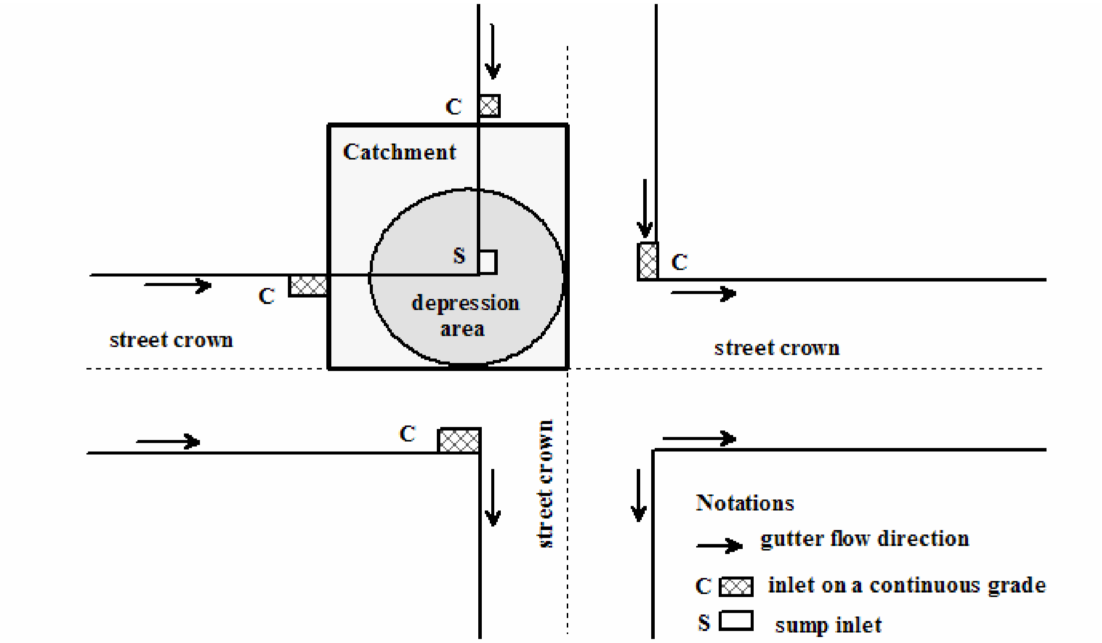

**Figure 2  Sump Locations at Street Intersection**

Water ponding on the street may be caused by drainage operational malfunctions such as the sewer system being backed up or the sump inlet being clogged. However, from the design point of view, the main reason to have a street intersection flooded is an underestimation of the storm water volume that overwhelms the local depressed area. In the current practice, the design of street drainage begins with inlets and sewers sized to pass a 2- or 5- year peak discharges. Having the minor drainage system completed, it is necessary to make sure that the major storm runoff under a 100-year event can be jointly conveyed by the sewer system and the street within allowable water spreads. On a sloping street, the water spread is estimated by the revised Manning's equation with the specified street cross sectional geometry (Guo, 2000). However, such a conveyance approach fails to evaluate the water spread at a depressed street section because the performance of a sump inlet is related to its storage capacity rather than its hydraulic conveyance. Therefore, the conveyance-based method becomes indeed inadequate in estimation of water spread around a sump inlet. Often, depressed street sections are assumed to have the same performance as the sloping. This practice can be misleading and result in a wider water spread in a sump area than that on a continuous grade.

In this study, it is suggested that the performance of a sump inlet be evaluated by a volume based method (Urbonas and Guo, 1996). The required storm water detention volume at a sump area can be maximized by choosing different storm duration (Guo 1999). The corresponding water depth and spread can then be predicted by using the conic shape to model the depression storage volume at a street intersection. This approach provides an assessment of the water spread and drain time at a sump inlet. Having a minor drainage system sized for a street, a complete assessment of the street storm water drainage capability shall be evaluated by applying the conveyance method to sloping sections and the volume method to depressed sections.

## STORM WATER VOLUME AT A STREET SUMP

In an urban area, the distance between street inlets is approximately 200 to 400 feet. The catchment area for a sump inlet is approximately one to three acres. The volume-based approach is applicable to such a small urban catchments (Guo, 1999). To predict the peak runoff from such a small urban watershed, the Rational method states:

$$Q_d = \alpha C I_d A \tag{1}$$

The rainfall intensity in Eq 1 can be described as:

$$I_d = \frac{a}{(T_d+b)^n} \tag{2}$$

in which $\alpha$ = unit conversion factor, equal to 1 for English units, and 1/360 for SI units, C = runoff coefficient, A= watershed area in acres (hectare), $I_d$ = rainfall intensity in inch/hr (mm/hr), $T_d$ = rainfall duration in minutes, $Q_d$= peak runoff rate in cfs (cms) and a, b, and n= constants on the Intensity- Duration- Frequency (IDF) formula. Street inlets and sewers are generally sized for the 2- or 5-year event by Eq's 1 and 2. Under a major event, all street inlets are inadequate to pass the major peak flow rates. As a result, the excess storm water will flow toward the sump areas where the accumulated storm water will be released to the sewers by the sump inlet. The basic concept in the volume-based method is to find the maximum volume difference between the inflow and outflow volumes under a series of storm events with different duration (Department of the Army and the Air Force in 1977). At a sump inlet, the inflow runoff volume is determined by the net rainfall volume as:

$$V_i = \alpha C A I_d T_d \tag{3}$$

The outflow volume can be estimated by the sump inlet capacity as

$$V_o = Q T_d \tag{4}$$

in which Q = sump inlet capacity designed to pass a 2- or 5-year peak discharge. The required storage volume is the difference between Eq 3 and Eq 4. Aided by Eq's 1, 2, 3 and 4, the storage volume, V , is obtained as

$$V = [\alpha CA \frac{a}{(T_d+b)^n} T_d - QT_d] \ast 6 \qquad (5)$$

To maximize the storage volume, the derivative of Eq 5 with respect to rainfall duration is set to be zero, and yields:

$$\frac{dV}{dT_d} = \frac{-nT_d}{(T_d+b)^{n+1}} + \frac{1}{(T_d+b)^n} - \frac{Q}{\alpha aCA} = 0 \qquad (6)$$

Solution of Eq 6 is:

$$T_d = \frac{1}{n}[(T_d+b) - \frac{Q}{\alpha aCA}(T_d+b)^{n+1}] \qquad (7)$$

in which $T_d$ = design rainfall duration. When the value of b in Eq 7 is numerically negligible, the approximate solution of Eq 7 can be:

$$T_d = [\frac{\alpha aCA(1-n)}{Q}]^{\frac{1}{n}} \qquad (8)$$

In fact, Eq 8 can also provide the first approximate solution to Eq 7 during the trial and error procedure. Substituting $T_d$ obtained from Eq 8 into Eq 5, the required storm water storage volume can be determined.

## STREET STORAGE CAPACITY

At a street intersection, crossing flows are prevented by raising the street crown. Storm water entrained at a depressed area must be drained by a sump inlet. Considering that a depression storage volume is similar to a conic volume with the street transverse slope as the conic side slope, the conic volume is calculated as:

$$V = \frac{1}{3} hA_h \qquad (0 \le h \le H_c) \qquad (9)$$

or

$$V = \frac{1}{3} H_c A_c + \frac{(h-H_c)}{3}(A_c + A_h + \sqrt{A_c A_h}\,) \qquad (H_c \le h \qquad (10)$$

in which h = water depth, $H_c$ = curb height, $A_c$ = water surface area at $H_c$, $A_h$ = water surface area at depth h, and V = storage volume. Figure 3 indicates that the storage capacity around a sump inlet consists of volumes below and above the curb height. As illustrated in Figure 4, the depressed area can be approximated by a circle for water depths above the curb height, or by a fraction of a circle for water depths below the curb height, depending on the configuration of the street section.

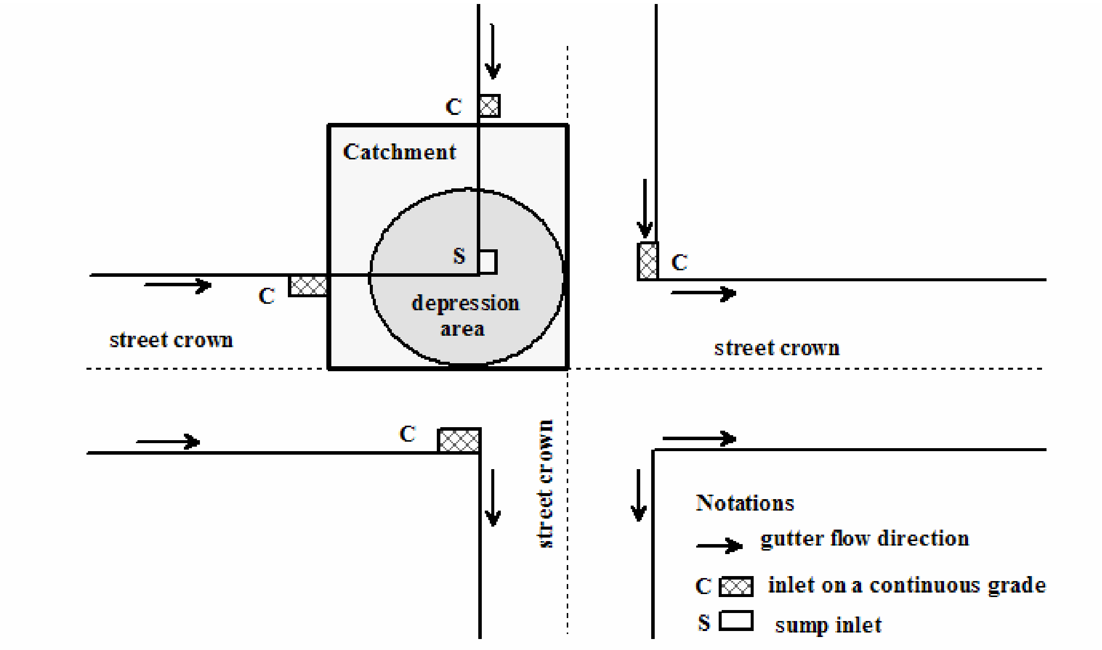

Figure 4 Ponding Area as Faction of Circle

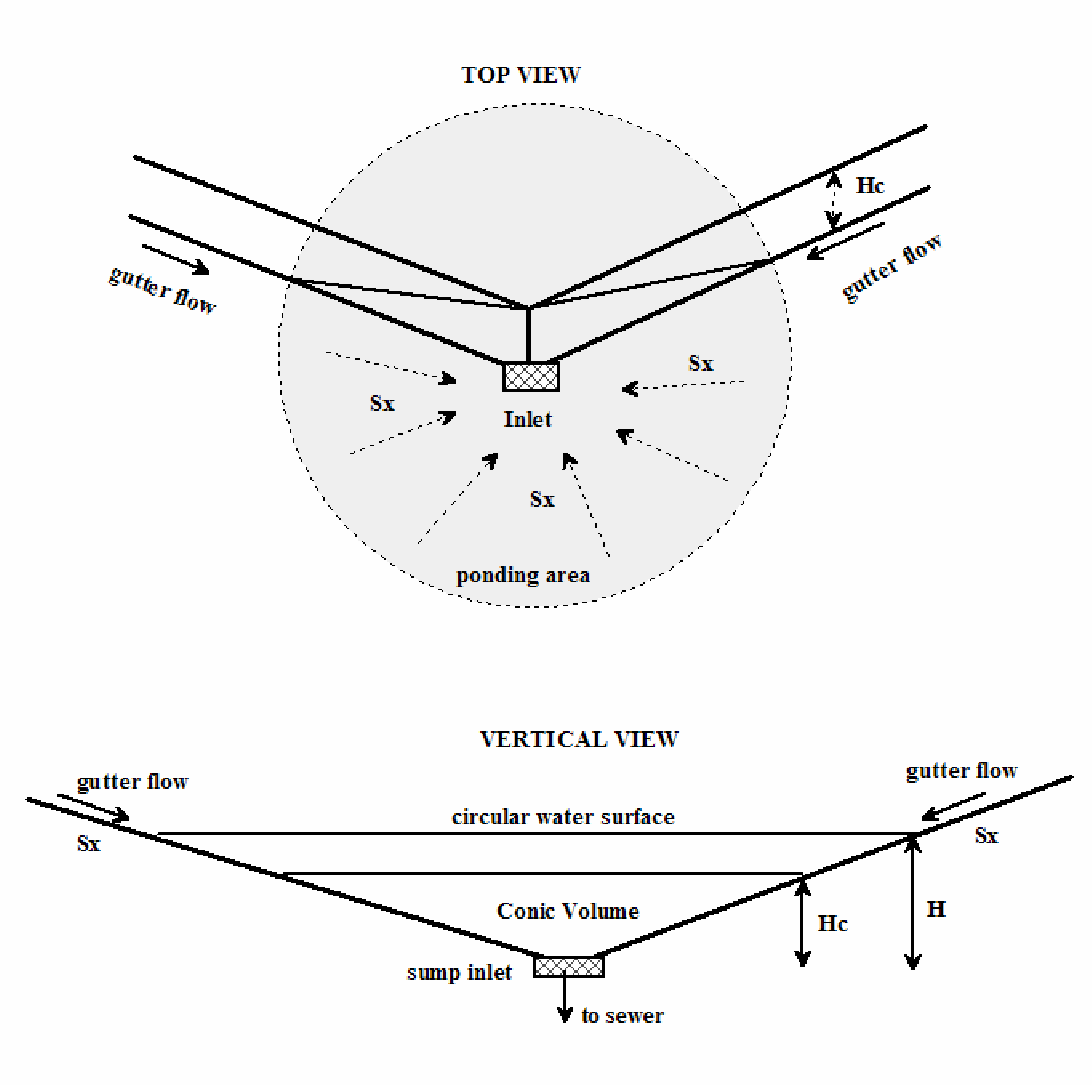

**Figure 3 Storage Volume at Sump**

For instance, k = 0.5 for the low point at a straight depressed street section, k= 0.75 for the intersection area of two sloping streets, k= 0.25 for the low point at a curved depressed street sections, and k =1 when the water depth can freely expand out above the curb height. Knowing the value of k, the water surface area at a depth h is:

$$A_h = k\pi R_h^2 \qquad (11)$$

in which k = a faction of a circle, and $R_h$ = radius of the circular water area at a sump. The radius of the water surface can be approximated by the water depth, h, and the street transverse slope, $S_x$, as:

$$R_h = \frac{h}{S_x} \qquad (12)$$

Substituting Eq's 11 and 12 into Eq 9 yields

$$V = \frac{k}{3}\frac{\pi h^3}{S_x^2} \qquad \text{for } 0<h<H_c \qquad (13)$$

Substituting Eq's 11 and 12 into Eq 10 yields

$$V = \frac{k}{3}\frac{\pi H_c^3}{S_x^2} + \frac{\pi(h-H_c)}{3S_x^2}\left(kH_c^2 + h^2 + \sqrt{k}\,H_c h\right) \qquad \text{for } h>H_c \qquad (14)$$

In general, the transverse slope of 0.02 to 0.04 is used for street designs. Figure 5 is an example to plot Eq 14 as a function of depth with $S_x$ = 0.02 and $H_c$ = 0.5 foot. In design, the runoff storage volume is first determined by Eq's 5 and 7. The maximum water depth, H, can be predicted by Eq's 13 and 14. The water spread, $R_h$, due to the temporary storage process at a depressed street corner can be predicted by Eq 12.

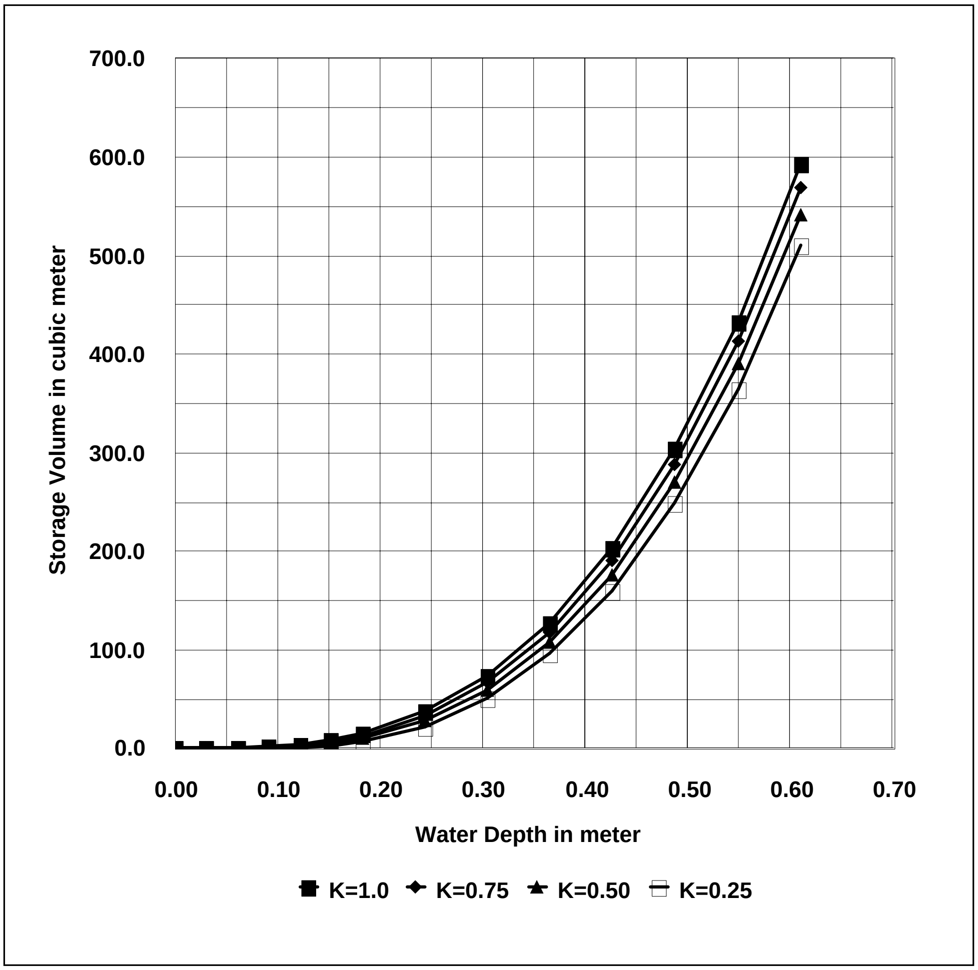

**Figure 5 Street Storage Capacity Versus Water Depth with Various K Values for $H_c$ = 0.15 meter and $S_x$ = 0.02**

## INUNDATION TIME

The design rainfall duration, $T_d$, in Eq 7 is not a clock time, but the most intense period during the storm event. It is also the filling time to load up the sump area. After the inflow becomes less than the capacity of the inlet, the ponding water begins to recede. Therefore, the inundation time at an inlet is defined as the sum of loading time, i.e. rainfall duration, and drain time. At a sump, the inlet can be a curb opening, inlet grate, or combination of both. An inlet of any type can operate like either a weir or an orifice, depending on the depth of water. In this study, an inlet grate is used as an example to establish the methodology for estimating the inundation time. This approach can be repeated to other types of inlet. The hydraulics of a grate in a sump can be modeled by an orifice flow when the water depth is deep or a weir flow when the water depth is shallow. The orifice formula is:

$$Q = C_o e A_o \sqrt{2gh} \qquad (15)$$

in which Q = inlet capacity, $C_o$ = orifice coefficient such as 0.65, e = ratio of opening area to grate area, $A_o$ = grate area, and g = gravitational acceleration. The weir flow formula is:

$$Q = C_w P h^{1.5} \qquad (16)$$

in which $C_w$ = weir coefficient such as 3.00, and P = wetted perimeter of the grate. The wetted perimeter of a grate depends on the gutter depression and curb height (HEC12 by FHA 1979). The minimum water depth, Y, to trigger a switch of inlet hydraulics from weir flow to orifice flow has been suggested as (HEC12 by FHA 1979):

$$Y=1.4\ H_c \qquad (17)$$

Eq 17 is empirical and does not take the inlet grate geometry into consideration. In practice, the minimum depth can be determined by the intersection of the weir and orifice rating curves. In other words, equating Eq 15 to Eq 16 yields (Guo 1999):

$$Y = \frac{C_o e A_o \sqrt{2g}}{C_w P} = \frac{C_o e W L \sqrt{2g}}{C_w (1.8W + L)} \qquad (18)$$

in which Y = minimum water depth for a grate to be an orifice, W = width of grate, and L= length of grate in the flow direction. For instance, a Type 16 grate inlet has W=1.80 feet, L = 3.00 feet, and e = 0.6, this set of parameters yields Y= 0.90 foot. When the ponding depth exceeds the depth, Y, a Type 16 grate will operate like an orifice.

In this study, Eq 13 with K=1.0 is used as an example to derive the inundation time. In the case that the inlet operates like a weir flow for the range of ponding depth, the inundation time is estimated as:

$$\int_{t=T_m}^{t=T} dt = -\int_{h=H}^{h=0} \frac{A_h}{60 * Q}\, dh \qquad \text{for H<Y} \qquad (19)$$

in which t = time variable, and T = inundation time in minutes. Substituting Eq 13 and Eq 16 into Eq 19 yields

$$T = T_d + \tfrac{2}{3} M H^{1.5} \qquad \text{for H<Y} \qquad (20)$$

in which M = weir constant defined as

$$M = \frac{1}{60}\frac{k}{S_x^2}\frac{\pi}{C_w P} \qquad (21)$$

In the case that the water ponding depth is deeper than Y, a similar integration to Eq 19 can be performed with Eq 15 and yields:

$$T = T_d + \tfrac{2}{3} M Y^{1.5} + \tfrac{2}{5} N (H^{2.5} - Y^{2.5}) \qquad \text{for H>=Y} \qquad (22)$$

in which N = orifice constant defined as

$$N = \frac{1}{60}\frac{k}{S_x^2}\frac{\pi}{C_o e A_o \sqrt{2g}} \tag{23}$$

Knowing the ponding depth by Eq 14, the inundation time can be determined by Eq 20 or 22. As indicated in Eq's 20 and 22, the inundation time at a sump inlet consists of loading time and drain time. In comparison, the loading time, i.e. the design rainfall duration, dominates, and the drain time of a sump inlet is so short that it is not even sensitive to inlet hydraulics, i.e. weir or orifice flow.

## DESIGN EXAMPLE

The example used in this study is a sump inlet located in the City of Denver, Colorado. As a result, the total drainage area for the sump inlet is 2.20 acres with a runoff coefficient of 0.75. The IDF curves in Denver are developed to predict rainfall intensities in inch/hr using a = 27.1, 38.5, 45.9, 62.7, and 74.1 for 2-, 5-, 10-, 50-, and 100-year storm events respectively. The other two coefficients on Denver IDF curves are b = 10.0 and n = 0.789 for all events. The minor system for this example is designed to collect the 2-year peak discharge. The minor design discharge at the sump inlet is 5.0 cfs . With the consideration of debris clogging, a Type 16 inlet with W=1.80 ft, L = 3.0 ft, and n= 0.60 is selected. The street transverse slope is 0.02 and the curb height is 0.5 foot. The capacity of a Type 16 with a water depth at the curb height of 0.5 foot is 7.0 cfs by Eq 16 without considering debris clogging and 5.0 cfs with a clogging factor of 30%, i.e. 70% of 7.0 cfs. The capacity of an inlet is not only dictated by Eq's 15 or 16, but also influenced by the energy grade line in the storm sewer system. When the sewer system is designed to have an inflow of 5.0 cfs at the sump inlet, it is reasonable to assume that during a major event, an inflow of 5.0 cfs from the sump inlet can be sustained by the sewer system.

The conveyance capacities of both street sections upstream of the intersection in this case have been confirmed to be wide enough to pass the 100-year runoff without encroaching into the emergency traffic lane. Under the current practice, the hydraulic capacity at the intersection for this case will be assumed to be adequate. As shown in Table 1, the inundation at the street intersection is analyzed for the 5-, 10-, 50-, and 100- year storm events. For this case, a 5-year event requires a storage volume of 785.6 cubic feet which results in a ponding depth of 0.70 foot and a water spread of 35.0 feet at the street intersection. Similarly, the 100-year water depth and spread are estimated to be 1.16 and 58.0 feet. Although the upstream sloping street sections can adequately pass the 100-year storm water, such a spread at the intersection will encroach into traffic lanes and even result in crossing flows. For this case, the sidewalk has a curb height of 0.5 foot, and will be flooded by any storm event greater than the 2-year event. Table 1 also indicates that the inundation time increases with respect to the return period of the storm event. The 100-year inundation time is almost 2.5 times the 5-year inundation time for this case.

| Design Return Period year | Rainfall Duration $T_d$ minutes | Storage Volume V cubic feet | Maximum Water Depth, h, foot | Water Spread $R_h$ feet | Inundation Time T minutes |
|---|---|---|---|---|---|
| | Eq 8 | Eq 5 | Eq 13 | Eq 12 | Eq 22 |
| 5.00 | 6.12 | 785.60 | 0.70 | 35.00 | 8.27 |
| 10.00 | 8.15 | 1,346.50 | 0.83 | 41.50 | 10.81 |
| 50.00 | 12.57 | 2,975.50 | 1.04 | 52.00 | 16.33 |
| 100.00 | 15.41 | 4,254.90 | 1.16 | 58.00 | 19.98 |

**Table 1 Inundation at Street Section for a Type 16 Inlet with Sx = 0.02, A = 2.20 acres, at Denver, Colorado**

This example is typical in many street drainage designs and reflects the prevailing experience that a street intersection can often be flooded. Although a temporary inundation at a street intersection is in fact inevitable because a sump inlet requires a water depth to function, a prolonged flooding shall be avoided. Therefore, it is suggested that the street drainage system be evaluated by both the conveyance capacity applied to a sloping street section and storage capacity applied to a depressed street section. In the case of severe water encroachments into the emergency traffic lane at a depressed section, mitigation to such a problem may include: (1) to reduce the tributary catchment area by adding inlets upstream, (2) to divert storm water from the depressed area, (3) to upsize both the sump inlet and the downstream storm sewers, (4) to install a concrete swale across the street at the intersection, and (5) to increase the depression storage volume by applying a steep transverse slope.

## CONCLUSIONS

(1) The study presents an investigation on the reason why street intersections are often flooded. A new method was derived to estimated the storm water storage capacity at a depressed street section. It is suggested that street drainage consists of both conveyance capacities on sloping street sections and storage capacities at depressed street sections. When designing a street drainage system, both water spreads due to street storm water conveyance or storage must not exceed the safety limit. Otherwise, a proper mitigation must be considered.

(2) One inlet per every 300 to 400 feet along a street can be a rule of thumb for sloping street sections, but may result in a large amount of water accumulated around a sump inlet at a street intersection. Such a condition can be mitigated by reducing the tributary area to the sump inlet by adding inlets on the upstream sloping street sections.

(3) To increase the capacity of an inlet will reduce the accumulated storm water volume at a sump. However, the performance of an inlet is not just determined by its size but also influenced by the energy grade line in the storm sewer. Therefore, in the case that a higher inlet capacity is desired, it is necessary to upsize both the inlet and the sewer system downstream. When upsizing the drainage facilities is not feasible, storm water shall be either diverted upstream or guided at street intersection to cross the street.

(4) The inundation time is dominated by the rainfall duration to fill up the sump area and increases with respect to the frequency of the storm event. Although the capacity of an inlet is sensitive to its hydraulic performance, the estimated drain time by either weir flow or orifice flow is approximately the same.

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## APPENDIX II.  NOTATIONS

a = constant on the rainfall IDF formula
b = constant on the rainfall IDF formula
A= watershed area in acres (hectare)
$A_h$ = water surface area at a depth h
$A_c$ = water surface area at the curb height
$A_o$ = grate area
C = runoff coefficient
$C_0$ = orifice coefficient
$C_w$ = weir coefficient
e = ratio of grate opening area to grate area
g = gravitational acceleration
h = water depth
$H_c$ = curb height
$I_d$ = rainfall intensity with a duration of $T_c$
k = a area fraction of a circle.
L = length of inlet grate
M = constant for a weir inlet
n = constant on the rainfall IDF formula.
N = constant for an orifice inlet
P = wetted perimeter for inlet grate
Q = drain rate of a sump inlet
$Q_o$ = inlet capacity as an orifice flow
$Q_w$ = inlet capacity as a weir flow
$Q_d$ = peak runoff for the storm with a duration of $T_d$
$R_h$= water spread or the radius of the water surface at depth h
$V_i$ = inflow volume
$V_o$ = outflow volume
V = detention volume for the rainfall event with a duration of $T_d$
$S_x$ = transverse slope
Y = depth to have an orifice flow at a sump inlet
t =  time variable
T = drain time
$T_d$ = rainfall duration
W = width of inlet grate
$\alpha$ = unit conversion factor, equal to 1 for English units, and 1/360 for SI units,