EVALUATION OF RETROFITTING EXISTING STORMWATER DETENTION FACILITIES FOR WATER QUALITY

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Abstract. As part of the implementation of watershed protection strategies associated with the renewal of surface water discharge permits, municipal and county governments are increasingly addressing water quality in receiving waters downstream from development. Goals have been established that may require some portion of existing stormwater management systems to be retrofitted to address water quality. Fulton County, Georgia is in the process of developing a Capital Improvement Plan (CIP) to address these and other concerns. The CIP is an attempt to meet the approximately $265 M in identified needs. A portion of these needs is dedicated towards retrofitting existing stormwater detention facilities. A screening procedure has been developed to broadly evaluate retrofit criteria.

INTRODUCTION

Metropolitan Atlanta has undergone rapid growth in the past several decades. Fulton County, Georgia is the county in which the bulk of the City of Atlanta is located, is an extremely diverse county due in part to its geography; it stretches, in a North-South direction, from one end of the metropolitan region to the other (see Figure 1). Fulton County is divided into three distinct regions, the moderately developing southern portion of the county, the central region consisting mainly of the City of Atlanta and adjacent unincorporated areas south of the Chattahoochee River, and the rapidly growing and developing northern region. Historically, stormwater management has remained the primary responsibility of the private sector to address as development occurred. The result is an inconsistent approach to stormwater management from a systems perspective.

Recently, however, the county has started a process of watershed planning. The impetus for this effort was the watershed protection strategies associated with the renewal of NPDES (National Pollutant Discharge Elimination System) permits. These studies included both an assessment of the watershed from a hydrologic, water quality, and biologic point of view, and a management plan that addressed potential improvements to deal with the problems identified in the assessment phase. These management plans identify in excess of $265 M in needs that may form the nexus of a Stormwater Capital Improvement Program (CIP). Within each of these plans is a Geographic Information System (GIS) inventory of stormwater-related structures and facilities, including detention and retention ponds. Goals have been established for each of the watersheds studied. Some of these goals may require some portion of existing Stormwater management systems to be retrofitted to address water quality. Most of these older systems were originally designed to attenuate higher frequency hydrologic events (i.e., 10- and 25-year storm events), and most do not provide significant water quality treatment.

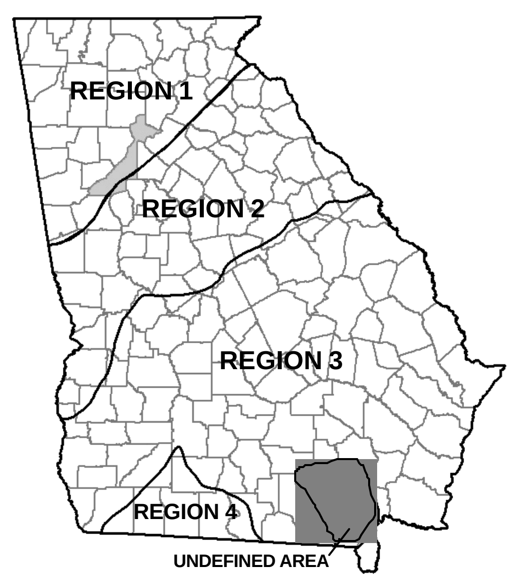

Figure 1. Location of Fulton County, Georgia (USGS, 1993) as presented in Atlanta Regional Commission (2001).

Because of the extensive number of facilities potentially involved, it is infeasible to perform detailed hydrologic routing at all of these facilities. Thus, a screening procedure is needed in order to select possible candidates from this group for further retrofit design analysis.

METHODOLOGY

In Fulton County, as in much of North Georgia, the former policies that governed detention pond sizing was to require storage necessary to retain the difference between the post development peak runoff with a return period of 10 years, or Q_{10i} and the 10-year predevelopment runoff, or $Q_{10,o}$. The result of many years of this practice is a checkerboard of hundreds of detention ponds, approximately 1 per every 50 developed acres, many of which do not perform any significant attenuation of peak flows due to both faults in their design and lack of maintenance. Fulton County has been interested for some time in retrofitting these structures to provide at least a minimum of water quality attenuation; the tradeoff being that a portion of the volume dedicated to providing water quality attenuation will then probably not be available for peak storm attenuation. An ongoing GIS inventory within Fulton County has collected data such as surface area and volume characteristics for many of these detention ponds. Digital data on catchment delineation, topography, slopes, and land use is available.

Based on applying different strategies, we have developed a methodology for quickly and expeditiously identifying possible candidate ponds to retrofit. The reader should be cautioned that this is not intended to be a rigorous hydrologic design. It is possible that due to inaccuracies in this method, some potentially good candidate ponds may be missed (a false negative). A key tradeoff is being made; i.e., attenuation volume is being traded for water quality volume; the assumption is that little if any attenuation is taking place at the present time. The method leaves room for additional hydrologic analysis to take place in identified circumstances where it is warranted.

Newly established practices in metropolitan Atlanta for detention/retention pond design are to attempt to satisfy multiple criteria, including an allowance for a water quality volume, a channel protection volume, and an attenuation volume (which could consist of multiple volumes for multiple storms). These volumes are viewed as additive, and stack, as shown in Figure 2

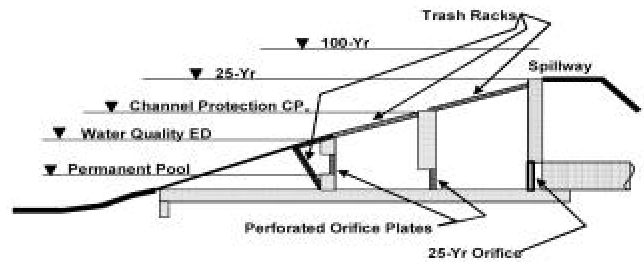

Figure 2. Detention Pond Volumes (from the Georgia Stormwater Manual (Atlanta Regional Commission, 2001).

from the Georgia Stormwater Manual (Atlanta Regional Commission 2001). However, these criteria were developed for the ideal case in which a new site is being developed. For a retrofit site, options for redesign of the pond itself are limited, and are usually limited to redesign of the outlet structure. In a few limited instances, additional volume can be developed by adding height to the top of the structure. In almost all observed cases, the choices are between water quality volume and attenuation volume; channel protection will need to be considered on a case by case basis in the outlet structure design, if a potential retrofit pond is identified.

The first step in the procedure, following Figure 3, is to collect information regarding the potential ponds. This information includes the storage volume of the pond, V_{det}, the contributing drainage area upstream from it, and to identify % imperviousness of the catchment upstream. The latter is usually the most difficult, however it can be done either by analysis of aerial photography or estimating from a table based upon land use characteristics and density.

Once this is done the water quality volumes and various attenuation volumes must be calculated. The water quality volume is calculated as follows. First the volumetric runoff coefficient, R_v, must be calculated as follows:

$$R_V = 0.05 + 0.009I \qquad \textbf{(1)}$$

where:

R_v = Volumetric Runoff Coefficient
I = % impervious cover expressed as a whole number (e.g., 10% would be 10)

Next, the water quality volume, V_{wq} is calculated as follows:

$$V_{wq} = 30.48(10)R_V A \tag{2}$$

where:

V_{wq} = Water quality volume, m^3
A = Drainage area in hectares

The coefficient 30.48 has been left out separately, as it represents 30.48 mm of rainfall (1.2 inches); which has been determined to be the 85% percentile rainfall in North Georgia, or the "first-flush" of rainfall, carrying the highest amount of the total suspended solids (TSS) loading. This percentile is significant in that it is assumed that 80% of the TSS loading from urban runoff will potentially be removed if the 85% percentile rainfall is captured and held for 24 hours. Since phosphorus is primarily attached to the TSS load, a concomitant reduction in phosphorus loading may be achieved as well at this percentile. If $V_{det} \le V_{wq}$, then the pond is not a good candidate for retrofit. Resources will be better spent on other areas where more TSS reductions can be achieved.

Next, if the $V_{det} \ge V_{wq}$ the attenuation volumes, or V_2, V_5, V_{10}, and V_{25} must be calculated. The USGS (1993) and the Atlanta Regional Commission (2001) present a method for calculating peak flows, for various return periods. This method is based upon generalized regression relationships between rainfall and streamflow throughout the Atlanta metropolitan area using two key variables, drainage area and the % impervious area. These relationships vary by region; Fulton County is located in what the USGS has identified as Georgia Region 1 (Figure 1).

The regression equations are listed in Table 1, grouped by urban, or developed conditions, Q_i; and by rural, undeveloped conditions, Q_o. The "i" and the "o" nomenclature is intended to reflect the inflow to the pond (developed) and the allowable outflow (undeveloped). For developed basins, the range of applicability of the regression is restricted to drainage areas with a minimum of 0.1 km^2, or about 25 acres, and a maximum of 50 km^2, and total impervious area % of between 1 and 62 %.

Table 1. USGS Peak Flow Regression Equations for North Georgia (Region 1)[1]

Return Period (years)	USGS Regression Equation[2] PostDeveloped Q_i	PreDeveloped Q_o
2	$2.36A^{0.73}I^{0.31}$	$3.15A^{0.654}$
5	$4.34A^{0.71}I^{0.26}$	$5.54A^{0.632}$
10	$5.89A^{0.70}I^{0.21}$	$7.57A^{0.619}$
25	$7.67A^{0.70}I^{0.20}$	$10.60A^{0.605}$
50	$9.44A^{0.69}I^{0.18}$	$13.29A^{0.595}$
100	$10.77A^{0.69}I^{0.17}$	$16.41A^{0.584}$

[1]Source USGS (1993) as presented in the Georgia Stormwater Manual, Atlanta Regional Commission (2001)
[2]A is in km^2 and I is in % impervious area, i.e., 10% is 10, not 0.10. Q is in m^3/second

Lag time (from the USGS, 1993) in Region 1 is calculated for developed, urban basins as follows:

$$T_L = \frac{0.395A^{0.35}}{I^{0.22}S^{0.31}} \tag{3}$$

where:

T_L = Lag time to the peak in hours
A = Drainage area in km^2
I = % impervious cover expressed as a whole number
S = Slope of the drainage area, m/m, dimensionless

For rural, or undeveloped basins, the following relationship governs:

$$T_L = \frac{0.481A^{0.69}}{S^{0.21}} \tag{4}$$

T_L is assumed to be approximately equivalent to t_c, or the time of concentration of the catchment. The resultant triangular hydrograph, based upon the SCS

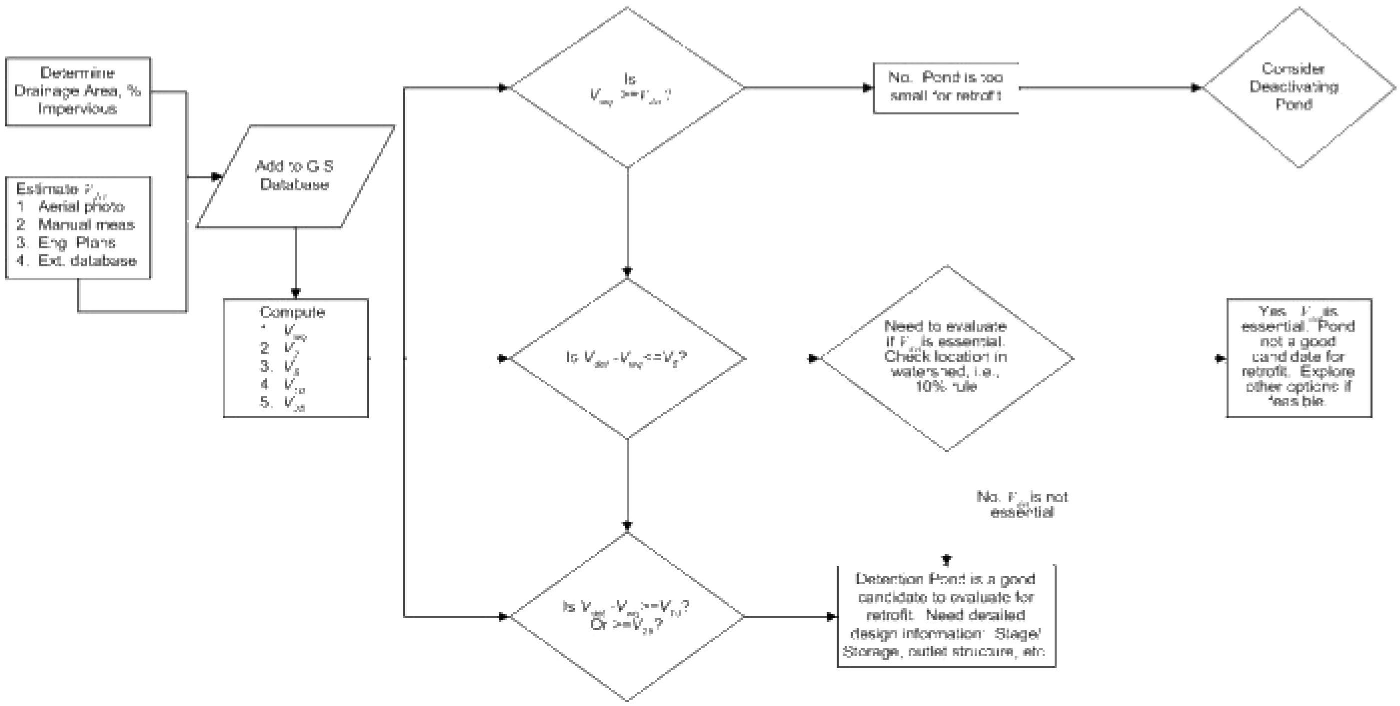

Figure 3. A Screening Procedure for Assessing the Feasibility of Retrofitting Existing Stormwater Detention Facilities.

method, according to Pilgrim and Cordery (1993) has a time to peak, T_P of:

$$T_P = 0.5D + 0.6t_c \quad \textbf{(5)}$$

where:

T_P = Time to peak of the hydrograph in hours
D = Duration in hours

For this purpose, a 1-hour duration will be assumed. Based upon triangular unit hydrograph, the volume needed to be held is:

$$V_n = (3600)(0.5)T_i\left(Q_i - Q_o\right) \quad \textbf{(6)}$$

where:

V_n = Required volume of pond, in m^3
n = (Subscript) Return period, i.e., 2, 5, 10, 25-year
T_i = Duration of contributing drainage area inflow, hours
Q_i = Peak developed pond inflow in m^3/s
Q_o = Peak predeveloped, or allowable pond outflow in m^3/s

The factor 3600 is required to convert time in seconds to hours. In the SCS method,

$$T_i \cong 2.67T_P \quad \textbf{(7)}$$

The volumes can now be evaluated via a spreadsheet or handheld calculator. At the time of this writing, a minimum goal of attenuation had not been set. If a retrofit is considered, and the water quality volume is subtracted, by default the residual is dedicated to attenuation. A reasonable goal would be a 5 or 10-year storm duration. If some minimum volume associated with a given return period storm is not met once the water quality volume is subtracted, another analysis may become necessary. In some cases, due to volumetric increases in runoff from increases in impervious surface, and/or timing, there may be a case for not retaining any volume for peak attenuation. Volumetric increases for the most part, may result from development in larger drainage areas that require a detailed hydrologic analysis; and are not typical of the basins in question. However, there exists the potential for a timing mismatch (see Figure 4).

The concept of timing is explained as follows (Atlanta Regional Commission 2001). Due to its location in a

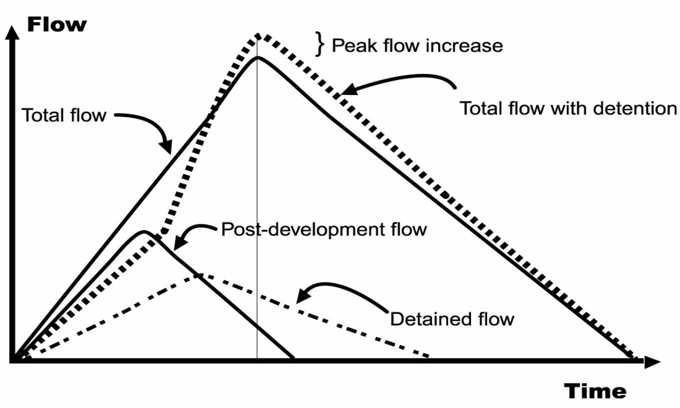

Figure 4. Detention Pond Timing Analysis (from the Georgia Stormwater Manual, Atlanta Regional Commission, 2001).

watershed, slowing down the peak runoff from the development may shave the peak discharge, but when this flow is aggregated with that of the entire basin, the resultant is an increase in discharge. This is due to the difference in lag times between the drainage area upstream from the detention pond, and the watershed it is a part of. In general, the timing problem can be avoided by evaluating the hydrology downstream to a point where the drainage area only consists of approximately 10% of the watershed. If the resultant hydrograph indicates an increase in peak discharge in a 5-year or 10-year event, then the pond probably should be redesigned to be a water quality pond, and attenuation neglected.

Once this procedure has identified possible candidate ponds for retrofit, a detailed design of the outlet structure and a more detailed hydrologic analysis may be done, if necessary. At this time, it will then be possible to re-evaluate the actual performance of the pond. For example, in U.S. Environmental Protection Agency (1986), a procedure was developed which allows the performance of the pond in terms of TSS removal to be simulated as in a treatment system. The volumes dedicated to a single purpose in the pond actually serve multiple purposes, a fact recognized in Guo (2002). This author develops a method for incorporating the probability of occurrence of multiple rainfall events, in terms of detention pond performance for water quality. This latter method has the effect of allowing the "dedicated" volumes to be shared; and then evaluating the performance of the system through statistical techniques. If these methods are included in the analysis, this may actually result in performance gains in terms of water quality treatment in the pond for a given volume.

APPLICATION

An example application of this method is on a detention pond located in a subdivision in the Sandy Springs area of North Fulton County. The area consists of a drainage basin of 0.04 km^2, which is 30% impervious, and has a slope of 0.0457. This is out of the range of the regression applicability, however, it is included here for illustrative purposes. A similar procedure can be developed based upon the Rational method for small sites. The subdivision detention pond's volume is approximately 515 m^3. Based upon the above analysis, the resultant volumes are presented in Table 2. Once the water quality volume is subtracted, a difference of 105 m^3 is left. Since this is less than the 2-year volume of 455 m^3, it appears that, unless the pond is located in a downstream subbasin (i.e., 10% rule), or additional volume can be added easily, it is not a good candidate for retrofit.

Table 2. Example Retrofit Evaluation

Type	Volume, m^3
V_{wq}	410
V_2	455
V_5	598
V_{10}	406
V_{25}	148

CONCLUSIONS

A procedure has been developed to identify potential candidates for retrofitting older detention/retention ponds into multi-purpose water quality/flood attenuation structures. This procedure makes several critical assumptions:

1. Detention pond volumes cannot be shared.
2. That the performance at removal of TSS is fixed at 80%.
3. That most of the ponds are not currently attenuating peak flows to a significant degree.

Relaxing the first assumption alone would result in less conservatism and would actually result in more ponds being identified, using a more complex statistical method. It would then also become necessary to choose an acceptable threshold for reliability as well as the design return period. However at this stage it is thought that targeting resources at retrofitting the best candidate ponds is the best strategy.

Performance in the field may vary from the 80%. If warranted, additional modeling could be done to attempt to predict treatment performance.

A possible incentive for retrofitting identified candidate detention ponds would be the offset of additional loading from point sources which are currently discharged to streams that have met their assimilative capacity and/or TMDL limitations. Prior to investing in widespread implementation of this approach, a Cost/Benefit analysis should be performed. Benefits could be estimated associated with the next level of treatment plant capacity.

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