CHANGES IN RUNOFF DUE TO STORMWATER MANAGEMENT POND REGULATIONS

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ABSTRACT: A continuous simulation stormwater management model (with 33 years of historical precipitation) was used to determine how the design criteria from five different stormwater management pond ordinances changed the runoff from a 7.77 ha watershed following a hypothetical development scenario. All five evaluated ordinances required that the postdevelopment runoff rates from the site be less than or equal to the predevelopment runoff rates for each return period (a zero increase criteria). However, none of the five ordinances were effective at limiting the increase in runoff peak rates for the 1- and 2-year return periods for the annual runoff series, the 1-year return period for the annual exceedance (partial) runoff series duration, or more frequent precipitation events. To better control the lower frequency runoff events from stormwater ponds, the 1- and 2-year return periods should always be analyzed for a basin's design. Additionally, there was a radical change in the frequency of small and moderate runoff events occurring from the watershed following the hypothetical development.

INTRODUCTION

The number of stormwater management requirements for new developments in the humid northeast has increased markedly in the past 30 years. Hagen (1995) reports that the number of hydrologic studies of small urban watersheds, conducted to comply with stormwater management requirements in the United States, is greater than 20,000 per year. These requirements are generally in the form of state regulations or local ordinances that are intended to require development designs so that storm event peak runoff rates are not increased by the reduction of the pervious area caused by construction of new developments. The end result in the design is often the construction of a stormwater management pond on the developed site. The use of stormwater retention and detention ponds is now a common feature of large and small developments.

Increase in the use of stormwater management ponds was one of the primary reasons the ASCE produced the manual on the *Design and Construction of Urban Stormwater Management Systems* (Design 1993). The manual recognizes that stormwater management design is bounded by tradition and that, ''Common sense has frequently been overridden by adherence to arbitrary standards.'' However, what the manual does not state is that the majority of small (<40 ha) watershed stormwater management designs are now being conducted by engineers, surveyors, or landscape architects not specifically trained in hydrology and hydraulics. As this trend grows, the reliance on arbitrary standards and prepackaged ''stormwater management models'' will likely increase.

The increase in regulatory standards and the use of computer models for stormwater management pond design are some of the reasons this study has been conducted. Another reason is that practitioners often forget what the original assumptions and limitations of computational methods and design practices are as the methods or practices become ''standard practices.''

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NUISANCE FLOODING

For the most part, stormwater management ponds are successful at attenuating peak runoff rates from developed areas for larger return period events (>5-year events). However, in several areas, a general mistrust in stormwater management practices has developed among the lay community and, in some cases, regulatory agents. This mistrust is illustrated by the large numbers of people who attend and protest against plans for future developments based on the proposed site's stormwater management issues. Engineers attending municipal planning meetings can often find themselves in front of hostile audiences trying to convince these people that no additional flooding will occur due to the proposed development. Public pressure is forcing some municipalities to require developers to unofficially (not prescribed by ordinance) model local historical precipitation events with proposed stormwater management ponds.

The mistrust in stormwater management practices often has developed when homeowners and businesses experience flooding land or buildings after upstream stormwater management ponds (hereafter referred to as ponds) were constructed in conjunction with developments. A classic example is a homeowner's basement that floods for the first time, and frequently thereafter, following the construction of a large impervious area and pond directly upslope. This type of flooding is defined as nuisance flooding, which although not normally life threatening, is a temporary inconvenience and causes financial burden on community residents impacted by the flooding.

Nuisance flooding can result from developed sites for which adequate stormwater management was not provided and also at sites where stormwater management ponds are constructed. This can occur even when a pond is designed such that the maximum peak rate of runoff from the site following development is less than the design specified peak rate of runoff prior to development for any given design storm event (a zero increase standard).

One way nuisance flooding occurs is when an engineer or designer over predicts the predevelopment runoff of a watershed. Using 37 gauged watersheds (759 total years of record, average = 20.5 years) Fennessey (2000) showed that of the 37 watersheds tested, 25 were either over or under predicting the historical runoff rates by more than 30% with 7 in error by 700% (up to 1,350%).

Nuisance flooding can also occur when property boundaries are used to represent watershed boundaries, as is commonly done in the land development industry. Using these ''hypothetical'' watersheds (property line based) often excessively concentrates the developed site's runoff at a single discharge point, which generally results in nuisance flooding directly below the pond's out-fall. Fennessey (2000) found that upslope hypothetical watersheds had traditionally determined curve numbers (CN) 10 to 40 values too high when used in the Natural Resource Conservation Service (NRCS, formerly SCS and from hereon referred to as the SCS) CN runoff model. The result was extremely high over-estimates of runoff rates (as compared to gauged runoff rates).

OBJECTIVES OF STUDY

The purpose of this study was to determine if regulatory design standards could be contributing to the occurrence of nuisance flooding below newly constructed stormwater management ponds in Pennsylvania. A multitude of different design standards exist from state-to-state, county-to-county, and municipality-to-municipality. Pazwash (1993) recommended that the State of New Jersey and its municipalities and counties adopt a unified drainage code for stormwater management purposes. Unfortunately, until now, New Jersey as well as the other eastern states have regulations based on established engineering practices, intuitive reasoning, and localized results, without a comprehensive analysis of the impact of different types of regulatory standards on a watershed's overall hydrologic response. Therefore, the objective was to provide regulatory agencies and design engineers with an analysis of long-term simulation data regarding how different stormwater management pond design criteria affect a watershed's runoff response. The study evaluated the simulated pond discharge versus the historical runoff for 33 years for five common regulatory standards that are used in the design of stormwater management ponds.

OVERVIEW OF METHODOLOGY

A small pasture research watershed was analyzed for both precipitation and runoff characteristics for a period of 33 years. The compiled historical data consisted of: (1) the daily, annual, and annual exceedance (partial) series peak runoff rates; (2) annual and daily runoff volumes; (3) daily, 24-hour, and 2-day total precipitation depths for both the annual and annual exceedance series; and (4) the annual precipitation depths. The annual and annual exceedance series runoff peak rates were used to compute a Log Pearson Type III probability distribution. Following the historical analysis, the watershed was assumed to be developed as a commercial site that included a stormwater management pond. The site's stormwater management pond was designed using the Virginia Tech/Penn State Urban Hydrology Model (VTPSUHM) (Seybert and Kibler 1997) so the design would be approved in each municipality that had its stormwater management pond criteria evaluated. (Fennessey, Aron, and LaSota designed and constructed well over 300 stormwater management ponds, including at least one from each county for which ordinance criteria were evaluated.)

The pond outlet structure was designed using synthetic SCS Type II, 24-hour precipitation events so that the site discharge was controlled using five different, but common, zero-increase ordinances. This phase was identical to what actually is done in the stormwater management industry in the eastern United States. This design phase was conducted to determine the stage/discharge and stage/storage relationships for the pond, which an engineer would use so that the site complied with the stormwater management ordinances.

A continuous simulation stormwater management model was used to model the full 33 years of continuous historical precipitation data using the five different pond designs that were developed in the synthetic design phase. The results of the pond discharges for each of the five ordinances were then compared to the historical data. A flowchart of the overall methodology can be seen in Fig. 1.

WATERSHED HISTORY AND DESCRIPTION

The watershed used in the study is located on the Southern Piedmont Conservation Experiment Station near Watkinsville, Ga. Table 1 provides a summary of physical characteristics of the watershed, and Fig. 2 shows the original SCS watershed map. Although the site is located in the southeastern United

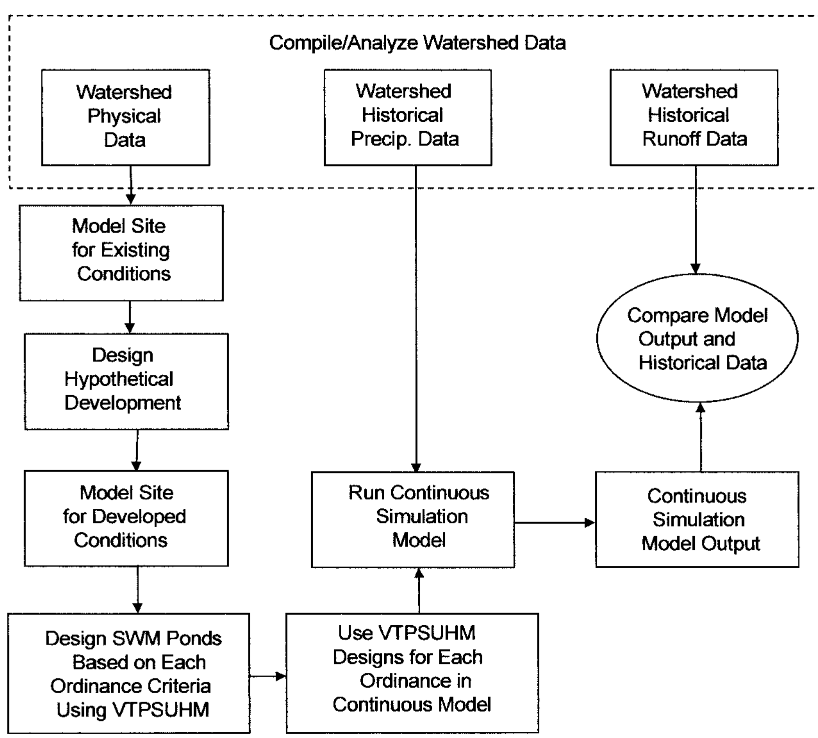

FIG. 1. Flow Chart of Methodology

TABLE 1. Description of Study Watershed Near Watkinsville, Georgia

Physical characteristics	Value
ARS ID. No.	10001
Nearest town	Watkinsville
State	Georgia
County	Oconee
Historical watershed No.	W1
Latitude	335338
Longitude	832530
USGS quadrangle	Athens West
Approximate location	7 mi. SW of Athens
Landcover 1947–1979	Pasture
Area (ha)	7.77
Aspect	West
Average slope (%)	7
Slope range (%)	3–10
NRCS hydrologic soil group	B
ARS flow type	Ephemeral continuous
Type of watershed	Natural draw
Shape of watershed	Fan
Overall length (m)	366
Maximum width (m)	275

States, it was chosen for several reasons: (1) the watershed's size and shape make it an ideal candidate for a small commercial development; (2) the hydrologic model (SCS's TR-55) reasonably predicted the peak runoff rates, thereby eliminating the potential of nuisance flooding due to an over prediction of the runoff by a model; (3) the watershed soils are comprised of a single SCS Hydrologic Soil Group, Group B; (4) the watershed cover was good continuous pasture for 33 years (from 1947 to 1979); (5) the watershed data are well documented with both runoff and precipitation data in continuous form; and (6) the period of record contained no snowfall or snowmelt events.

Runoff and precipitation data collection at the watershed started on September 13, 1939. Kudzu was first planted in 1944 and became well established in 1947. Grazing started in the summer of 1950. From 1947 to 1979, the watershed was in continuous grass cover (ranging from kudzu-rescue grass to coastal Bermuda grass). Runoff and precipitation data collection continued until 1984, and data are available from the Agricultural Research Service (ARS) Water Database. However, land cover data beyond 1979 are not publicly available and, therefore, the last year used in the study was 1979.

ORDINANCES TESTED

The five ordinances tested are commonly used on the east coast of the United States. The ordinances were:

- Zero increase in peak runoff rate for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year precipitation events (referred to as the 1-100 year design).
- Zero increase in peak runoff rate for the 2-, 5-, 10-, 25-, 50-, and 100-year precipitation events (referred to as the 2-100 year design).
- 75% release rate of the peak runoff rate for the 2-, 5-, 10-, 25-, 50-, and 100-year precipitation events (referred to as the 75% release rate design).
- The State of New Jersey's zero increase in peak runoff rate for the 2-, 10-, and 100-year precipitation events with a 1-year water quality event (referred to as the NJ WQ design). The water quality criteria require that no more than 90% of the runoff from a SCS 1-year, 24-storm or a 2-hour storm of 3.175 cm/hour intensity is released in 36 hours for a commercial development. A SCS storm was used in this study.
- The 10-year postdevelopment to 2-year predevelopment match with a zero increase for the 100-year precipitation event (referred to as the 10 post- to 2 predesign). This is a common ordinance in Pennsylvania and is often used in areas with a history of flooding.

Assuming that the modeled predevelopment ground cover is all meadow, regardless of the actual ground cover, is a common ordinance standard used in Pennsylvania that attempts to underestimate predevelopment peak runoff rates. These ordinance criteria were not evaluated in this study for two reasons; first, the watershed's actual ground cover was already pasture, and second, this method of applying a factor of safety is without any apparent physical justification. However, the method's existence and use show that many municipalities are concerned with the results of using current synthetic design methods for stormwater management purposes.

HYPOTHETICAL SITE DEVELOPMENT

The site was hypothetically developed using a design with 70% impervious coverage (impervious area includes the building square footage, parking areas, and drives). The impervious area was selected at 70% because of its common use in zoning ordinances for commercial developments. The pond was considered a part of the 30% pervious area. The pond was not considered to have an impervious lining and was allowed to provide an abstraction for surface runoff from the impervious areas. The site was graded so that all surface runoff was directed to the pond via overland flow over pavement or through storm drains. The site design is shown in Fig. 3. The stormwater design criteria were:

- All runoff from impervious areas flowed directly to the pond, and the impervious areas were considered connected.
- Any up-slope, off-site runoff was assumed to be diverted around the site.
- Only one pond was used.
- Emergency spillway elevation was set at the peak 100-year water surface elevation.
- Pond design attempted to discharge 100% of the design development values, and uncontrolled losses were assumed negligible.
- Outflow orifices were designed to match design discharge without regard to safety or maintenance issues.
- Pond stage/storage relationship was the same for each ordinance tested.

Additionally, to use a realistic site design, the following site design criteria were used: (1) storm drains were approximately 1.5 m below the macadam surface; (2) the site was graded with an average slope of 3.5% in the parking area; (3) the site earthwork (cut and fill) was balanced and, therefore, set site elevations and access roads were designed in accordance with Pennsylvania Department of Transportation specificiations; (4) two hypothetical off-site access roads were assumed to exist along the southern boundaries; (5) the pond abutments had a combined slope of 5:1 and were 3 m wide at the crest; and (6) the pond bottom slope was maintained at 2%.

INDUSTRY SYNTHETIC DESIGN MODELS USED

The methods prescribed in *Technical Release No. 55* (TR-55) (USDA SCS 1986) were used as the basis for design of the stormwater ponds. This method was used, because over 60% (12,000+) of the hydrologic studies conducted per year in the United States were identified as using variants of the CN method, not including the HEC-1 analyses that may have been performed using CN methods (Hagen 1995). Hagen

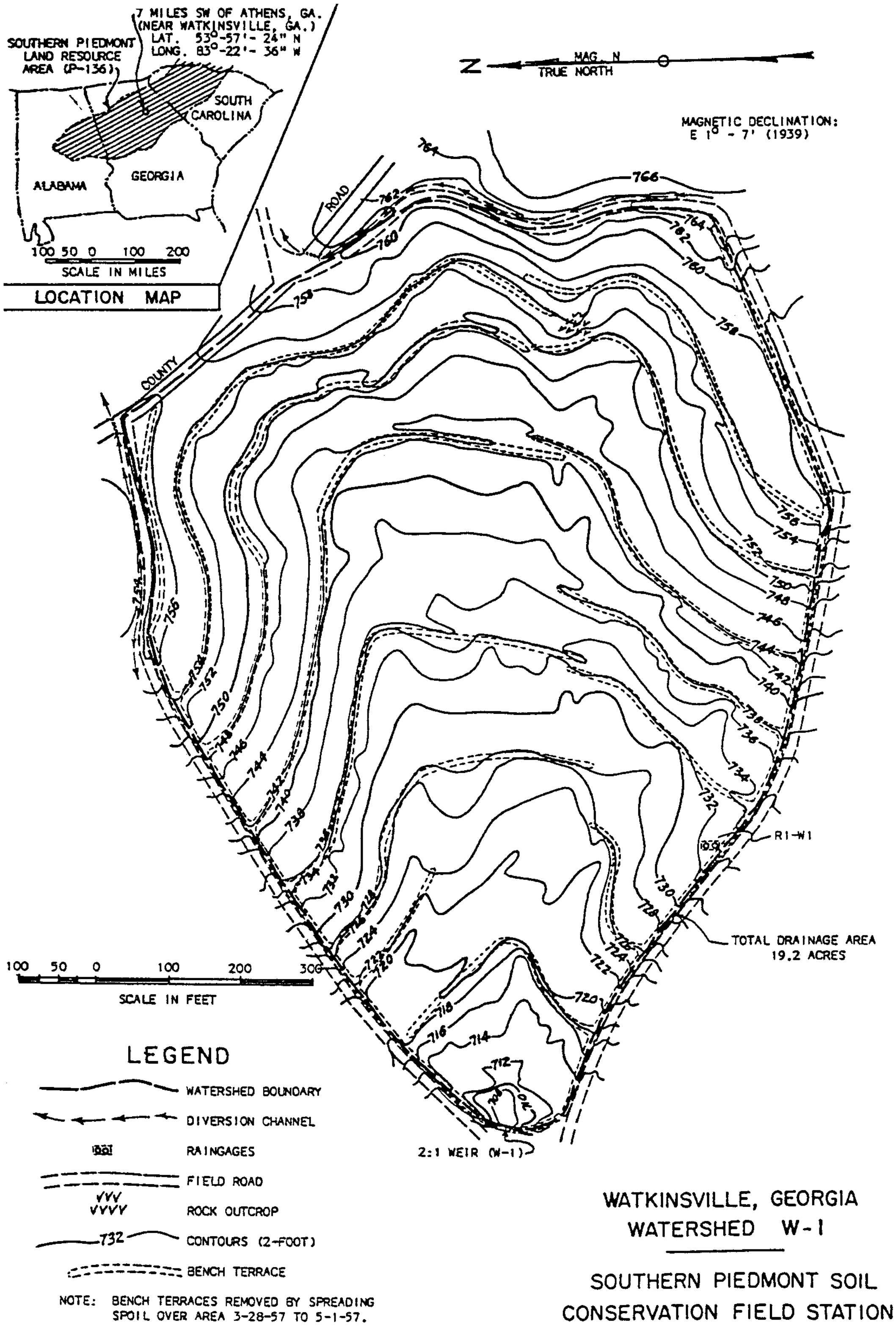

FIG. 2. Original Watershed Map (from USDA SCS 1953)

noted that 10,763 of these 12,000+ were conducted using TR-55. TR-55 is a simplified CN-based methodology that is used most often to design or plan stormwater management structures (Miller and Woodward 1994).

VTPSUHM was used to conduct a standard industry site design and analysis. VTPSUHM contains a TR-55 subroutine, which was used for both pre- and postdevelopment hydrograph generation. The Multiple Stage Routing Module (MSRM) in VTPSUHM was used for the pond design. The synthetic precipitation events used were the SCS's Type II, 24-hour depths, which are the same as the U.S. Weather Bureau *Technical Paper No. 40* (TP-40), 24-hour depths (USDA 1986). The TP-40 storm precipitation depths represent the partial series precipitation depths (USDC 1961), therefore, model estimates reflect the partial series runoff rates.

The SCS's segmental method was used for time of concentration (Tc) computations for both pre- and postdevelopment design, in accordance with industry convention and as recommended by the SCS (Miller and Woodward 1994). Although the Tc is one of the most uncertain and misused syn-

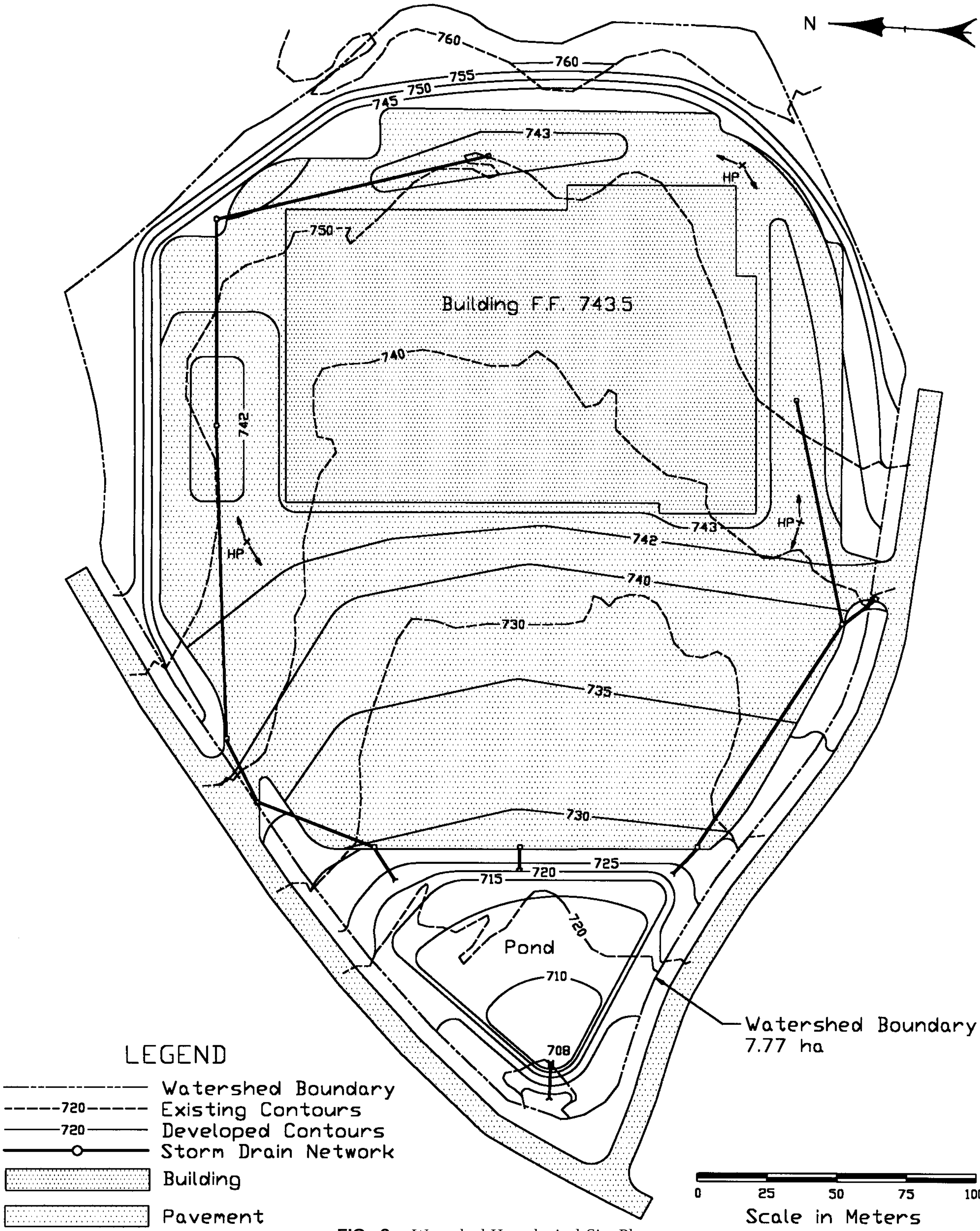

FIG. 3. Watershed Hypothetical Site Plan

thetic design parameters in stormwater management pond design, it should be noted that the TR-55 computations predicted the runoff rates (predevelopment) reasonably (as compared with the historical partial series duration data). Table 2 provides a summary of all five stormwater pond designs.

For each of the five ordinances tested, column 2 of Table 2 represents the predevelopment runoff rate estimates from TR-55 for each return period, and column 3 of Table 2 represents the postdevelopment runoff rates. It should be noted that pre- and postdevelopment runoff rate estimates are the same for each return period, regardless of the ordinance criteria tested. Column 4, Table 2 presents the actual pond discharges that were designed for each structure.

The pond outlet structure was designed considering only the return periods required by the ordinance without any regard to other return periods. This can be observed in Table 2 by reviewing the results of the Standard 1-100 year (a) and Standard 2-100 year (b) designs. One will note that the 2-100 year

TABLE 2. Synthetic Stormwater Management Site Designs for Watkinsville Watershed Using TR-55 (Partial Series Runoff Rates)

Return period (years)	TR-55 Pre-development runoff (*L/s*)	TR-55 post-development runoff (*L/s*)	VTPSUHM MSRM pond outflows (*L/s*)	Pond Outflows as a % of TR-55 predevelopment runoff (%)	Historical runoff (*L/s*)	Difference between pond outflows and historical runoff (%)
(a) 1-100 Year Design						
1	173	1643	173	100.0	142	22.0
2	326	2107	309	94.8	280	10.1
5	626	2860	583	93.2	493	18.4
10	895	3466	796	88.9	722	10.2
25	1303	4288	1266	97.2	1172	8.0
50	1750	5117	1747	99.8	1580	10.6
100	1844	5285	1841	99.8	2053	−10.3
(b) 2-100 Year Design						
2	326	2107	326	100.0	280	16.4
5	626	2860	596	95.3	493	21.0
10	895	3466	892	99.6	722	23.5
25	1303	4288	1261	96.8	1172	7.5
50	1750	5117	1747	99.8	1580	10.6
100	1844	5285	1844	100.0	2053	−10.2
(c) 75% Release Rate Design						
2	326	2107	239	73.3	280	−14.8
5	626	2860	466	74.5	493	−5.3
10	895	3466	674	75.3	722	−6.7
25	1303	4288	945	72.5	1172	−19.4
50	1750	5117	1307	74.7	1580	−17.3
100	1844	5285	1407	76.3	2053	−31.5
(d) New Jersey Water Quality Design						
WQ	173	1643	—	NA	142	NA
2	326	2107	314	96.5	280	12.1
10	895	3466	886	99.1	722	22.7
100	1844	5285	1827	99.1	2053	−11.0
(e) 10 Post to 2 Predesign						
2	326	2107	NA	NA	280	NA
10	895	3466	326	100.0	722	−54.9
100	1844	5285	1844	100.0	2053	−10.2

[a]WQ = 90% max release of runoff volume from a 1-year, 24-SCS event in 36 h.

design is not simply the 1-100 year design with the 1-year event removed. This method tests for differences between the ordinance design criteria, and not the common sense or skill of the design engineer. This is valid because, if the designer of a stormwater pond does not use a return period for a pond's design, say the 5-year event, then there is no way for the engineer to target or know if the 5-year return period has a zero increase if the designer only used the 2- and 10-year return periods for the pond's design.

Some may argue that using the TR-55 predevelopment estimates (Table 2, column 2) instead of the actual historical runoff rates (column 6 of Table 2) would introduce significant error in the study. However, of 37 watersheds analyzed by Fennessey (2000), the difference between the TR-55 predevelopment estimates and the historical runoff rates shown in Table 2 can be considered within a reasonable range of estimation. In fact, the watershed used in this study, which was one of the 37 watersheds, had the fourth best overall TR-55 estimates. Nonetheless, the difference between the design pond outflows and the historical runoff rates (Table 2, column 7) will be discussed further in the results section.

CONTINUOUS STORMWATER MANAGEMENT SIMULATION MODEL

General Model Description

The model is a simple, lumped parameter, continuous simulation hydrograph model linked to a continuous simulation version of the Modified Puls Method. The model consists of 12 linked subprograms/steps and is written in FORTRAN 77 for use on a personal computer. The program is written so that computations and results are written to files instead of arrays, therefore, the maximum period of record for model simulation is dependent only on the FORTRAN compiler or PC used. Check files, which are used for verification purposes, are written in each program section. The model uses historical precipitation data from the ARS Water Database and determines the pond discharges for conditions when the site is developed with a stormwater management pond.

The program first converts the original ARS files, which are variable time increment files, into a precipitation file with a 1-min time step. The precipitation distribution is assumed to be linear between ARS time steps. The 1-min time step coincides with application of the SCS unit hydrograph theory to the Watkinsville watershed. The program defines the number of distinct storms in the record set by the length of time between precipitation events. Although Victor Mockus, in a 1964 personal letter (Rallison and Miller 1981) defined a continuous storm as one with no periods without rain more then 1-h long in respect to the CN (USDA 1993), a storm separation for the model of 1440 min was used to define the beginning of a new storm. The definition of a new storm allows the initial abstraction to be set back to zero. Some would argue that this time is unrealistically long, however, one must remember that the simulated site has been hypothetically developed with 70% impervious area. In reality, ponding (or other abstractions) on impervious surfaces generally lasts longer than a couple of hours. The storm separation time is also valid for the pond abstractions, because the 1440-min time period coincides with the average drawdown time of the pond. Although these as-

TABLE 3. Parameters Used to Model Watkinsville Watershed in Continuous Stormwater Simulation Model

Parameters	Value
Total site area (ha)	7.77
Percent impervious area (%)	70
Storm seperation time (min)	1440
Computational time interval (min)	1
SCS Dimensionless unit hydrograph K	484
Initial abstraction, Ia	0.2S
Antecedent moisture condition, AMC	AMC II
(a) Subarea 1	
Total area (ha)	5.96
Impervious area (ha)	5.44
Pervious area (ha)	0.52
CN for impervious area	98
CN for pervious area	61
Weighted CN (rounded down)	94
Tc (hours)	0.1
(b) Subarea 2	
Total area (ha)	1.81
Pervious area (ha)	1.81
CN for pervious area	61
Tc (hours)	0.1

sumptions are simplistic, it seems that no physically based model could simulate the scenario more accurately because of a lack of data and the complex interactions and unknowns caused by constructed soil lining and the variable (generally close) depth of the water table to the pond bottom.

The model computes the excess precipitation hyetograph using the SCS CN approach as an abstraction term (considering that 70% of the watershed is impervious). So that the upslope grass area would not be modeled as an abstraction term for the impervious areas, the watershed was modeled as two subareas. The first subarea included all the site impervious area and the internal area of the stormwater management pond. Therefore, the model allowed abstractions of the pavement runoff in the pond only. The other subarea consisted of pervious surfaces on the site.

The SCS dimensionless unit hydrograph theory was used for the runoff hydrograph generation from each subarea and the two hydrographs were combined. Parameters in the model can be changed for different simulations; however, the parameters must be held constant for an entire model run. The resulting hydrograph for the full record can be run through a continuous version of the Modified Puls Method. The parameters used to model the Watkinsville watershed are listed in Table 3.

Model Sensitivity Analysis

Individual components of the simulation program were verified for several single and multiple events using established models. Additionally, the postdevelopment synthetic SCS Type II, 24-hour precipitation events that were used in the design phase with VTPSUHM were modeled in the continuous simulation model to verify that the two models produced similar results. The following parameters were modified to determine model sensitivity: (1) storm separation tme; (2) initial abstraction; (3) postdevelopment Tc; and (4) antecedent moisture condition (AMC). Additionally, the contribution of grass runoff was removed completely from the model (total drainage area to pond was changed from 7.77 ha to 5.96 ha).

Storm separation time, initial abstraction, and post development Tc changed the output negligibly. Altering the AMC value from AMC II to AMC I resulted in a slight reduction in the magnitude of runoff peaks; however, the frequency of runoff remained virtually unchanged. Removing the grass area resulted in a minor reduction in the magnitude of large pond discharge events but did not affect runoff frequency.

RESULTS

Model results showed that all five ordinances were effective at reducing the peak runoff for the largest historical events (>10-year event) for both the annual and partial duration series, as shown in Tables 4 and 5. However, none of the ordinances were effective at producing a zero increase in runoff peaks (postdevelopment as compared to historical runoff) for the 1- and 2-year return periods for the annual series or for the 1-year return period for the partial series duration.

Annual series results are presented in Table 4. The 1-100 year design produced results that most closely matched the historical runoff for the entire series and also produced the lowest runoff rates for return periods less than 1.5 years. The 1-100 year design still slightly increased all the annual discharge rates less than or equal to a 2-year return period, as compared to the historical series. The 2-100 year design compares closely with the 1-100 year design except for runoff rates less than the 2-year return period, as would be expected. For the 1-year return period, the 2-100 year design increased the discharge from the site by over 600%, as compared to the historical runoff. Slight differences between the 1-100 and 2-100 year design runoff rates greater than the 2-year return period are due to the actual discharge rate attained in the synthetic design phase of the pond (Table 2, column 5).

The 75% release rate design produced the lowest discharges between the 2- and 5-year return periods and was the second best at lowering discharges between the 5- and 25-year return periods. Although the 75% release rate design, which used the 2-, 5-, 10-, 25-, 50-, and -100 year return periods, was quite effective it still was not as effective as simply using a 100% release rate with the 1-100 year return periods for the 1-year return period annual series.

The two most complex ordinances to implement in a site design (NJ WQ and 10 post to 2 pre) produced some of the poorest results. The NJ WQ design increased (on average) the runoff rates for return periods greater than a 1.5-year return period, the most of any design evaluated. This increase is due to the required retention time for the water quality criteria of previous rainfall events in the pond at the start of a new storm event. Surprisingly the NJ WQ design produced higher discharges for the 1-year return period than both the 1-100 year and 75% release rate designs.

The 10 post to 2 preordinance produced the highest discharge rates for the 1-year return period and the second highest for the 2-year return period. This is because the design did not account for the 1-, 2-, and 5-year return period events (the design criteria stipulated that the 10-year postdevelopment runoff rate did not exceed the 2-year predevelopment runoff rate). The design did produce the lowest discharge rates for return periods greater than 5 years.

The partial series produced results similar to the annual series analysis, with the standard 1-100 year and the 75% release rate 2-100 year designs producing the best overall results (Table 5). All the ordinance designs were better at producing similar results to the historical partial series than the annual series results, likely because partial series precipitation data were used for the synthetic design event. The most notable difference between the partial and annual series results is that the 10 post to 2 predesign produced lower discharge rates than the 2-100 year design for the majority of the partial series.

Some of the differences in runoff rates between the pond discharges and the historical data are attributable to the fact that the TR-55 model estimates were used instead of the historical runoff rates. However, the range of differences for the partial series data (Table 5) between the historical runoff rates

TABLE 4. Annual Series Runoff Rates from Historical Data Compared to Simulated Annual Series Pond Outflows for Five Selected Stormwater Management Ordinances

Rank of storm event based on runoff rate	Approximate historical peak rate return period (years)	Historical Qp (L/s)	1-100 year design simulated pond outflows Qp (L/s)	2-100 year design simulated pond outflows Qp (L/s)	75% R.R. design simulated pond outflows Qp (L/s)	NJ WQ design simulated pond outflows Qp (L/s)	10 post- to 2 pre-design simulated pond outflows Qp (L/s)
1	25	1320	942	1037	796	1000	647
2		942	774	733	598	994	314
3		780	726	661	595	960	309
4	10	693	689	651	539	837	309
5		619	649	628	514	814	307
6		619	585	580	467	633	305
7		581	457	494	362	434	297
8	5	410	346	320	251	421	280
9		390	285	308	245	398	275
10		368	280	298	244	387	274
11		348	277	271	237	359	267
12		315	263	268	225	350	266
13		307	255	255	222	350	262
14		216	238	251	204	344	259
15		193	215	251	198	336	259
16	2	183	212	251	195	331	259
17		156	212	251	194	321	256
18		153	194	251	191	316	252
19		126	193	250	189	304	250
20		123	191	250	187	293	246
21		119	169	236	187	290	245
22		110	167	231	183	275	239
23		103	165	229	181	257	239
24		98	162	228	167	254	237
25		81	146	227	167	240	237
26		78	145	227	159	224	237
27		66	123	221	155	203	231
28		40	110	220	154	201	231
29	1	30	93	220	152	189	231
30		30	88	218	149	183	227
31		24	85	204	137	170	213
32		19	78	197	136	157	208
33		16	47	193	127	151	203

and the pond discharges are 28% to 101% for the 1-year return period compared to the percentages between the historical runoff rates and the TR-55 estimates (Table 2, column 7). Even more dramatic are the differences between the annual series historical runoff rates and pond discharges, which ranged from 210% to 670% for the 1-year return period compared to the percentages between the historical runoff rates and the TR-55 estimates (Table 4).

More significant than the increase in low return period annual or partial series runoff peaks is the change in the total population of runoff events from the site following development (Table 6, Fig. 4). The total number of days simulated in the 33 years was 12,053; 3,384 days of precipitation occurred based on historical data. Only 994 days of runoff occurred in the 33 years (ratio of 1:12.1). After the hypothetical development, the simulated number of days that runoff occurred increased to an average (for all five ordinances evaluated) of 2,712 days (ratio of 1:4.4). As the results show, the additional runoff days did not consist simply of unsubstantial flows.

However, it must be clearly stated that the change in the frequency of runoff is not simply due to the use of a stormwater management pond. On the contrary, without a pond the frequency of runoff would be nearly identical to the pond simulations; however, the magnitude of runoff events would be significantly higher. These data illustrate how the different ordinance criteria changed the overall populations in comparison to each other.

The days runoff implies a total day count and not single runoff events. Therefore, a runoff event that started at 10 p.m. one day and ended at 2 a.m. on the following day would show in the table as two days, in which the peak runoff rate may have been high. This same counting method was used for both the historical and simulated data. The radical change in runoff following the hypothetical development was not simply the result of the pond discharges being extended over additional days. The increase was also due to days in which major precipitation events occurred that produced no runoff during the historical period, but produced large volumes of runoff due to the impervious area. In the 33 years of historical data, there were 813 days that the total daily precipitation was greater than 0.76 cm, and there was no runoff.

For runoff between 170 and 198 L/s (the 2-year historical annual series peak runoff rate range), a radical change in the watershed runoff can be observed for some ordinances. Historically only 32 days had runoff greater than 170 L/s; however, after development, there were 44, 261, 50, 110, and 318 days with runoff greater than 170 L/s as simulated for the 1-100, 2-100, 75% release rate, NJ WQ and 10 post to 2 pre ordinance designs, respectively (summation of number of days in Table 6). Even more dramatic was the change in the number of events with runoff between 28.3 and 42.5 L/s (the 1-year historical annual series peak runoff rate range). Historically, only 139 days had runoff occur greater than 28.3 L/s; however, after development, there were 1,149, 1,435, 1,360, 435, and 1,362 days exceeding 28.3 L/s for the 1-100, 2-100, 75% release rate, NJ WQ, and 10 post to 2 pre ordinance designs, respectively. The change in frequency of events exceeding various peak runoff rates for the annual series return period can be observed in Fig. 4. This radical change in the frequency of runoff was not simply due to the type of precipitation that occurs in Georgia.

The selection of the design return period events does not

TABLE 5. Annual Exceedance Series Runoff Rates from Historical Data Compared to Simulated Annual Exceedance Series Pond Outflows for Five Selected Stormwater Management Ordinances

Rank of storm event based on runoff rate	Approximate historical peak rate return period (years)	Historical Qp (L/s)	1-100 year design simulated pond outflows Qp (L/s)	2-100 year design simulated pond outflows Qp (L/s)	75% R.R. design simulated pond outflows Qp (L/s)	NJ WQ design simulated pond outflows Qp (L/s)	10 post- to 2 pre-design simulated pond outflows Qp (L/s)
1	25	1320	942	1037	796	1000	647
2		942	774	733	598	994	314
3		780	726	661	595	960	309
4	10	693	689	651	539	837	309
5		619	649	628	514	814	307
6		619	585	580	467	633	305
7		581	457	494	362	579	297
8	5	529	369	346	257	462	285
9		424	346	344	256	434	285
10		410	336	320	252	421	280
11		390	288	308	251	398	275
12		368	285	307	245	388	275
13		348	280	298	237	387	274
14		336	277	281	225	359	271
15		315	263	276	222	350	267
16		307	260	271	210	350	266
17	2	288	255	270	207	347	266
18		262	251	268	207	344	263
19		216	238	258	206	336	262
20		214	215	255	204	331	261
21		198	212	254	198	326	261
22		193	212	251	197	324	259
23		188	212	251	195	321	259
24		183	211	251	194	316	259
25		172	205	251	194	307	256
26		171	201	251	194	304	252
27		165	196	251	191	302	251
28		156	194	251	191	301	250
29		153	193	251	190	293	250
30		151	191	251	189	292	249
31		150	188	251	187	290	247
32	1	144	184	250	187	290	246
33		135	179	250	187	289	246

greatly change how ponds are built or the amount of land developers would need to set aside for pond use (considering the same maximum event). In all five simulations, because of the use of multiple stage outlet structures, the design 100-year peak water surface elevations differed by only 12 cm in elevation (using the same pond stage/storage design for all five designs).

CONCLUSIONS

The model simulation shows that none of the ordinances actually resulted in a zero increase of runoff peak rates following development (as compared to the historical runoff rates). As lay people observe, the frequent, low-return period events appear to be where the largest increases in runoff due to stormwater management ponds occur. Complaints of increased nuisance flooding, on an annual or daily basis, appear to be justified based on these findings. Additionally, typical first flush, stormwater management detention ponds that are designed with no dead storage cause a radical change in the frequency (substantial increase) of runoff from a site when used with large commercial developments, which potentially affect downstream morphology.

This study confirms the ASCE statement that, ''Urbanization has a greater impact on frequent events than on rare events'' (Design 1993), even when stormwater management ponds are employed. In studying the data, it becomes obvious that to adequately control runoff from low return period events, the 1-, and 2-year return period designs should always be used in the site design in conjunction with the larger return period events. The majority of stormwater management ordinances in Pennsylvania require that the smallest return period to be used in the design of a pond is the 2-year return period event. This study clearly shows that requiring the 1-year event to be used in the design can greatly reduce the impact of a development on the frequency of increased runoff in locations downstream.

RECOMMENDATIONS

Because of the ready availability of ''pre-packaged'' stormwater management models, all municipalities concerned with the effects of nuisance flooding should, at a minimum, incorporate the 1-, 2-, 10-, and 25-year return period designs into their stormwater management pond ordinances. These events should be analyzed even when using a more complex requirement, such as the predevelopment all meadow requirement, release rate districts, or water quality events. This would result in lowering the magnitude of the low frequency runoff rates from the developed site. Ponds should also incorporate a capture volume (dead storage) to reduce the large number of small runoff events that occur following development.

Any municipality experiencing regular complaints regarding nuisance flooding below stormwater management ponds should first conduct a hydrologic analysis to determine if the prescribed stormwater management models adequately reflect the dominant regional hydrologic processes. If the models are adequate, then a continuous stormwater management simulation model should be run (as described herein). Such a simulation should consider various ordinance criteria in conjunction with long-term local precipitation data prior to developing a plan of action.

TABLE 6. Frequency of Occurrence (Number of Days) of Peak Runoff Rates for Historical Data and Five Selected Stormwater Management Ordinances

Range of runoff events	Historical frequency	Standard 1-100 year design simulated frequency	Standard 2-100 year design simulated frequency	75% R.R. 2-100 year design simulated frequency	NJ WQ design simulated frequency	10 post- to 2 pre-design simulated frequency	Approximate return period: Partial series	Approximate return period: Annual series
0 < Days Qp < 2.8 L/s =	421	507	327	489	697	22		
2.8 <= Days Qp < 5.7 L/s =	160	195	192	208	465	369		
5.7 <= Days Qp < 8.5 L/s =	87	159	140	133	805	307		
8.5 <= Days Qp < 11.3 L/s =	54	119	116	114	191	223		
11.3 <= Days Qp < 14.2 L/s =	33	84	108	113	2	151		
14.2 <= Days Qp < 17.0 L/s =	29	83	104	77	3	58		
17.0 <= Days Qp < 19.8 L/s =	26	110	112	64	3	73		
19.8 <= Days Qp < 22.7 L/s =	27	116	69	62	4	59		
22.7 <= Days Qp < 25.5 L/s =	10	100	62	47	56	46		
25.5 <= Days Qp < 28.3 L/s =	8	90	47	43	52	44		
28.3 <= Days Qp < 42.5 L/s =	40	797	196	163	31	198		1
42.5 <= Days Qp < 56.6 L/s =	14	220	127	137	20	119		
56.6 <= Days Qp < 70.8 L/s =	11	9	106	158	36	104		
70.8 <=Days Qp < 85.0 L/s =	10	25	117	183	55	79		
85.0 <= Days Qp < 113 L/s =	12	20	181	425	104	170		
113 <= Days Qp < 142 L/s =	12	15	146	202	44	128	1	
142 <= Days Qp < 170 L/s =	8	19	301	42	35	246		
170 <= Days Qp < 198 L/s =	7	15	151	21	16	174		2
198 <= Days Qp < 227 L/s =	2	7	59	12	21	72		
227 <= Days Qp < 255 L/s =	1	2	27	5	17	41		
255 <= Days Qp < 283 L/s =	4	5	7	2	15	20		
283 <= Days Qp < 354 L/s =	6	4	9	2	23	10	2	
354 <= Days Qp < 425 L/s =	4	2	0	2	6	0		
425 <= Days Qp < 566 L/s =	1	3	2	3	5	0	5	5
566 <= Days Qp < 708 L/s =	4	3	4	2	2	1		
708 <= Days Qp < 850 L/s =	1	2	1	1	2	0	10	10
850 <= Days Qp < 991 L/s =	1	1	0	0	1	0		
991 <= Days Qp < 1133 L/s =	0	0	1	0	2	0		
1133 <= Days Qp < 1274 L/s =	0	0	0	0	0	0	25	25
1274 <= Days Qp < 1416 L/s =	1	0	0	0	0	0		
1416 <= Days Qp L/s =	0	0	0	0	0	0		

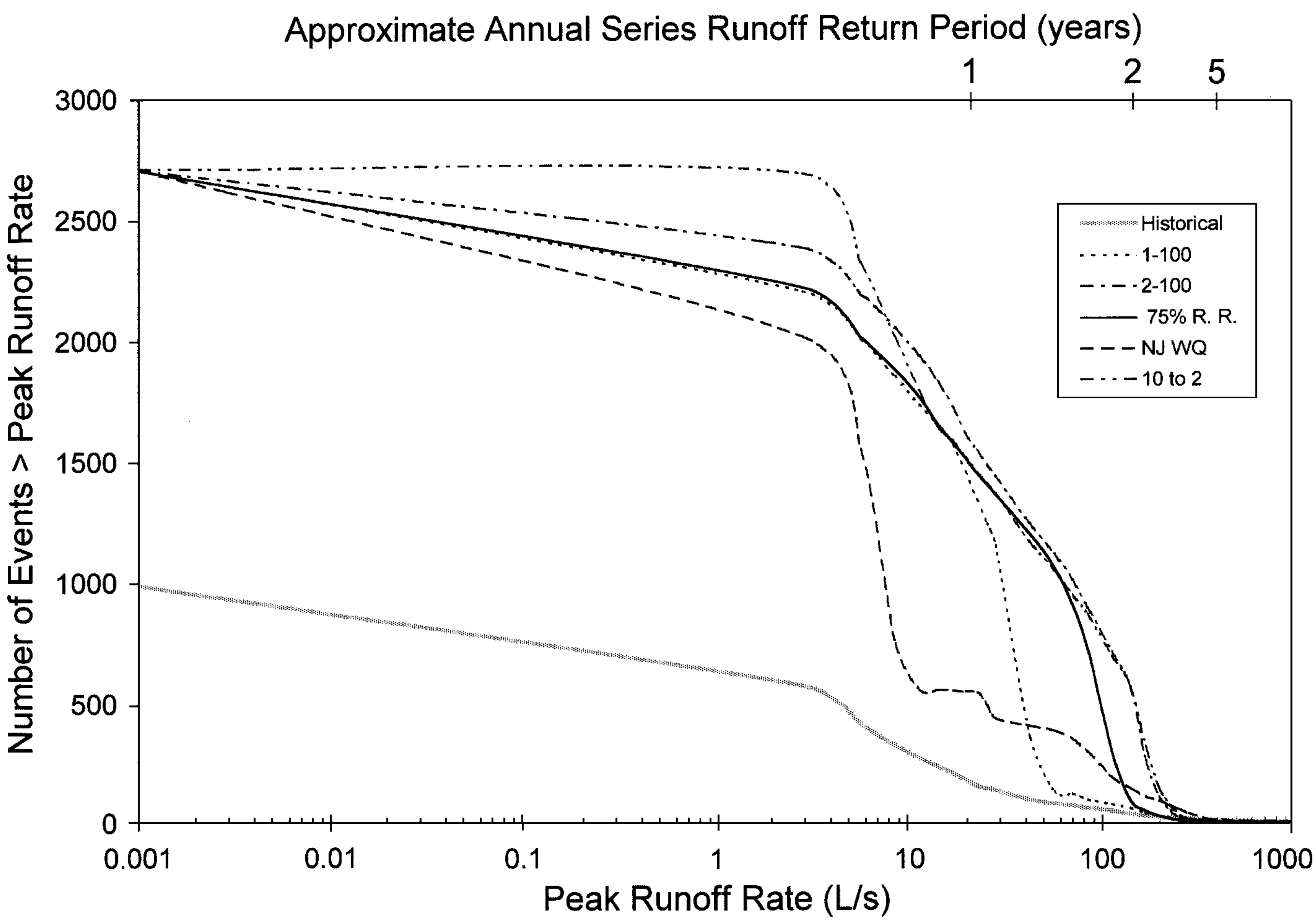

FIG. 4. Total Runoff Population for Historical Data and Five Evaluated Ordinance Criteria

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