16

Modification of Detention Basin Outlet Structures using Calibrated SWMM Models

Charles D. Morris and Joel P. Asunskis

The U.S. Environmental Protection Agency Storm Water Management Model, SWMM, is a large, relatively complex software package capable of simulating the effects of runoff from sub-catchments through pipe/channel networks and storage facilities, and finally to receiving waters. This chapter describes a study where detailed SWMM models were developed as an integral part of a study of the outlet structure and associated modifications. Different outlet structure configurations were compared, thereby resulting in an optimal configuration of the structure.

The need was based on changes in the Metropolitan St. Louis Sewer District's (MSD) detention basin design requirements (Akan, 1989). It was previously required that the outlet structure controls the outflow of the 15-year storm to the same level as the pre-development discharge rate. It is now required that the outlet structure control the outflow of both the 2-year and 100-year outflows to the pre-development discharge rate or to the required basin-wide release rate determined by MSD (McEnroe, 1992). Another factor motivating modification of outlet structures is the observation by a number of interested individuals and communities of "little or no storage" of storm water occurring in these basins during storm events with subsequent downstream flooding. Detailed SWMM modeling of eight different basins in the St. Louis, Missouri, metropolitan area showed "little or no" reduction of the 2-year storm outflow, but significant reduction of the post-development 100-year storm outflow. Thus, the objective reached by modifying the existing outlet structures is to

---

Morris, C.D. and J.P. Asunskis. 2002. "Modification of Detention Basin Outlet Structures using Calibrated SWMM Models." *Journal of Water Management Modeling* R208-16. doi: 10.14796/JWMM.R208-16.

www.chijournal.org ISSN: 2292-6062 (Formerly in Best modeling practices for Urban Water Systems. ISBN: 0-9683681-6-6)

reduce the 2-year storm outflow to the pre-development or the required basin wide release rate determined by MSD. Also, if possible, the present 100-year storm outflow rate should be reduced to the pre-development or required release rate. If this is not possible, then the resulting maximum outflow attained by the structure modification should be reduced to the existing structure's maximum outflow, or reduced as much as possible with one foot of freeboard (Whipple, Randall, 1983).

Given these objectives and the detailed SWMM models, optimal modifications of the existing outlet structures were designed for these basins (Walesh, 1989). The detailed SWMM models of six basins were calibrated with 5-minute incremental precipitation and basin depth of water and then used to compute the effects of the modifications. This chapter presents these calibrated models and shows the results of these modifications, including general recommendations for modifying all basins designed under the old detention basin criteria.

16.1 Gauging Stations

Gauging stations were used to measure precipitation and water depths at detention basins during storm events for the calibration of the SWMM models. Six recording stations, assembled at the University of Missouri-Rolla (UMR), were composed of a Stevens tipping bucket rain gage, differential pressure transducer, and a two-channel digital data logger. The gauging stations were fastened to auguring ground anchors located close to the detention basin outfall structures, keeping the digital logger case above the high water levels. The pressure transducers were mounted approximately five to ten feet (1.5-3 m) upstream of the low-water pipes along the swale. Incremental precipitation and depth measurements were taken at five-minute intervals during a number of storm events. The following six drainage systems in the MSD's district where the gauging stations were installed were:

- Sunset Ford Car Dealership
- Windemere East Sub-Division
- Telegraph Crossing Shopping Center
- Dartmouth Sub-Division
- West (A) Basin (see Figure 16.1)
- Center (B) Basin
- East (C) Basin

Figure 16.1 Dartmouth Sub-Division - West (A) Basin.

16.2 Model Calibration

Gauge data from the drainage systems were examined and at least three separate and distinct storm events for each drainage system were selected for use in model calibration. The incremental precipitation data for each event were inputted into the SWMM models of each drainage system. The computed basin stage data were then compared to the observed basin stage data. The calibration steps for each of the drainage systems are documented below and the plots of the observed versus computed stage for each of the drainage systems for each calibration step were plotted. The values for each parameter were only adjusted within sensible values based on the physical information of the watershed when attempting to match simulated to observed hydrographs.

16.2.1 Sunset Ford Drainage System

The dates of three separate storm events used in the calibration of the Sunset model were 6-8-98, 6-11-98, and 7-30-98.

The first calibration step taken in the Sunset model was the re-evaluation of the subcatchment and infiltration parameters using the same aerial-photos used in the original evaluation. Subcatchment estimates of area, percentage of impervious area, width of overland flow, and slope were checked. The slope of the runoff yielding subcatchment from the sales and service buildings' roofs was re-evaluated because it was underestimated due to probable direct connection of roof drains to the storm sewer. To account for the possibility of roof drainage entering the pipe network directly, the time to peak of the subcatchment runoff needed to be reduced and subcatchment runoff needed to be increased because roof drainage entering the pipe network directly results in decreasing runoff travel time and increasing subcatchment runoff. This connection could not be determined in more detail because of a lack of information on the roof drainage design. Therefore, these subcatchment slopes were increased based on a roof/ground area-weighted-slope approximation. This weighted-slope approximation method uses the percentage of a subcatchment containing roof area vs. percentage containing ground area, and is similar to the area weighted curve number estimate method. An average roof slope of 1:2 (Horizional: Vertical) was determined. Horton infiltration parameters were re-estimated using the Soil Conservation Service's soil maps for St. Louis County along with the general Horton infiltration information in the SWMM manual. The soil type within the Sunset Ford drainage basin was labeled as 16C or Urban Land, Harvester (Unified Soil type CL). This soil belongs to hydrologic group B, yielding an average asymptotic or minimum infiltration rate (F_c) of 0.23 in (5.8 mm) per hour. An average maximum or initial infiltration rate (F_0) of 0.33 in (8.4 mm) per hour was used for a partially drained clayey soil. A program default value of 0.00115 sec-1 is used for the decay rate of infiltration. It was assumed that the impervious and pervious depression storage values were set to zero to account for the wetted condition of the watershed before a major rainfall event.

The second calibration step taken in the Sunset model was the re-evaluation, adjusted within sensible values, of the offsite runoff contribution and the detention basin outlet structure's entrance loss coefficient (estimated to be 0.5). Originally, the Sunset model's offsite area contribution was delineated using an aerial photograph of the surrounding region. A site investigation of the offsite drainage system led to a better delineation of the offsite contribution area.

The third calibration step was the re-evaluation of the detention basin storage characteristics and the input of a detention basin outflow-rating curve. A quick survey performed on the detention basin showed a stage/volume relationship similar to that of the design plans. A new stage/volume estimate was not necessary. Model complications (transients, uncharacteristic variations in flow and stage) arose during the SWMM weir/open channel to orifice flow transition computations. The transients also increased the program execution time through convergence problems at the storage node. Because of these program instabilities at the outfall structure, a rating curve estimating weir and low-water pipe flow was entered into the model. Assuming 'inlet control' conditions, the rating curve for the outfall structure predicted normal flow conditions for the low-water pipe open channel flow confirming 'inlet control' conditions. The energy equation was then used to determine the corresponding headwater. Once the pipe was submerged, orifice flow conditions were assumed. In the orifice equation, a coefficient of discharge of 0.81 (projected pipe, with no separation) was used, based on published values. Even though the coefficient of discharge for the low-water pipe is bounded at the bottom instead of being completely projected, the projected pipe coefficient estimate is adequate because it will lead to a lower outflow estimate. The lower outflow estimate will account for debris that may exist at the entrance of the pipe. The weir overflow coefficient of discharge was estimated using the well-known sharp crested weir formula of Rehbock (Chow, 1959). Sharp crested weir properties as contrasted to the broad crested weir were used here because the length of flow is negligible and an increase in loss due to length would not be significant.

The fourth and final calibration step was to adjust the model routing parameters and simulation tolerances. Also, the SWMM weir flow computation was again utilized since it was observed that the model complications that arose during the SWMM weir to orifice flow transition were not serious, and because the rating curve accounted only for the low-water pipe outflow. These changes in the routing parameters involved specification of a head loss coefficient due to expansion/contraction turbulence from one conduit to the next. The coefficients were set at 0.5 and 0.3, respectively.

16.2.2 Windemere Drainage System

The dates of three separate storm events used in the calibration of the Windemere model were 6-11-98, 7-22-98, and 7-30-98. The 7-30-98 event was the largest event and was only used in the fourth calibration trial because of the difficulty in calibrating to low level storm events. Also, the 7-30-98 event was comparable in frequency to MSD's specified design criteria; therefore, it was used to determine if further calibration was necessary.

The first calibration step taken in the Windemere model was the re-evaluation of the infiltration input. The soil type within the Windemere drainage basin was found to be of the same type as in the Sunset Ford model. Therefore, the Horton infiltration parameters used in the Sunset model were applied to the Windemere model. Also as before, the impervious and pervious depression storage values were set equal to zero to account for the wetted condition of the watershed before a major rainfall event.

The second calibration step taken in the Windemere model was the re-evaluation of the areas directly contributing runoff to the detention basin. Two subcatchments were added to the catchment data that directly contribute runoff into the detention basin. Also, to increase retention and reduce outflow the detention basin outlet structure's entrance loss coefficient was set to 0.5, which is within sensible values based on the physical information.

The third calibration step dealt with changes to both runoff and routing parameters. The routing parameter changes involved setting the head loss coefficient and a contraction loss coefficient to 0.5 and 0.32 to account for the energy losses throughout the pipe network. The entrance loss coefficient for the detention basin was increased to 0.78 (for a projected pipe inlet), again, to increase the basin retention and reduce outflow. The detention basin stage/area (volume) relationship was re-evaluated showing a drastic difference in basin storage characteristics justifying the use of a new stage/area relationship. The runoff parameter modification involved reducing the time step to 60 seconds in an effort to improve model continuity. The Manning's roughness coefficient (n) for pervious area was increased from 0.03 to 0.25, which is more reasonable for grassed sheet flow and the Horton initial abstraction rate (F_0) was increased from 0.33 to 0.5 inches (8.4-12.7 mm) per hour in an attempt to reduce immediate runoff. The immediate runoff needed to be reduced to compensate for the change in the basin storage relationship. Finally, some general input errors in pipe slope prior to the basin were corrected.

After examining the observed versus computed basin stage hydrographs of the third calibration step, questions of why a fourth calibration step was necessary may arise. Because of program instabilities caused by severe transients at the outfall structure, a rating curve estimating low-water pipe flow was inputted into the model. Assuming 'inlet control' conditions, the rating curve for the outfall structure was based on normal flow conditions during low-water pipe open channel flow. The energy equation was used to determine the corresponding headwater. Once the pipe was submerged, orifice flow conditions were assumed. In the orifice equation, a coefficient of discharge of 0.81

(projected pipe with minimal flow separation) was estimated. In the end, the Horton initial infiltration rate (F_0) was set back to its original estimate of 0.33 in/hr (8.4 mm/hr) leading to better computed versus observed basin detention comparisons.

16.2.3 Telegraph Crossing Drainage System

The original Telegraph model was re-developed after three failed calibration attempts. It was found that several new commercial developments were tied into the Telegraph drainage system. These developments were then added to the model leading to a more accurate estimate of the true drainage area. The dates of three separate storm events used in the calibration of the Telegraph Crossing model were 6-14-98, 6-29-98, and 7-11-98.

The first calibration step taken in the Telegraph model was the re-evaluation of the infiltration input. The soil type within the Telegraph drainage basin was found to be the same as in the Sunset Ford and Windemere drainage systems. So, the same Horton infiltration parameters used in the Sunset/ Windemere models were applied to the Telegraph Crossing model. The impervious and pervious depression storage values were set equal to zero to account for the usually wetted condition of the watershed before a major rainfall event. Also, the originally modeled overflow structure, modeled as an orifice, was re-modeled as a weir and orifice because of program computation problems with modeling orifices.

The second calibration step taken in the Telegraph Crossing model included re-evaluation of the areas contributing runoff directly to the basin, as well as the detention basin stage-area (volume) relationship. The impervious area estimates, originally underestimated, were increased. Also, modifying the routing parameters, a head loss coefficient of 0.5 was set for the drainage system to account for the energy losses throughout the pipe network.

The third stage calibration of the Telegraph Crossing model included re-evaluating the detention basin storage, routing parameters, and runoff parameters. The basin stage-area (volume) relationship was changed after a survey was performed showing a drastic difference between the design plans and the constructed detention basin. The drainage area contributing direct runoff to the detention basin was also re-estimated during this survey. The routing parameters were changed by re-setting the entrance loss for the low-water outflow pipe of the detention basin to 0.78 and by setting the head loss coefficient at 0.32 for the entire drainage system, increasing basin retention and increasing peak basin inflow, respectively. In order to reduce the execution time and increase model stability, a rating curve was used to calculate the outflow through the low-water box in the Boston Market detention basin, a latter developed area tied into the Telegraph Crossing drainage system. Manning's n was increased for the pervious areas to 0.25 (increase the time to peak of catchment runoff) and the time step in the model was reduced to 60 seconds.

16.2.4 Dartmouth West (A) Drainage System

The dates of three separate storm events used in the calibration of the Dartmouth West model were 6-8-98, 6-21-98, and 7-30-98.

The first step toward calibration of the Dartmouth West model was the re-evaluation of the infiltration input, runoff parameters, and offsite area adjustments. The soil type within the Dartmouth West drainage basin was found to be the same as in the previously studied basins, so the same Horton infiltration parameters were applied. The impervious and pervious depression storage values were set equal to zero to account for the usually wetted condition of the watershed before a major rainfall event. Manning's n for pervious area was reduced to a more realistic value for short grass of 0.2. Finally, The offsite areas contributing runoff were re-estimated using contoured aerial photographs.

The second step toward calibration of the Dartmouth West model was the re-evaluation of the routing parameters and simulation tolerances. The routing method was changed from the Kinematic Wave to the Dynamic Wave method to better model any backwater effects that may occur. Also, a head loss coefficient of 0.5 was set for the drainage system to account for the energy losses throughout the pipe network. Results are presented in Figure 16.2.

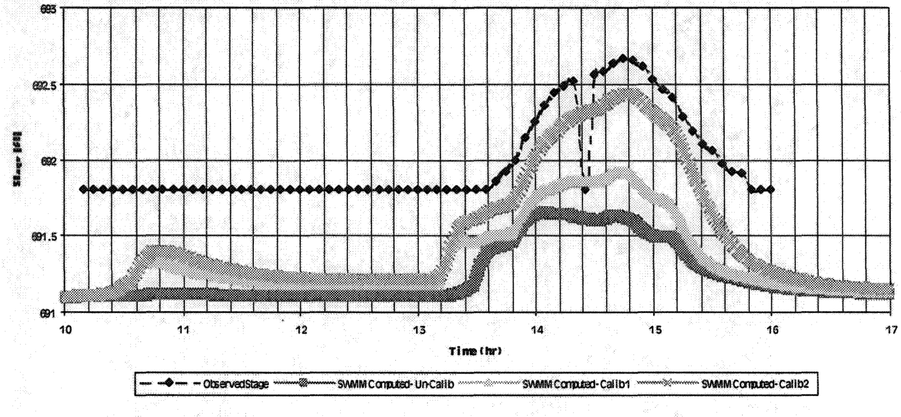

Figure 16.2 Dartmouth West Basin stage comparison of observed vs computed rain event 6/8/98.

16.2.5 Dartmouth East (C) Drainage System

Problems with the stage measurements during later events created large discrepancies between the observed and computed stages in this system. Because of the similarities in the design of the Dartmouth East and West detention basins, as well as the similarities in observed data from the two, the Dartmouth East computed stages, plotted against its actual observed stage, were calibrated and compared to the Dartmouth West storm event data. The dates of the three separate storm events used in the calibration of the Dartmouth West were 6-21-98, 7-30-98, and 8-4-98.

The first calibration step taken in the Dartmouth east model was to re-evaluate the infiltration input, runoff parameters, and offsite area adjustments. The soil type within the Dartmouth East drainage basin was found to be the same as the other drainage systems, so the same Horton infiltration parameters were used. Manning's n was reduced to 0.2. The offsite areas also contributing runoff were re-estimated with the use of contoured aerial photographs.

The second step in the calibration process was to correct errors and adjust routing parameters. The detention basin storage characteristics were delineated again, digitized, and changed accordingly in the model. Conduit invert errors just upstream of the detention basin were corrected and offsite areas contributing runoff to the basin were re-estimated. Also, a default head loss coefficient of 0.5 was set for the drainage system to account for the energy losses throughout the pipe network.

The third measure taken to calibrate the Dartmouth East model was to re-evaluate the infiltration parameters and subcatchment parameters of catchments with large quantities of runoff. To improve the comparison between the computed and observed data for the storm event on 6-21-98 (the best recorded single storm occurrence), subcatchments with large quantities of runoff were checked for errors and none were found. In an attempt to better fit the computed hydrograph shape to the observed for the several storm events, the Manning's n for pervious areas was reduced to 0.075 while increasing the Horton initial abstraction parameter (F_o) to 1.0 in/hr and decreasing the Horton infiltration equation's rate of decay to 0.00069 sec^{-1}.

16.2.6 Dartmouth Center (B) Basin

After sorting through the gauge data of the center basin, it was noticed that there was no measurable water detention over the duration of the gauging. Due to the similarities in the three Dartmouth drainage systems, calibration similarities between the west and east basins were applied to the center basin. In the event of conflicting parameters, the higher basin inflow value was selected.

The calibrations performed on the center basin included correcting original model errors and re-evaluating the infiltration and routing parameters. Invert errors were fixed in the conduits just upstream of the detention basin, more accurately approximating the storage characteristics of the detention basin. The same Horton infiltration parameters were used. The routing parameters were adjusted so that a head loss coefficient of 0.5 was set for the drainage system to account for the energy losses throughout the pipe network.

16.3 Outlet Structure Modifications

After each drainage system model was developed, the proposed modifications (low-flow /low-water pipe (LFP) and/or overflow weir) were incorporated into the models. In the case where numerical instability occurred with the overflow weir, a user-specified rating curve was used. The rating curves for the modified outlet structures were made in the same manner as the present condition rating curves. The modifications for the detention basin outlet structures are listed in Table 16.1.

Table 16.1 Modifications for the detention basin outlet structures.

Drainage System	Description of Modification
Dartmouth West (A)	42" to 15" LFP; 2'x 8' orifice; Raise 4 walls, 2'
Dartmouth Center (B)	18" to 6" LFP
Dartmouth East (C)	30" to 6" LFP; Raise 3 walls 2 ft
Sunset Ford	2 - 33" to 2 - 8" LFP
Telegraph Crossing	15" to 6" LFP
Windemere East	27" to 6" LFP

The comparison plots of the computed stage and outflows for the present and modified conditions for each of the calibrated events, as well as for the mandated design storm events, were performed. The magnitudes of the 2-year outflows are compared in Table 16.2, and those of the 100-year outflows in Table 16.3.

In most cases, a peak flow reduction, which is below the specified release rates, is estimated for both recurrence intervals. During the 2-year event, the only circumstances where the release rate is being exceeded occurred in the Dartmouth systems. The Dartmouth modifications significantly reduce the peak flows and reasonably contain the 100-year storm event to acceptable flow rates below the pre-developed flow rate and, with the exception of the center basin, keep flow rates below the specified release rate. Since the impact from

Table 16.2 Magnitudes of the 2-year outflows.

Drainage System	2 - Year Pre-developed Flow Rate (SCS Estimated) (cfs)	Specified Release Rate (cfs)	2 - Year Existing Basin Outflow (SWMM Computed) (cfs)	2 - Year Modified Basin Outflow (SWMM Computed) (cfs)	Flow Difference: Existing - Modified (cfs)
Dartmouth West (A)	50.03	14	100.65	65.72	34.93
Dartmouth Center (B)	5.61	2	15.28	10.83	4.45
Dartmouth East (C)	17.56	8	54.45	28.40	26.05
Sunset Ford	30.90	30.90	25.48	31.96 *	-6.48
Telegraph Crossing	21.57	21.57	16.63	17.13	-0.5
Windemere East	21.75	21.75	42.29	24.72	17.57

* Modified flow calculated with proposed lot addition.

Table 16.3 Magnitudes of the 100-year outflows.

Drainage System	100 - Year Undeveloped Flow Rate (SCS Estimated) (cfs)	Specified Release Rate (cfs)	100 - Year Existing Basin Outflow (SWMM Computed) (cfs)	100 - Year Modified Basin Outflow (SWMM Computed) (cfs)	Flow Difference: Existing - Modific'n (cfs)
Dartmouth West (A)	242.97	94	197.84	80.38	117.46
Dartmouth Center (B)	28.18	12	39.86	18.79	21.07
Dartmouth East (C)	94.51	50	78.81	38.69	40.12
Sunset Ford	143.64	143.64	147.08	158.40 *	-11.32
Telegraph Crossing	106.82	106.82	43.74	40.35	3.39
Windemere East	112.64	112.64	75.70	91.88	-16.18

* Modified flow calculated with proposed lot addition.

the 2-year events for downstream flooding is not as severe as it would be for the 100-year event, further system modifications would not reduce the overall flood risk.

An increase in peak outflow is predicted for the Windemere East system (100-year event), Telegraph Crossing system (2-year event), and the Sunset Ford system (100-year event). The increase in flow in the Telegraph Crossing and Windemere East systems is acceptable because both peak outflows are below the specified release rates. This is not the case with the Sunset Ford system.

The original design of the Sunset Ford system specified that the outfall structure be composed of two 33-inch (840 mm) low-water pipes, and an overflow weir 4 feet (1.2 m) above the inverts of the low-water pipes. A property owner downstream of the detention basin, having flooding problems, partially blocked the two outlet pipes with some wood sheets and posts, and caused a reduction of the individual pipe area to approximately 20% of its original area. This is how the existing basin outflow conditions are modeled. It must therefore be realized that there is an overall reduction in basin outflow with respect to the original design. Although the specified release rate is exceeded during the 100-year event, the modification of the low-water pipe is still justified and should help with the severity of downstream flooding. The overflow weir controls the majority of peak flows. Since the maximum basin stage is only 6 feet (1.8 m) above the invert of the low-water pipes, there is not much room to safely raise the weir elevation. The optimum solution to this problem would be to modify the basin storage capacity (raise the dam), but this will require further analysis.

16.4 SCS Pre- and Post-Development Model Calibration

Computations using the original SCS model, which is MSD's prescribed method for the design of detention basin outlet structures, for post-developed basin inflow did not compare well to the calibrated SWMM model estimations. It was felt that improved results would be seen with the pre-development flow estimate with calibration of the SCS post-development basin inflow models. Changes to the post-development models could then be applied to the pre-development model for re-assessment. Table 16.4 shows the results of the post-development peak basin inflow rates for both recurrence intervals obtained by SWMM and SCS Methods.

The adjustments made to the SCS model dealt with the time of concentration (t_c) and curve number (CN). The peak flow values do not completely match up because SWMM has many more parameters to calibrate. Also, the SWMM models are highly detailed, whereas the SCS models use simple catchments that generalize the basin runoff properties.

While calibrating the SCS models, it was noticed that the post-development calibrated model parameters differed from those previously estimated. Table 16.5 shows the time of concentration (t_{c}.) calculated using several different estimation methods.

Table 16.4 Post-development peak basin inflow rates.

Drainage System	2 -Year Basin Inflow Rate (SWMM Estimated) (cfs)	2 -Year Basin Inflow Rate (SCS Estimated) (cfs)	100 -Year Basin Inflow Rate (SWMM Estimated) (cfs)	100 -Year Basin Inflow Rate (SCS Estimated) (cfs)
Dartmouth West (A)	124.82	92.40	236.93	311.60
Dartmouth Center (B)	17.18	12.31	44.25	39.97
Dartmouth East (C)	62.98	33.14	107.45	129.72
Sunset Ford	71.39	91.35	206.67	190.75
Telegraph Crossing	74.68	72.44	122.92	174.69
Windemere East	62.86	59.70	177.95	114.68

Table 16.5 Post-developed time of concentration.

Drainage System	2-year SWMM Estimate (min)	100-year SWMM Estimate (min)	TR-55 Method (min)	Kirby-Hathaway and Kirpich Method (min)	Calibrated t_c Used In the SCS Model (min)
Dartmouth West (A)	43.13	29.13	7.96	14.93	12
Dartmouth Center (B)	15.83	11.24	6.05	7.34	6
Dartmouth East (C)	34.60	24.39	36.56	22.68	20
Sunset Ford	36.39	25.39	24.14	19.56	20
Telegraph Crossing	15.90	13.20	6.57	17.15	17
Windemere East	26.12	18.65	20.45	16.20	18

For the overland flow section, the SWMM time of concentration (t_c) estimates involved average velocity approximations for the hydraulically most remote catchment (node) using Manning's equation and assuming sheet flow. The sheet flow depth for half of the peak flow (assumed to be average flow for the catchment) was back-calculated based on SWMM estimates and catchment input information. An average velocity was then calculated using half of the peak flow, sheet flow depth, and a catchment width. Knowing the distance of overland flow, the time of concentration (t_c) was estimated.

For the pipe network, the SWMM time of concentration (t_c) estimates were determined by summing the time it would take for water to flow through each individual pipe at the average of the SWMM estimated velocities. It was assumed that the average velocity would be about half of the peak velocity.

After calibrating the post-development SCS models, the pre-development models were re-evaluated. Shown in Table 16.5, the calibrated post-developed time of concentration (t_c) usually fell within the range of the TR-55 and Kirby-Hathaway/Kirpich methods. The one exception occurred in the Dartmouth East system where the time of concentration (t_c) was below the range determined using the TR-55 and Kirby-Hathaway/Kirpich methods. Using this information, the pre-development time of concentration (t_c) estimates were made. An approximate average between the TR-55 method and the Kirby-Hathaway/Kirpich method was used for the pre-development model. The exception occurred in the Dartmouth West system, where the Kirby-Hathaway/Kirpich method was used, since the TR-55 method fell below the post-development time of concentration (t_c) estimate. Table 16.6 shows the pre-development time of concentration (t_c) estimates using the various methods.

Table 16.6 Pre-developed time of concentration.

Drainage System	TR-55 Method (min)	Kirby-Hathaway and Kirpich Method (min)	Calibrated t_c Used In the SCS Model (min)
Dartmouth West (A)	10.71	15.69	16
Dartmouth Center (B)	27.60	16.41	21
Dartmouth East (C)	39.00	23.42	30
Sunset Ford	28.80	22.07	26
Telegraph Crossing	25.16	25.16	25
Windemere East	35.56	19.71	28

The difference in results between the SWMM post-developed time of concentration (t_c) and the other methods needs to be addressed. For instance, the difference between the post-developed 2- and 100-year SWMM results (Table 16.5) shows how much the precipitation intensity or depth of flow influences the time of concentration (t_c). Also, noting the difficulty in accurately calibrating small watersheds with simple single-catchment SCS models (Table

16.4), the difference in the number of adjustable parameters between the SWMM and the SCS models is quite significant. This is indicative of the inability of the SCS model to accurately compute flow rates for small urban watersheds for both of the required retention storm events. Other SCS parameters, such as shape factor of the unit hydrograph and the initial abstractions estimate, need to be further studied. A re-evaluation of all the SCS parameters might increase the chances for better peak flow estimates for a range of recurrence intervals.

16.5 Recommendations, Modifications and Conclusions

The objective of modifying the existing outlet structures to reduce the 2-year and the 100-year storm outflows to the pre-developed or the basin-wide release rate was met in most cases. Also, the modifications result in increased use of basin storage for the more frequent storm events. However, it was not possible to reduce the event outflows of Dartmouth drainage systems to the basin wide release rates determined by MSD. Doing so would require costly storage capacity adjustments.

This analysis and modification process has led to the conclusion that there is no simple versatile modification procedure that can be applied to other problematic detention basin systems. Modifications need to be made on a basin-by-basin basis. As seen from the original SWMM pre-calibrated model estimates, the original models of all the systems underestimated basin detention (stage vs. time). One reason for this is that some of the drainage systems were not constructed according to their design plans. Another reason involves the difficulty of delineating the precise areas contributing (i.e. new-development pipe network tie-ins).

A regional development through calibration of all the SCS parameters will increase the chances of computing better peak flow estimates for a range of recurrence intervals. Generally, calibration with observed data is the best means of creating an accurate model including the proposed modifications to the outlet structures. This work indicates that even a sophisticated, detailed SWMM model can be improved by calibration. The simpler SCS model can also be improved by calibration. Each calibration parameter should only be adjusted within sensible values based on the physical information of the watershed. More sophisticated, detailed SWMM models should produce, on the average, estimates closer to the measured storm responses than the simpler, SCS model with or without calibration.

References

Akan, A.O. "Detention Pond Sizing for Multiple Return Periods, " J. Hydraulic Engineering, Vol. 115, No. 5, May 1989, pp. 650-664.

McEnroe, B.M. "Preliminary Sizing of Detention Reservoirs to Reduce Peak Discharges," J. Hydraulic Engineering, Vol. 118, No. 11, November 1992, pp. 1540-1549.

Walesh, S.G. Urban Surface Water Management. New York: Wiley-Interscience, 1989.

Whipple W., and Randall, S.W. "Detention and Flow Retardation Devices," Stormwater Management in Urbanizing Areas, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1983.

Bibliography

Akan, O.A. and Paine, J.N., "Optimal Sizing of Detention Basins," Water Resources Engineering, ASCE, Vol. 2, 1995, pp. 1249-1253.

Basha, H.A. "Nonlinear Reservoir Routing: Particular Analytical Solution," J. Hydraulic Engineering, Vol. 120, No.5, May 1994, pp. 624-632.

Basha, H.A. "Routing Equations for Detention Reservoirs," J. Hydraulic Engineering, Vol. 121, No. 12, December 1995, pp. 885-888.

Fiuzat, A.A. "Design Criteria for Detention Basin in Urban Drainage." Engineering Hydrology: Proceeding of the Symposium, ASCE, 1987, pp. 370-375.

Froehlich, D.C. and Kowalski T.R., "Graphical Sizing of Small Single-Outlet Detention Basins," Water Resources Engineering, ASCE, Vol. 2, 1995, pp. 1071-1075.

Metropolitan St. Louis Sewer District. Rules and Regulations and Engineering Design Requirements for Sanitary Sewerage and Stormwater Drainage Facilities. February 1997.

Pazwash, H. "Urban Stormwater Management Practices; Concepts and Misconceptions." Hydraulic Engineering: Proceeding of the 1990 National Conference, Vol. 1, 1990, pp. 506-511.

Smith, D. P., Jr., and Bedient, P.B., "Detention Storage for Urban Flood Control," J. Water Resources Planning and Management Div., Vol. 106, No. 2, July 1980, pp. 413-425.

"XP-SWMM, Stormwater and Wastewater Management Model," Version 6.1, XP software, Tampa, Florida, 1999.

Yanmaz, A.M. "Reliability Assessment in Detention Basin Design." Water Resource Engineering 98: Proceedings of the International Water Resources Engineering Conference, ASCE, 1998, pp. 820-825.