THESIS

MODELING THE EFFECT OF STORMWATER CONTROLS ON SEDIMENT TRANSPORT IN AN URBAN STREAM

Submitted by

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In partial fulfillment of the requirements

For the Degree of Master of Science

Colorado State University

Fort Collins, Colorado

Fall 2004

COLORADO STATE UNIVERISTY

September 8, 2004

WE HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER OUR SUPERVISION BY CHRISTINE A. ROHRER ENTITLED "MODELING THE EFFECT OF STORMWATER CONTROLS ON SEDIMENT TRANSPORT IN AN URBAN STREAM" BE ACCEPTED AS FULFILLING IN PART REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE.

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ABSTRACT OF THESIS

MODELING THE EFFECT OF STORMWATER CONTROLS ON SEDIMENT TRANSPORT IN AN URBAN STREAM

Urbanization of a watershed increases impervious area, and consequently increases stormwater runoff. When left uncontrolled, these increases in stormwater runoff cause downstream flooding, accelerate channel erosion, and impair aquatic habitat. Increases in the magnitude and duration of stormwater runoff that accompany uncontrolled development allow a stream to carry more sediment than it could prior to watershed development. When a watershed cannot supply the stream with the sediment it has the capacity to carry, channel degradation may occur in the form of incision, lateral adjustment or a combination of both.

This study evaluates the sensitivity of sediment transport to runoff from typical residential development in a prototype headwater stream. Event-based and continuous simulations were conducted with the USEPA Stormwater Management Model (SWMM), using rainfall records for two climatically different locales, the semiarid climate of Fort Collins, Colorado and the other in a typical, wetter, southeastern climate, Atlanta, Georgia. Five conditions were evaluated for the study watershed, including: existing (partially developed) conditions; fully developed conditions, without stormwater controls; and fully developed conditions with stormwater controlled using (a) the City of

Fort Collins flood control standard, (b) the City of Fort Collins flood control standard and water quality capture volume (WQCV) criteria, and, (c) using common standards of practice in the United States: control of the 100- and two-year storms to historic peak discharge rates and control of the WQCV. For each scenario examined, sediment transport potential is evaluated for two noncohesive soil types: medium gravel and medium sand. Results indicate that precipitation patterns and bed material both play a large role in the type of stormwater controls most appropriate to minimize increases in sediment transport potential due to urbanization.

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ACKNOWLEDGEMENTS

I am very grateful for the support and guidance provided by Dr. Larry Roesner, my advisor. Thank you, Larry, for your patience as I formulated my ideas for this project and implemented them. I also appreciate the input provided by the other members of my graduate committee, Dr. Brian Bledsoe and Dr. Ellen Wohl.

I gratefully acknowledge Harold H. Short and Camp Dresser and McKee for funding my research. Their generosity allowed me to bring my project idea to life.

Thank you to Kevin McBride and Matthew Fater with the City of Fort Collins Stormwater Division. The data they provided gave me a head start on the SWMM model development. I would also like to recognize Brannan Davis, who as a Research Experience for Undergraduates summer intern helped me develop the SWMM models and Jennifer Davis who assisted with SAS plots.

Thank you to Joe Rathbun of the Michigan Department of Environmental Quality for his support and encouragement as I returned to academia. I also appreciate the input he has given on my research, and his willingness to answer questions night and day.

A special thanks to all of my fellow graduate students in the HHS Lab who have encouraged me over the years: Melanie Criswell, John Edgerly, Matthew Garcia, Jorge Gironas, Christine Marjoram, Seth Nerhke, Brian Younkin, and especially Rajesh Rajan, who is always eager to lend a hand. I am lucky to be surrounded by such wonderful people in the workplace. Thank you, also, to a good friend and kindred spirit, Jamie Gaskill-Fox, for her listening ear as I completed my work, and for her review of my final manuscript

I thank my family for their unwavering love and assistance throughout my life. Mom, Dad, and Cindy, you’ve supported every step I’ve taken to get to this point. Finally, and most importantly, I thank my partner in life and best friend, Brian Kraus. Without his support over the past years, my path to completion would have been much more difficult. His encouragement gives me strength to pursue my biggest dreams.

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1.0 INTRODUCTION

Urbanization of a watershed increases impervious area and consequently increases stormwater runoff. When left uncontrolled, these increases in stormwater runoff have the potential to cause downstream flooding, accelerate channel erosion and impair aquatic habitat. Increases in the magnitude and duration of stormwater runoff that accompany uncontrolled development allow a stream to carry more sediment than it could prior to watershed development. When a watershed cannot supply the stream with the volume of sediment it has the capacity to carry, channel degradation may occur in the form of incision, lateral migration or a combination of both (Roesner and Bledsoe, 2003).

Many municipalities and agencies require some level of stormwater control when development or redevelopment takes place within their jurisdiction. The intent of these practices ranges widely: from flood prevention to protection of water quality. Currently, few municipalities require design of stormwater controls specifically to minimize stream channel degradation.

In this study, event-based and continuous stormwater modeling were used to examine the effects of urbanization and current stormwater practices on flow frequency, flow duration, and sediment transport in a receiving stream. Two climate regions were examined: Fort Collins, Colorado and Atlanta, Georgia because the frequency, intensity, and duration of precipitation between these climates varies greatly, and generates different receiving water responses. In the semiarid climate of Fort Collins, Colorado, precipitation is variable, and often occurs at high intensities, for a limited duration. The resulting flow regime is flashy, and typically includes a sharply peaked runoff hydrograph. The humid, temperate climate of Atlanta, Georgia has frequent precipitation of variable intensity and long duration that produces runoff hydrographs of lower amplitude (Knighton, 1998). In both climates, after urbanization takes place, runoff rates respond much more directly to rainfall intensities, both in time, magnitude, and duration. (Roesner and Bledsoe, 2003).

The varied magnitudes, frequencies and durations of stormwater runoff produced by different climates result in differences in the sediment transport regime, making climate an important factor in deciding which stormwater controls are most effective for minimizing channel erosion. Likewise, streambed and bank materials are also important factors in determining which stormwater practices minimize erosion potential. In this study, the sensitivity of sediment transport to flood and water quality stormwater controls was evaluated for two types of bed material: sand bed ($d_s > 0.25$ mm) and gravel bed ($d_s > 8$ mm).

Chapter Two of this document presents a review of relevant literature. In Chapter Three, the study area is described, and an explanation of the methodologies used for analysis is presented. In Chapter Four, the results of the analysis are presented and interpreted. Chapter Five includes conclusions and recommendations for future research.

2.0 LITERATURE REVIEW

2.1 Effects of Urbanization on the Flow Regime

The hydrologic effects of urbanization are well known by watershed engineers and scientists. When a watershed is developed, the land is covered with impervious surfaces such as roads, parking lots, roofs, driveways, and sidewalks. These impervious surfaces restrict the amount of water that is allowed to infiltrate into the ground and increase the amount of surface runoff, both of which work in combination to alter the natural flow regime of the system. Nine case studies compiled by the United States Environmental Protection Agency (1997) document the hydrological effects of urbanization, including: increases in bankfull events, increased flooding, increased peak flows, decreased baseflow, stream enlargement, stream incision, severe stream bank erosion, sedimentation, changes in morphology, increased instream sediment load, increased sediment transport, aesthetic degradation, degradation of designated uses, and loss of fish populations. Roesner and Bledsoe (2003) summarize the fundamental hydrologic changes that are associated with urbanization as: (1) more frequent and higher magnitude flows; (2) increased duration of geomorphically significant flows; (3) flashier/less predictable flows; (4) altered timing and rate of change relative to riparian and floodplain connections; (5) altered duration of low-flow periods; and (6) conversion of subsurface distributed discharge inputs to surface (point) discharge.

Peak-flow increases from two- to more than 50-fold typify the changes brought by urbanization (Hollis, 1975; Urbonas and Roesner, 1992; Roesner, et al., 2001). Hollis (1975) showed patterns of increasing change in peak discharge with increasing percentage of impervious area and decreasing storm magnitude. Using 50 years of hourly rainfall records, Nehrke and Roesner (2004) showed that flow exceedance frequencies increased dramatically when development of a watershed was left uncontrolled. Nardi and Roesner (2003) showed that flow durations for small discharges increased significantly when uncontrolled development took place, and that the addition of extended detention best management practices produced the greatest increase in the flow duration curve.

2.2 Effects of Urbanization on Channel Morphology

The association between water and sediment discharge, particle size, and channel bed slope can be described using the relationship described by Lane (1955):

$$QS \propto Q_s d_s \qquad \textbf{Equation 2-1}$$

where Q = water discharge, S = channel bed slope, Q_s = sediment discharge, and d_s = particle size. When urbanization occurs, the balance between sediment transport capacity and the amount of sediment delivered from a watershed is disrupted. As instream discharges increase due to urbanization of a watershed, the ability of a stream to transport sediment increases. For example, if Q represented in Equation 2-1 increases due to urbanization, and if S and d_s remain constant, Q_s will increase.

Both anecdotal evidence and field research support the notion that the larger and more frequent discharges that accompany watershed development cause downstream channels to enlarge (Wolman, 1967; Hammer, 1972; Leopold, 1973; Graf, 1975; Urbonas, 1980; Whipple et al., 1981; Neller, 1989; Booth, 1990; Caraco, 2000; Booth and Henshaw, 2001). Neller (1988, 1989) found that urban-affected streams were on average four times larger than adjacent rural streams. Urbonas (1980) described drainage way erosion in semi-arid urbanizing areas. Additional studies suggest that urbanization can cause channels to expand two to five times their original size (Hammer, 1972; Moriwasa and LaFlure, 1979; Allen and Narramore, 1985; Booth, 1990). Booth and Henshaw (2001) studied rates of channel change in light to moderately urbanized watersheds in humid regions, and showed that rates of channel change did not correlate with development intensity, but rather that the nature of the geologic substrate strongly influenced whether or not significant channel change occurred.

It is commonly believed that urbanization results in increased sediment yield from a watershed, however, the increased sediment yield frequently found in urbanizing watersheds likely comes from in-channel erosion, rather than upland erosion. Graf (1975) observed that large quantities of sediment introduced to a stream during construction and a subsequent increase in runoff due to development led to expansion of floodplains followed by downcutting of streams. In an urban stream in California, Trimble (1997) estimated that bank erosion accounted for an estimated two-thirds of the measured sediment load. In contrast, bank erosion in rural streams compromises only five to twenty percent of the annual sediment budget (Caraco, 2000).

Decreases of upland sediment supply after urbanization have also been documented (Wolman, 1967; Wolman and Schick 1967; Douglas, 1985). Wolman (1967) found that whereas sediment yield increased by a factor up to 200 during the construction phase of urbanization, it declined to pre-urbanization levels after construction was completed. Douglas (1985) demonstrated that a stable, urban environment will have a low to moderate sediment yield as compared to a predevelopment environment consisting of cropping, which supplies a moderate to heavy sediment yield, depending upon cropping management practices. Decreases in upland sediment supply combined with an increase in sediment transport capacity due to increased discharge magnitudes and frequencies can result in channel erosion.

The discharge most effective in the long-term transport of sediment and that forms at least some of the morphological characteristics of a stream is commonly referred to as effective discharge. Traditional geomorphic estimates (Wolman and Miller, 1960; Leopold et al., 1964) indicate that storms of a one- to two-year recurrence interval are responsible for the form of an active channel. More recent studies, however, challenge this widely held notion. Ashmore and Day (1988) and Nash (1994) showed that the observed recurrence interval of the effective discharge in North American streams is highly variable, ranging from a week to several decades.

2.3 Effects of Stormwater Control Practices on Channel Morphology

Many municipalities have development ordinances that require large storms to be controlled so that the postdevelopment peak discharge for a given return interval storm does not exceed the predevelopment peak discharge for that same storm. Control of this peak discharge is often referred to as peak shaving, and it is commonly achieved through the use of stormwater detention ponds. Peak discharge shaving requirements vary widely across the United States, with practices ranging from control of the 100-year, 25-year, 10-year, or two-year return interval storms, to including a combination of return interval storms. For example, the City of Lincoln, Nebraska (2000) requires that postdevelopment 10- and 100-year peak discharges be controlled to predevelopment rates. On the other hand, the City of Fort Collins, Colorado requires that the peak discharge from the 100-year postdevelopment storm be controlled to the predevelopment two-year peak runoff rate (City of Fort Collins – Water Utilities, 1997).

Experience with peak shaving detention shows that although it reduces downstream flooding, it is not effective for reducing erosion in stream channels. Studies have shown that peak shaving of the two-year return interval storm may actually exacerbate erosion (McCuen, 1979; Moglen and McCuen, 1988; MacRae, 1993 and 1997). Peak shaving of the 100- through two-year return interval storms is thought to be ineffective for erosion control for two reasons: (1) detention basins commonly subject stream channels to erosive flows for a longer duration and at increased frequencies, and (2) the outlets only attenuate the flow of the storm for which they are designed, allowing smaller, more frequent, storms to pass through unrestricted (Roesner et. al, 2001). Postdevelopment control of the two-year return interval storm typically releases water above the critical discharge for sediment transport for a longer period of time than in predevelopment conditions, which results in cumulative transport of more sediment. Whereas the magnitudes of peak discharges are unchanged from predevelopment to postdevelopment under two-year peak shaving, the duration and frequency of erosive flows increase dramatically. As a result of this increase, the effective discharge in the channel is shifted to smaller runoff events that range from the half-year event up to the 1.5-year runoff event (MacRae and Rowney, 1992). MacRae (1993, 1997) also documented that two-year control triggers channel expansion, causing widening by as much as three times the predevelopment condition.

Other options for peak shaving detention include variations of the over control method described for Fort Collins, Colorado. These options include controlling the postdevelopment peak discharge rate to a fraction (e.g., 50 percent) of the predevelopment level or restricting the two-year postdevelopment discharge rate to the one-year predevelopment rate using the 24-hour storm event (Center for Watershed Protection, 2004). Analysis by MacRae (1997) indicates that over control design criteria do not protect the stream channel from erosion, and that depending on the bed and bank material, the channel may either degrade or aggrade.

Recent emphasis on removal of pollutants in urban runoff has targeted control of high frequency events (e.g., those smaller than the two-year return interval). WEF and ASCE (1998) propose that appropriate stormwater quality is achieved when 70 percent of all runoff-producing events are fully captured and subjected to some form of treatment (or pollutant removal) such as surcharge storage above the permanent pool of a detention pond. The size of this surcharge storage is referred to as the water quality capture volume (WQCV). Although control of the WQCV allows the peak discharges of storms with a return interval of less than two-years to be reduced, increased durations of discharge from these basins and trapping of sediment may exacerbate channel erosion, especially in streams that are highly sensitive to changes in low flows, e.g. sand bed channels. Nehrke and Roesner (2004) showed that control of the WQCV in addition to peak shaving of the 100-year, 10-year and two-year flows provides an excellent match of the postdevelopment flow frequency curve to predevelopment conditions. The flow frequency curves generated by peak shaving alone provided less of a match.

McCuen and Moglen (1988) proposed an alternative detention basin design that limits the total bed-material load after development to that which existed prior to development. MacRae (1993) proposed a philosophy for control of instream erosion potential referred to as distributed runoff control. Distributed runoff control minimizes channel erosion by maintaining the erosion potential of the channel boundary materials to predevelopment conditions. Distributed runoff control requires field assessment to determine the hydraulic stress and erosion potential of bank materials. This method does not reproduce predevelopment flows in all aspects, but does reproduce the 1:2 year predevelopment hydrograph for flows above the erosion threshold defined for the channel.

In addition to watershed imperviousness, the sediment trapping efficiency of stormwater detention ponds may also decrease sediment supply to a stream. Without an adequate supply of sediment from upstream, increases in discharge magnitudes and duration can further exacerbate channel stability. If changes in sediment supply do occur, reproduction of the predevelopment flow regime does not necessarily maintain the stability of streams. Streambeds and banks that are comprised of sand are especially susceptible, since general physical principles indicate that sand-bed channels are more likely to shift in the sub-bankfull flow regime (Bledsoe, 2002). In contrast, gravel and cobble bed streams are generally less sensitive to changes in sediment load (Howard, 1980; Hey and Thorne 1986; Bledsoe, 2002). Recognition of changes in the magnitude and duration of flows, changes in sediment supply, and the potential response of different stream types are each important aspects of understanding channel erosion in urbanizing watersheds (Bledsoe, 2002; Bledsoe and Watson, 2001; MacRae, 1997).

3.0 STUDY APPROACH

This study evaluates sediment transport rates affected by two distinct precipitation patterns: those in Fort Collins, Colorado and those in Atlanta, Georgia. Hydrologic and hydraulic models were used to simulate stormwater runoff in a headwater stream in Fort Collins, Colorado. The same prototype watershed was used for both the Fort Collins and Atlanta analysis and the rainfall input was the only variable changed between the two locales. The runoff system and receiving stream were identical and the watershed hydrogeometric properties were held constant. Watershed state variables, such as Horton infiltration values, Manning n values, pervious and impervious storage values and evaporation rates, were also held constant. These variables were held constant so that the sensitivity of instream sediment transport to differences in climatically varied flow regimes could be examined without the influence of changes in the other variables. It is recognized, however, that under different climates and geographic regions, the hydrogeometric properties, state variables and receiving stream hydrogeomorphic characteristics are likely to differ.

The prototype watershed is located south of Fort Collins (shown in Figure 3-1), and is a subbasin within the Fossil Creek Watershed. Land use in the study area currently consists of 12.1 ha (30 acres) of low-density residential development in the form of estate lots and 9.9 ha (25 acres) of pastureland.

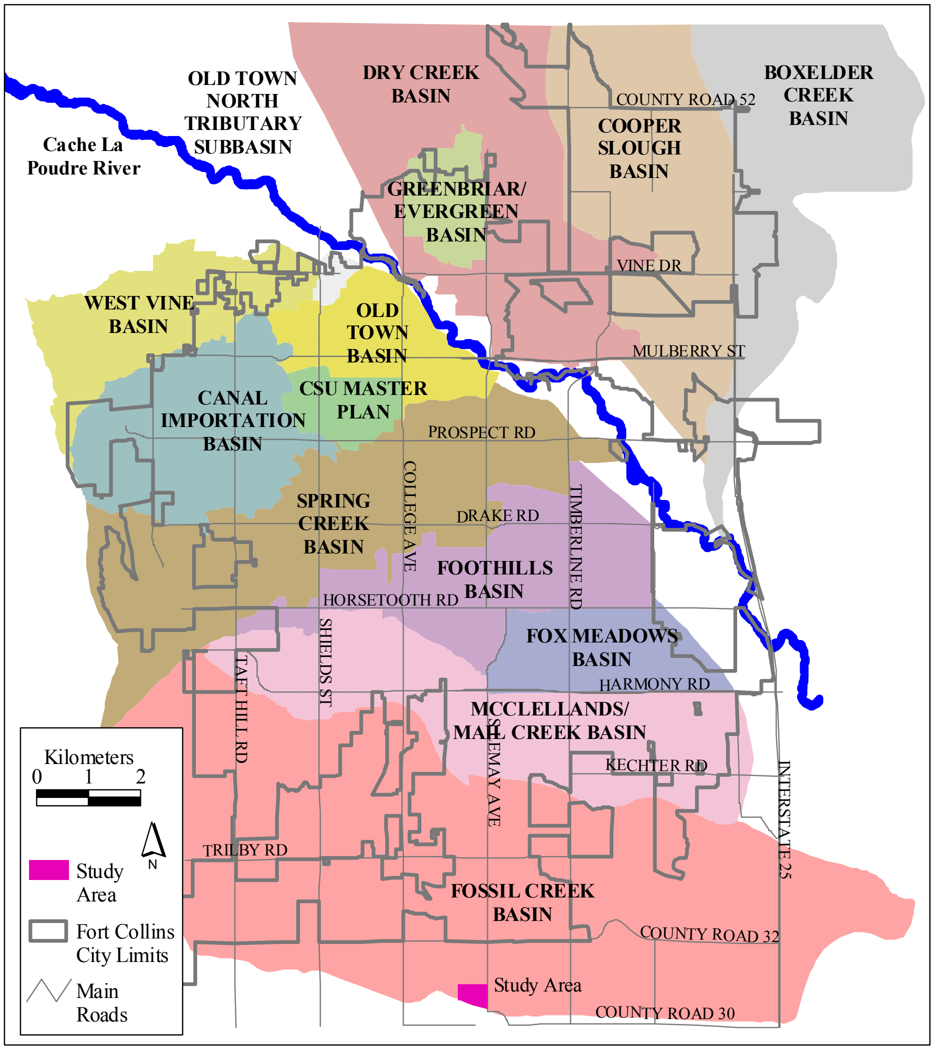

Figure 3-1. Location of study watershed.

Five scenarios were evaluated for the study watershed. These scenarios include existing conditions, fully developed conditions without stormwater controls, and fully developed conditions with stormwater controlled using three methods. The stormwater control methods examined use: (a) the City of Fort Collins flood control standard, (b) the City of Fort Collins flood control standard and water quality capture volume (WQCV) criteria, and (c) a common standard of practice in the United States: control of the 100- and two-year storms to historic peak discharge rates and control of the WQCV.

A six-step approach was employed for this study, primarily using programs contained in Version 44h of the USEPA Stormwater Management Model (SWMM) suite of tools. (1) Continuous precipitation data were processed by SWMM Rain for use in SWMM Runoff. (2) SWMM Runoff generated surface runoff based on local precipitation data (continuous and event-based), land use and topography. (3) SWMM Extran routed event-based flows through the drainage system and stormwater basins, creating stage-discharge relationships for use in SWMM Transport. (4) SWMM Transport routed continuous and event-based flows through the drainage system and stormwater basins. (5) The SWMM Statistics block generated flow frequency data from Transport. (6) SAS, a statistical analysis/programming package, was used to calculate sediment transport capacities, flow duration curves, average boundary shear stress and specific stream power.

3.1 Precipitation Data

Fifty years of continuous hourly precipitation data were used to simulate the impacts of development on flow frequency, flow duration, and sediment transport potential. Continuous precipitation data were obtained from the National Climatic Data Center (NCDC, 2003) for both Fort Collins and Atlanta. Hourly precipitation values from the Fort Collins, Colorado (NCDC COOP ID: 053005) and Atlanta Hartsfield International Airport (NCDC COOP ID: 090451) rain gauges were examined for a period of record ranging from January 1, 1951 to December 31, 2000. Rainmaster, a program used to process and analyze continuous precipitation data (Heineman, 2001), was used to calculate precipitation depths for various return intervals from the 50-year records.

A review of the continuous precipitation data for Fort Collins revealed an error in the data set on September 20, 1980. A value of 6.5 inches (165 mm) was recorded in hour 2400. NCDC data flags indicate that this value was a cumulative total over 26 days. A web search (Doeskin, 2003; Fort Collins, 2003) on the history of large Front Range events did not indicate that a large event took place on or near that date. To be consistent with work conducted with Fort Collins precipitation data in a previous study (Nehrke and Roesner, 2004), this value was changed to 0.65 inches (16.5 mm).

Event-based precipitation data, commonly referred to as design storms, were used to size stormwater conveyance facilities (e.g., gutters and swales) and to size the stormwater detention ponds. Design storm hyetographs for the Atlanta analysis were generated by applying Denver Urban Drainage and Flood Control District (UDFCD, 2001) distribution criteria to two- and 100- year, 1-hour total storm depths generated by the Rainmaster analysis. Design storm hyetographs for Fort Collins were provided by the City of Fort Collins-Utilities (1999). Design storm hyetographs for Atlanta and Fort Collins are presented in Tables A.1 of Appendix A.

Total depths for two- and 100-year return interval storms for Fort Collins and Atlanta are summarized in Table 3.1. The depths reported for the 50-year record are based upon analysis conducted using Rainmaster, and the design storm depths were calculated as the event total for the hyetographs given in Table A.1.

3.2 Watershed Characteristics

Three drainage subwatersheds were delineated using 0.6096-meter contour maps, and are shown in Figure 3-2. Subwatershed areas, slopes and average runoff lengths were obtained using ArcView. Subwatershed widths were calculated by dividing the

Table 3.1. Two-Hour Design Storm Depths.

Location	Precipitation Depth (cm)			
	2-Year		100-Year	
	50-Year Record	Design Storm	50-Year Record[1]	Design Storm
Atlanta, GA	4.83	4.38	9.14	8.97
Fort Collins, CO	3.02	2.49	7.87	9.32

[1]Note that based upon the 50-year record, Rainmaster calculated an 83.7-year return interval rather than a 100-year return interval. The 83.7-year return interval is used to represent the 100-year event.

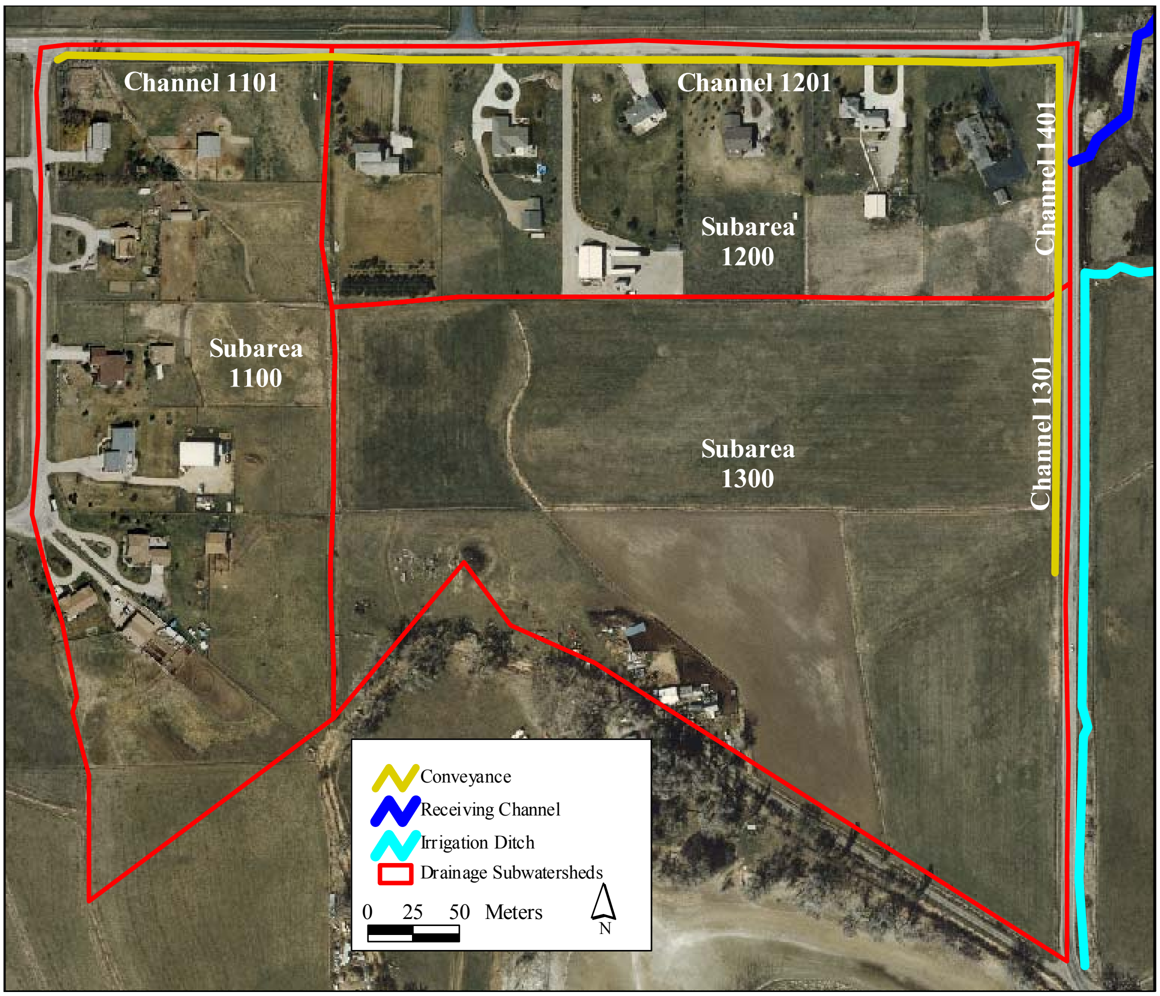

Figure 3-2. Drainage Subwatersheds: Existing Conditions.

subwatershed area by estimated average runoff length. Aerial digital photographs were used to estimate imperviousness for the low-density residential areas (13 percent). The impervious area of the undeveloped pastureland was assumed to be five percent.

Subwatershed specific characteristics used in SWMM Runoff to model existing conditions are summarized in Table 3.2.

Table 3.2. Watershed Characteristics: Existing Conditions.

Subwatershed	Width (m)	Area (ha)	Percent Impervious	Slope
1100	161	6.47	13.1	0.0150
1200	408	5.66	14.0	0.0127
1300	224	9.95	5.0	0.0187

Potential development within the watershed was simulated by converting the existing pastureland (Subarea 1300) to a medium-density residential subdivision shown in Figure 3-3. Representative lot sizes were modeled after existing residential subdivisions in the vicinity of the study watershed. Average lot sizes in the simulated subdivision were 0.15 ha including 27 m of frontage. The developed subwatershed was divided into seven subbasins, with an average imperviousness of 25 percent. The remaining watershed was left unchanged. Percent impervious values of subbasins containing the proposed residential lots were calculated using percent impervious values from subdivisions in the vicinity of the study watershed. Percent impervious values for subbasins that contained both residential lots and open space were calculated by using a weighted average of residential imperviousness and an assumed zero percent imperviousness for the open space. The percent impervious value for subbasin 1360, which contains the stormwater detention pond, was calculated using a weighted average, assuming the surface area of the WQCV was 100 percent impervious and the remaining surface area was zero percent impervious. Subwatershed specific characteristics for developed conditions are summarized in Table 3.3.

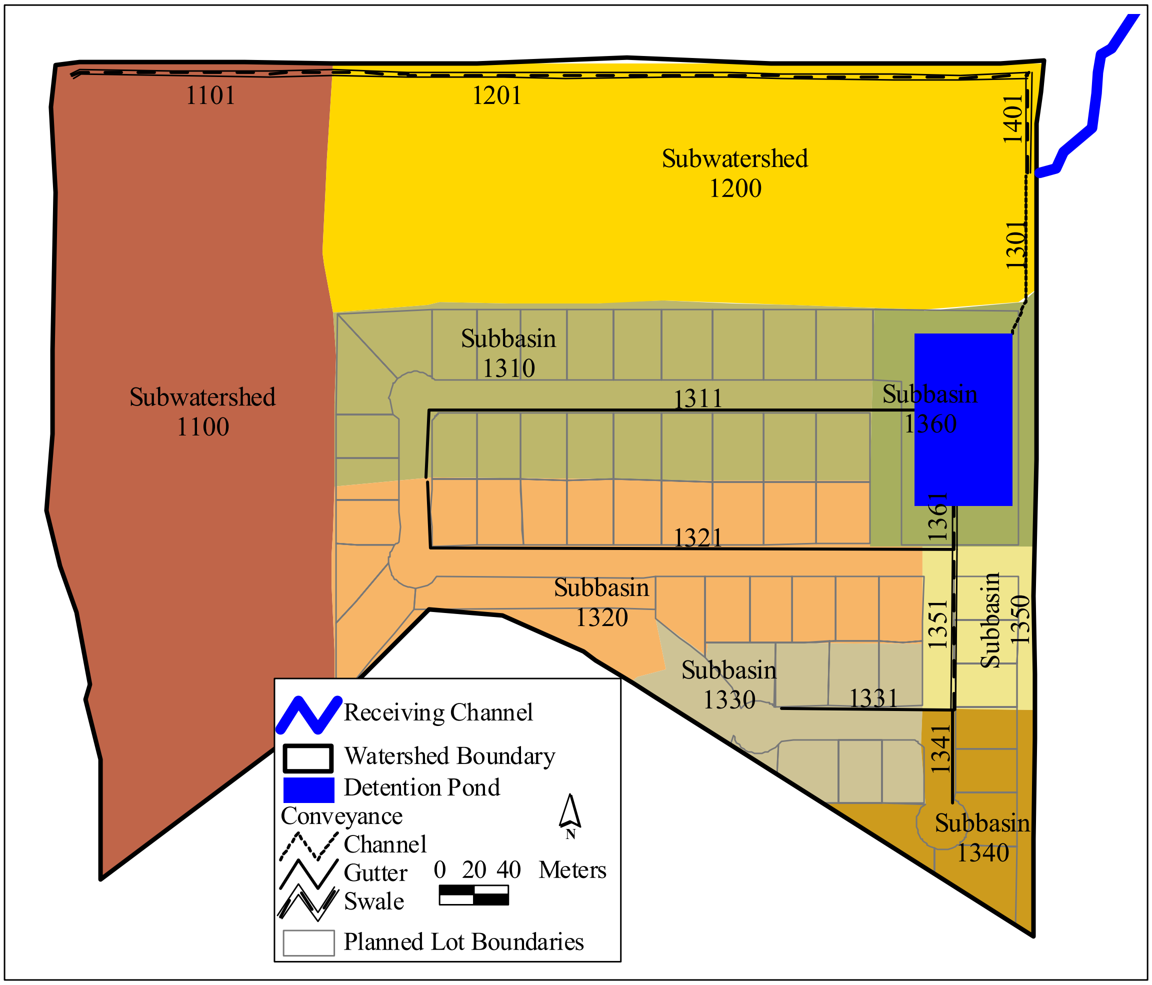

Figure 3-3. Drainage Subwatersheds and Subbasins, Developed Conditions.

Table 3.3. Watershed Characteristics: Developed Conditions.

Subbasin	Width (m)	Area (ha)	Percent Impervious	Slope
1100	161	6.47	13.1	0.0150
1200	408	5.66	14.0	0.0127
1310	906	3.11	29.2	0.0141
1320	812	2.97	24.6	0.0130
1330	232	1.13	26.6	0.0202
1340	189	0.81	29.2	0.0222
1350	157	0.59	29.2	0.0077
1360	614	1.31	10.9	0.0136

Under existing conditions, runoff from each subwatershed travels overland to grassed swales. Under developed conditions, runoff from each subbasin in Subarea 1300 travels overland to respective street gutters and grassed swales. Runoff conveyance from Subareas 1100 and 1200 was left unchanged. Gutters and swales within Subarea 1300 were sized to allow full conveyance of the 100-year storm. These conveyance channels were modeled in SWMM Runoff, which requires input characteristics describing channel length, slope, and geometry. Characteristics of the channels modeled in SWMM Runoff for existing conditions are described in Table 3.4, and for developed conditions in Table 3.5.

Table 3.4. Conveyance Channel Characteristics (Runoff): Existing Conditions.

Channel Number	Length (m)	Bottom Width (m)	Side Slope (m/m)	Depth (m)	Slope (m/m)	Manning n
1101	151	1.83	4	0.3810	0.010	0.035
1201	399	1.83	4	0.6096	0.005	0.035
1301	153	0.91	1	0.6096	0.008	0.040
1401	128	1.22	1	0.9144	0.006	0.035

Table 3.5. Conveyance Channel Characteristics (Runoff): Developed Conditions.

Channel Number	Length (m)	Bottom Width (m)	Side Slope (m/m)	Depth (m)	Slope (m/m)	Manning n
1101	151	1.83	4	0.3810	0.010	0.060
1201	399	1.83	4	0.6096	0.005	0.060
1311	341	0.15	12	0.2438	0.012	0.016
1321	339	0.15	12	0.2286	0.011	0.016
1331	98	0.15	12	0.2286	0.016	0.016
1341	54	0.15	12	0.1524	0.011	0.016
1351	92	0.91	4	0.5334	0.007	0.060
1361	80	0.91	4	0.6096	0.012	0.060
1401	128	1.22	1	0.9144	0.005	0.060

It should be noted that because gutters were sized to convey the 100-year storm, peak runoff values from the model are greater than those that would be generated under normal design conditions. Construction standards for the City of Fort Collins (1984) require a height of 15.2 centimeters for vertical curb and gutter. For this study, gutters reach a maximum depth of 24.4 centimeters. Sizing the gutters to allow full conveyance simplified the model, but reduced the runoff period of mid to high flows. Figures A-1 and A-2 of Appendix A show a comparison of outflow hydrographs using conventional sizing of gutters and sizing of gutters to convey the 100-year storm.

In addition to data describing watershed size, shape, slope & imperviousness, SWMM Runoff requires input of watershed characteristics related to infiltration, roughness, evaporation, and depression storage. The integrated form of the Horton equation was used to calculate infiltration capacity as a function of time. Use of the integrated form avoided an unwarranted reduction of infiltration capacity (f_p) during periods of light rainfall (Huber and Dickenson, 1988). For the continuous simulations, infiltration capacity was regenerated during dry weather. Input parameters for infiltration used in both the Atlanta and Fort Collins simulations were:

- minimum or ultimate value of f_p: 12.7 mm/hr,
- maximum or initial value of f_p: 13.0 mm/hr, and
- decay coefficient for regeneration of infiltration capacity: 0.108 min^{-1}.

These parameters were taken directly from the City of Fort Collins Storm Drainage Design Criteria (1997).

The SWMM Runoff model was run with the following watershed characteristics for all simulations:

- Snowmelt was not simulated,
- Evaporation from channels was not allowed,
- Default evaporation rate = 0.254 cm/day,
- Impervious area Manning's roughness = 0.016,
- Pervious area Manning's roughness = 0.250,

- Impervious area depression storage = 2.54 mm, and
- Pervious area depression storage = 7.62 mm.

These parameters were also taken from the City of Fort Collins Storm Drainage Design Criteria (1997).

Existing and developed condition runoff hydrographs were generated by version 44h of the SWMM Runoff model using the input data described above. Two- and 100-year design storms were modeled in addition to the 50-year continuous simulations. A listing of file names for each SWMM simulations, as well as continuity error percentages for each simulation, are given in Table D.1 of Appendix D.

3.3 Flow Routing and Stormwater Controls

Downstream flow routing and stormwater detention of the runoff hydrographs were modeled using SWMM Extran and SWMM Transport. Stormwater detention ponds and outlets for Subarea 1300 were sized in SWMM Extran, using simulations of 100- and two-year design storms. Stage-discharge curves developed from SWMM Extran simulations were used as input data for SWMM Transport, which was run for the 50-year continuous simulations.

SWMM Extran solves the St. Venant equation for gradually varied, unsteady open channel flow and simulates orifice discharge by solving the standard orifice equation given as:

$$Q = C_0 A \sqrt{2 g h} \qquad \textbf{Equation 3-1}$$

where: C_0 = orifice coefficient (0.65), A = area of orifice, g = gravitational constant (9.81 m/s), and h = hydraulic head on the orifice. After solving the orifice equation, SWMM

Extran converts the orifice to an equivalent pipe, using the Manning pipe flow equation (Roesner et. al., 1988). SWMM Transport uses a kinematic wave approach to route flows through the channel system. Detention pond storage and discharge are modeled using a stage-volume-surface area-discharge data set input by the user.

In addition to examining the existing and developed, uncontrolled conditions three levels of stormwater control within Subarea 1300 were examined in this study. The first stormwater control examined is an over control method, and is based upon detention storage requirements in the City of Fort Collins for areas where master drainage plans are not in place. These requirements are summarized as follows:

> *In basins where a master drainage plan has not been approved, the City may require detention storage in accordance with this section to protect irrigation structures or downstream development. The stormwater runoff shall not be released from developments at a rate greater than the two-year historic runoff. The amount of runoff to be detained on-site shall be the difference between the 100-year runoff under developed conditions and the two-year historic runoff. This includes runoff from the crown of adjacent streets whether they are existing or proposed streets. In all cases, it should be assumed that detention is required unless proven otherwise, such as when an adequate outfall to a major drainage way is available to discharge 100-year developed flows.*
>
> - *City of Fort Collins – Water Utilities, 1997.*

The second level of stormwater control examined includes the addition of control for water quality to the existing City of Fort Collins over control detention standards. This is referred to hereafter as the Over Control + BMP scenario. BMP represents the term best management practice, which in this study is a facility designed to treat the water quality capture volume (WQCV).

The third level of stormwater control examined is control of the WQCV combined with peak shaving practices for stormwater detention that are commonly required across the United States. Typical peak shaving practices for stormwater detention require control of:

- the 100-year postdevelopment runoff rate to the 100-year historic rate, and
- the two-year postdevelopment runoff rate to the two-year historic rate.

Existing condition runoff rates for Subarea 1300 are shown in Table 3.6. These historic rates are the peak runoff rates estimated by simulations of existing watershed conditions in SWMM Runoff.

Table 3.6. Historic runoff rates from Subbasin 1300.

Design Storm	Peak Discharge (cms)	
	Atlanta	Fort Collins
2-Year	0.17	0.07
100-Year	0.71	0.75

Detention ponds and outlets for the three stormwater control scenarios were sized in SWMM Extran. The SWMM Extran model was established to accept the two- and 100-year hydrographs generated by SWMM Runoff, route the runoff generated from subbasin 1300 through the various levels of detention, and convey the water to the downstream receiving channel. Table 3.7 lists the hydraulic characteristics of the conveyance channels, which are the same for both the Atlanta and Fort Collins simulations. It should be noted that channels were modeled as the same size for both the Atlanta and Fort Collins simulations to allow a sensitivity analysis comparison in different climate regions, even though it is recognized that receiving streams of watersheds of the same size within different climate regions will have varying channel geometry. The conveyance channels were also sized to allow full conveyance of the 100-year storm without overbank flow, consistent with sizing of the gutters and swales in the SWMM

Table 3.7. Hydraulic Characteristics of SWMM Extran Conveyance Channels.

Channel Number	Length (m)	Bottom Width (m)	Side Slope H:V (m:m)	Depth (m)	Slope (m/m)	Manning n
1001	153	0.76	4	1.22	0.034	0.04
1301	120	0.61	4	1.22	0.010	0.04

Runoff model. This is also recognized as a limitation of this study, and was implemented due to the difficulty of modeling overbank flow in SWMM Transport.

The WQCV was computed as the product of the mean runoff event and a drawdown coefficient, resulting in the capture of 70-90 percent of all runoff-producing events in their entirety. The WQCV for Atlanta was calculated using procedures recommended by WEF and ASCE (1998) as a national standard. Using a draw-down time of 24-hours, the resulting WQCV was calculated as:

$$P_0 = (a\,c)P_6 \qquad \textbf{Equation 3-2}$$

where, a = drawdown coefficient (1.299), P_6 = mean storm precipitation over the watershed (18.0 mm (WEF and ASCE, 1998)) and c = runoff coefficient. The runoff coefficient was calculated using:

$$c = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \textbf{Equation 3-3}$$

where, i = watershed imperviousness ratio. The WQCV for Fort Collins was calculated using a slightly modified method described by UDFCD (2001). The calculated WQCV for Atlanta was 0.046 ha-m, and for Fort Collins, 0.041 ha-m. Orifice outlets for both water quality ponds were sized using an iterative procedure in Excel.

The physical dimensions of the WQCV ponds were designed according to the recommendations of UDFCD (2001), and are summarized in Table 3.8. The maximum depth of each pond was equal to 0.3048 m, with a side slope ratio (w:d) of 4:1. The UDFCD recommended pond width to length ratio of 1:2 was achieved for the Fort Collins pond and a width: length ratio of 1:1.75 was achieved for Atlanta. The shapes of the WQCV ponds were approximated as elliptical frustrums, and volumes were calculated using a modified form of the volume equations for pyramid and conical frustrums provided by Pontuti (2001):

$$V = \tfrac{1}{3}\, h\left(A_1 + A_2 + \sqrt{A_1 A_2}\right) \qquad \textbf{Equation 3-4}$$

where, A_1 = surface area of top of pond, A_2 = surface area of bottom of pond, and h = depth of pond.

Table 3.8. WQCV Pond Characteristics.

Location	Total Volume (ha-m)	Bottom Surface Area (m^2)	Top Surface Area (m^2)	Depth (m)	Outlet Diameter (cm)
Atlanta	0.047	1454	1630	0.3048	7.62
Fort Collins	0.041	1262	1430	0.3048	7.62

Dimensions of the detention ponds were similar to the WQCV ponds, using the same bottom surface area, side slope width to depth ratios, and length to width ratios for each scenario. Ponds depths were sized to allow for the UDFCD (2001) recommended 0.3048 meters of freeboard. Detention outlets were sized to restrict postdevelopment peak discharges to existing condition rates using the 100-year and two-year return intervals.

In the Atlanta and Fort Collins analyses, detention outlets designed to over control the 100-year postdevelopment peak discharge rate to the two-year historic peak discharge rate were sized using peak discharge rates calculated when the two- and 100-year design storm hyetographs presented in Table A.1 were applied to the watersheds. Similarly, for the Atlanta peak shaving detention design, the 100- and two-year detention outlets were sized using discharges generated by the application of these same design storm hyetographs.

In the Fort Collins peak shaving detention design, the 100-year outlet was sized to restrict the 83.7-year return interval peak discharge to the peak discharge rate estimated by the existing condition 50-year continuous simulation. A two-year detention outlet was not designed for the Fort Collins peak shaving detention design because the total runoff volume of the two-year design storm was nearly equal in volume to the WQCV. Rather than designing an additional outlet for the two-year storm, the two-year storm was allowed to discharge through the WQCV orifice.

A summary of the designed orifice diameters and detention pond dimensions is presented in Tables 3.9 and 3.10 for Atlanta and Fort Collins, respectively. A profile view of the outlet headwalls for each stormwater pond is shown in Figure 3-4.

Time series stage data from the detention ponds and discharge data from the orifice(s) were used to create the stage-discharge relationship input for SWMM Transport. Stage-discharge relationship equations were calculated in Excel by fitting trend lines to the SWMM Extran stage-discharge output. Separate equations were generated for stages below orifice submergence conditions and for stages above orifice submergence conditions. Simulations in SWMM Transport were run using the 50-year precipitation record output from SWMM Runoff. File names and continuity error percentages for each simulation are presented in Table D.1.

Table 3.9. Characteristics for Detention and Water Quality Ponds in Atlanta.

Scenario	Total Volume (ha-m)	Top Surface Area (m^2)	Max 100 Year Depth (m)	Detention: 1st Stage		Detention: 2nd Stage	
				Outlet Diameter (cm)	Depth Above Invert (m)	Outlet Diameter (cm)	Depth Above Invert (m)
Over Control	0.644	3377	2.42	22.9	0.000	N.A.	N.A.
Over Control + BMP	0.644	3377	2.43	22.2	0.305	N.A.	N.A.
Peak Shaving + BMP	0.454	2884	2.02	30.5	0.305	54.6	1.143

N.A. = Not Applicable.

Table 3.10. Characteristics for Detention and Water Quality Ponds in Fort Collins.

Scenario	Total Volume (ha-m)	Top Surface Area (m^2)	Max 100 Year Depth (m)	Detention: 1st Stage	
				Outlet Diameter (cm)	Depth Above Invert (m)
Over Control	0.679	3363	2.84	14.0	0.000
Over Control + BMP	0.679	3363	2.84	12.1	0.305
Peak Shaving + BMP	0.407	2635	2.23	41.9	0.406

3.4 Flow Frequency Analysis

The SWMM Statistics block was used to examine the frequency of peak flows generated by individual events during the 50-year records. Frequency exceedance curves were developed from the partial duration series of peak flows generated by SWMM Transport. This approach, which is in contrast to the examination of the annual maximum series, was used because it allowed for the analysis of high frequency, low runoff producing storms. In addition, this approach describes the frequency with which a given flow probably occurs as opposed to simply describing the annual maximum flow. A six-hour inter-event time and minimum threshold of 0.003 cms was specified to separate the flow data into individual events.

The Cunnane (1978) formula was used to calculate the frequency of an event peak flow, T, as:

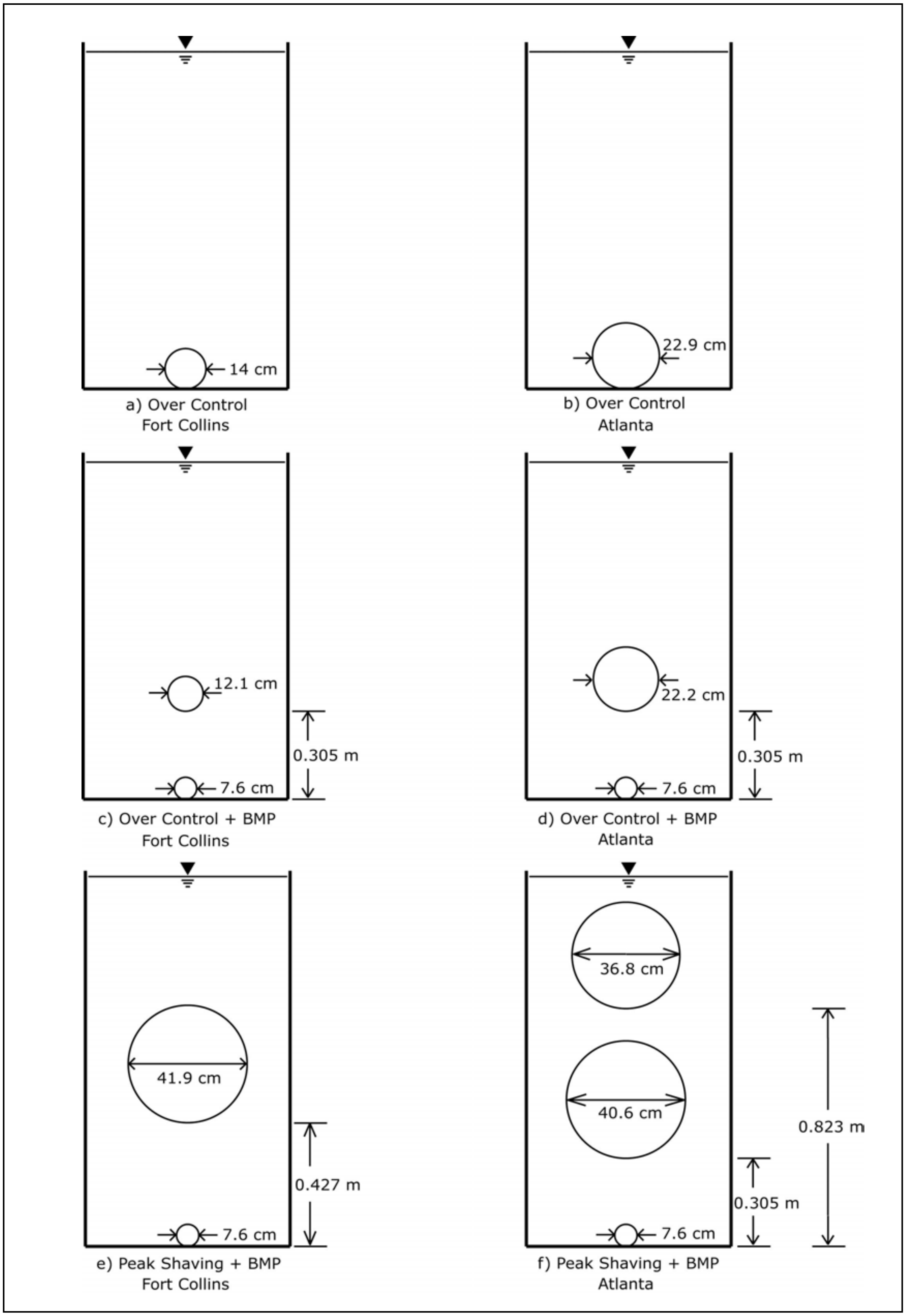

Figure 3-4. Profile view of stormwater pond outlets.

$$T = \frac{N + 1 - 2A}{M - A}$$ **Equation 3-5**

where, T = return interval (years), N = number of years of record, M = rank of the event (in descending order of magnitude), and A = plotting position parameter (0.4). The return interval was converted to exceedances per year, E, using:

$$E = \frac{1}{T}.$$ **Equation 3-6**

3.5 Sediment Transport Analysis

Stage and discharge output from the 50-year continuous simulations in Transport were used to calculate indicators of sediment transport capacity (average boundary shear stress and specific stream power), as well as sediment transport rates using a bedload and a total load equation. Two noncohesive bed materials were evaluated for sediment transport capacities under the existing watershed conditions and the four proposed development scenarios. Bedload transport was evaluated for streambed material consisting of medium gravel ($d_s > 8$ mm) and total load was evaluated for streambed and material consisting of medium sand ($d_s > 0.25$ mm).

Streambed and bank materials for the receiving water immediately downstream of the study area were not collected, however, a grain size distribution of bed and bank materials for a reach of Fossil Creek immediately upstream it's confluence with the tributary serving the watershed in this study were presented by Lidstone and Anderson (1992). Results from the washed-sieve analysis showed that the d_{16} of stream banks at three locations were all classified as medium silt (0.016 mm < d_s < 0.031 mm) and the d_{84} of stream banks at the three locations were all classified as fine sands. The stream bank

d_{50} values ranged from coarse silt (0.031 mm < d_s < 0.062 mm) at two locations to very fine sand (0.062 mm < ds < 0.125 mm) at the third location. Streambed materials varied more widely. The d_{16} of the six streambed samples ranged from fine sand (0.008 mm < d_s < 0.016 mm) to very coarse sand (1.0 mm < d_s < 2.0 mm), with an average value in the coarse sand (0.5 mm < d_s < 1.0 mm) category. The d_{84} of the streambed samples ranged from very coarse sand to very coarse gravel (32 mm < d_s < 64 mm), with an average value in the size range of medium gravel (8 mm < d_s < 16 mm). The d_{50} of the streambed samples ranged from medium sand (0.25 mm < d_s < 0.50 mm) to fine gravel 4.0 mm < d_s < 8.0 mm) with an average size equal to very fine gravel (2.0 mm < d_s < 4.0 mm).

Noncohesive bed particles enter motion when the shear stress applied to the bed material exceeds the critical shear stress. Threshold conditions for movement occur when the fluid flow around a sediment particle exerts a force that is balanced with the resisting force of the particle weight. This condition of balance is called incipient motion (Julien, 1995). Incipient motion is evaluated by comparing a *calculated* Shield's Parameter, τ_*, to a *critical* Shield's Parameter, τ_{*_c}, specified for the streambed materials examined. When the *calculated* Shield's Parameter is greater than the *critical* Shield's parameter, it is assumed that the particle is in motion. Table 3.11 summarizes the critical Shield's Parameters used for this study. The Shield's Parameter was calculated using:

$$\tau_* = \frac{\tau}{(\gamma_s - \gamma) d_s} \qquad \textbf{Equation 3-7}$$

where, γ_s = specific weight of the sediment particle, γ = specific weight of the fluid mixture (assumed to be same as water), d_s = particle size (e.g., median value, such as d_{50}), and τ = average boundary shear stress, calculated as:

$$\tau = \gamma R_h S_f$$ **Equation 3-8**

Table 3.11. Summary of Threshold Conditions at 20° C.

Class Name	Particle Diameter (mm)	ω_o (mm/s)	τ_{*c}	τ_c (Pa)
Medium Sand	> 0.25	36	0.048	0.194
Medium Gravel	> 8	338	0.044	5.7

Source: Julien, 1995.

where, R_h = hydraulic radius of channel, and S_f = friction slope of channel.

Specific stream power has been suggested as a descriptor of hydraulic conditions and sedimentation processes in stream channels (Bledsoe, 1999). Specific stream power is the energy expended per unit area of streambed, relating velocity, depth, and energy slope. Specific stream power was calculated as:

$$\omega = \frac{\gamma Q S}{w}$$ **Equation 3-9**

where, Q = discharge and w = the channel width.

Typically coarse particles move by rolling or sliding in a thin layer near the streambed, called the bed layer. The bed layer thickness is approximately twice the particle diameter (Julien, 1995). Bedload, q_{bv}, is the total mass of particles moving in the bed layer and can be calculated as a function of grain size and the difference between the *calculated* and *critical* Shield's parameter. Bedload can be estimated using the Chien (1956) form of the Meyer-Peter and Müller (MPM) equation presented by Julien (1995):

$$q_{bv} = 8(\tau_* - \tau_{*c})^{3/2}\sqrt{(G-1)gd_s^3}$$ **Equation 3-10**

where, G = specific gravity, and g = gravitational acceleration.

Total load is a sum of the total sediment discharge of a stream. Total load includes suspended sediment discharge, as well as bedload sediment discharge. It is also a sum of the bed discharge and the washload (Julien, 1995, Richardson et. al., 2001). For this analysis, the total load calculations assume that the sediment supply is unlimited, and do not take into account whether the supply comes from the streambed, or from the watershed as washload. Total load was calculated using Brownlie's (1981) method, presented by Julien (1995). Brownlie calculates total load, C_{ppm}, using:

$$C_{ppm} = 7115\, c_b \left(\frac{V - V_c}{\sqrt{(G-1) g\, d_s}} \right)^{1.978} S_f^{0.6601} \left(\frac{R_h}{d_s} \right)^{-0.3301} \quad \textbf{Equation 3-11}$$

where,

c_b = Brownlie coefficient, assumed 1.0 for this analysis, and

$$V_c = \sqrt{(G-1) g d_s} * 4.596\, \tau_{*c}^{0.529} S_f^{-0.1405} \sigma_g^{-0.1606} \quad \textbf{Equation 3-12}$$

where, σ_g = gradation coefficient. The gradation coefficient was assumed to be 1.0 for this analysis.

4.0 RESULTS

4.1 Discharge Magnitude, Frequency and Duration

The magnitude, frequency and duration of discharges are critical for determining impacts that changes in flow regime will have on geomorphology. This section discusses changes in each of these between the scenarios evaluated in this study.

Detention ponds and outlets were sized to restrict postdevelopment outflows to the peak discharge criteria summarized in Section 3.3 for the three levels of stormwater controls examined. Table 4.1 summarizes the peak discharge values for the two- and 100-year return interval storms for each scenario evaluated. The peak discharge values presented in Table 4.1 are for discharges from Subarea 1300 only, and were generated by the SWMM Transport model. For detention ponds with multiple orifices (e.g., BMP scenarios), the peak discharge is the sum of the discharge rate from the BMP orifice and the detention orifice(s). For existing conditions, Table 4.1 summarizes both peak

Table 4.1. Peak Discharges for the Developing Watershed.

Climate	Return Interval	Peak Discharge (cms)					
		Continuous	Design Storm				
		Existing	Existing	Dev. Uncont.	Over Control	Over Control + BMP	Peak Shaving + BMP
Atlanta	2-Year	0.17	0.14	0.80	0.10	0.09	0.13
	100-Year	0.71[1]	0.71[1]	0.71	0.17	0.17	0.68
Fort Collins	2-Year	0.03	0.07	0.32	0.02	0.02	0.01
	100-Year	0.45[1]	0.75	3.29	0.08	0.08	0.48

[1]These values are for the 83.7-year return interval, based upon the 50-year record, which was used in place of the 100-year storm.

discharge values generated by both the continuous 50-year simulations and the design storm simulations that used the hyetographs presented in Table A.1. The remainder of the reported peak discharge values were generated by the design storm simulations, rather than the continuous simulations.

Table 4.1 demonstrates that the over control criteria result in 100- and two-year peak discharge values well below the existing condition peak discharge values generated by respective design storms. Table 4.1 also shows that for Fort Collins, the two-year peak discharge from the Peak Shaving + BMP scenario is much less than the Existing scenario peak discharge. This is because the WQCV for Fort Collins is nearly the same value as the volume of the two-year design storm. The 7.6 cm orifice required to draw down the WQCV over a 24-hour period prevents the peak discharge of the two-year storm from reaching the existing condition peak discharge values. Outflow hydrographs for the 100- and two-year design storms applied to Subarea 1300 for Atlanta and Fort Collins are given in Appendix B, and outflow hydrographs for the full watershed are given in Appendix C.

Differences between existing condition peak discharges generated by the continuous simulations and the design storm simulations are significant for the Fort Collins analysis, as shown in Table 4.1. This phenomenon was also observed by Nehrke and Roesner (2004) and occurs because the precipitation data used for the event-based simulations were design storms taken from City of Fort Collins requirements (1999), rather than design storms computed from the actual data record used for continuous simulations. In the case of the 100-year return interval, the 100-year storm designated by the City of Fort Collins (1999) is significantly larger than the design storm estimated from the 50-year record, as was shown in Table 3.1.

The City of Fort Collins (1999) 100-year design storm was originally used to design the detention pond outlet for the Peak Shaving + WQCV scenario, restricting the 100-year peak discharge to 0.75 cms. When the flow frequency curve for this scenario was plotted alongside the flow frequency curve for existing conditions, it became apparent that using the design storm criteria would not allow the flow frequency curves to match, as shown in Figure 4-1. This instigated use of the continuous simulation peak discharge value of 0.45 as the target peak discharge rate for the 100-year storm in the final Peak Shaving + BMP design. The design using the continuous simulation peak discharge value allowed the Peak Shaving + BMP scenario flow frequency curve to better match the curve for existing conditions, as demonstrated in Figure 4-1.

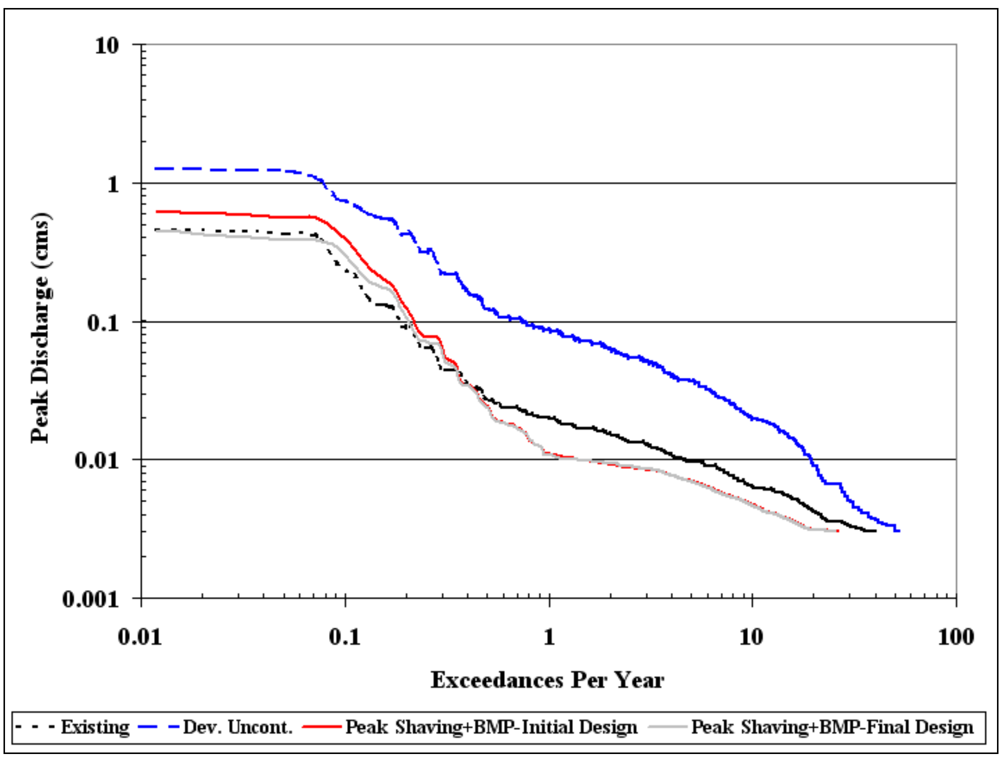

Figure 4-1. Initial and Final Design Peak Shaving + BMP Curves.

Figures 4-2 and 4-3 show the peak discharge exceedance curves resulting from continuous simulation of the existing and postdevelopment conditions examined for Atlanta and Fort Collins, respectively. These figures show discharge exceedances from Subarea 1300 only. As expected, Figures 4-2 and 4-3 show that when the watershed is developed without stormwater controls, the peak discharges for storms of all return intervals are greater than peak discharges under existing conditions.

The two Over Control scenarios show that peak discharges for the 83.7-year storms (0.011 exceedances per year) do not exceed the two-year peak discharge (0.5 exceedances per year) values for the Existing scenarios. When the Developed Uncontrolled 83.7-year peak discharge is controlled to the existing two-year level, peak discharges for all storms greater than the two-year storm are significantly lower than under existing conditions, as shown in Figures 4-2 and 4-3. Peak discharges from the Over Control scenarios are greater than existing condition peak discharges that are exceeded between 1.3 – 11.5 times a year for Atlanta, and 0.5 – 6.2 times a year for Fort Collins. This means that peak discharges are uncontrolled for storms with return intervals between 0.8 – 0.1 years for Atlanta, and 2.0- to 0.two-years for Fort Collins. Over Control scenario peak discharges that occur more than 11.5 times a year for Atlanta and more than 6.2 times a year in Fort Collins are less than those that occur under existing conditions.

Peak discharges from the Peak Shaving + BMP scenarios track fairly close to the Existing scenario peak discharges from the 83.7-year to the two-year return intervals for both Atlanta and Fort Collins. For Atlanta, peak discharges from the Peak Shaving +

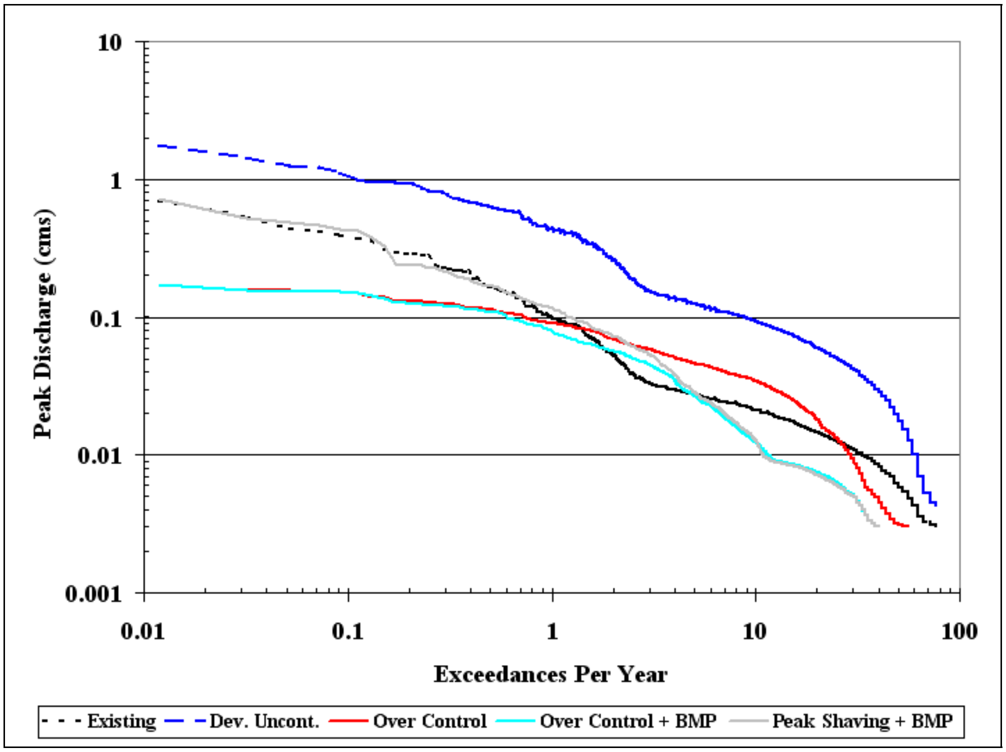

Figure 4-2. Peak flow exceedance frequency in Subarea 1300: Atlanta.

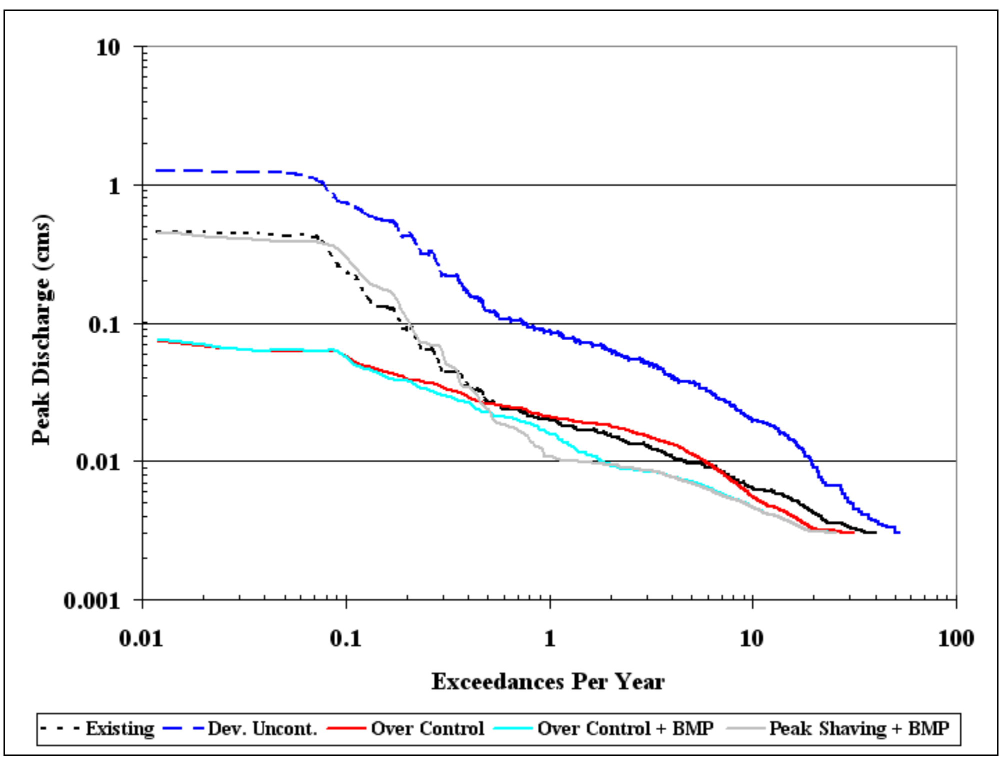

Figure 4-3. Peak flow exceedance frequency in Subarea 1300: Fort Collins.

BMP and Over Control + BMP scenarios become nearly the same for all storms that occur more than five times a year, indicating that the BMP is effective for storms with a return interval of 0.2 years. Peak discharges from storms that occur more than 5 times a year in the BMP scenarios are less than the peak discharges in the Atlanta Existing scenario.

For Fort Collins, peak discharges from the Peak Shaving + BMP and Over Control + BMP scenarios become nearly the same for all storms that occur more than 2 times a year. Peak discharges that occur more than 0.3 times a year in the Peak Shaving + BMP scenario are less than the peak discharges in the Existing scenario. Figure 4-3 illustrates that the Fort Collins Peak Shaving + BMP peak discharge at the two-year return interval is less than the peak discharge under existing conditions. The existing two-year peak discharge could not be reached because the WQCV was nearly equal to the volume of water generated by the two-year storm.

Figures 4-4 and 4-5 show the peak flow frequency exceedance curves for the full watershed; i.e., the peak discharges in channel downstream of the confluence of discharges from Subareas 1100, 1200, and 1300. These figures show that downstream changes in the flow frequency curves are not as pronounced between scenarios as they were in the peak flow exceedance frequency curves from Subarea 1300. The flow exceedance frequency curves are closer together because of the dampening effect of the runoff contribution from Subareas 1100 and 1200, where stormwater is uncontrolled. Flow frequency exceedance curves from Subareas 1100 and 1200 are unchanged between each scenario. Figure 4-4 shows that the largest peak discharge values for the Over Control and the Over Control + BMP scenarios are less than the Existing scenario two-

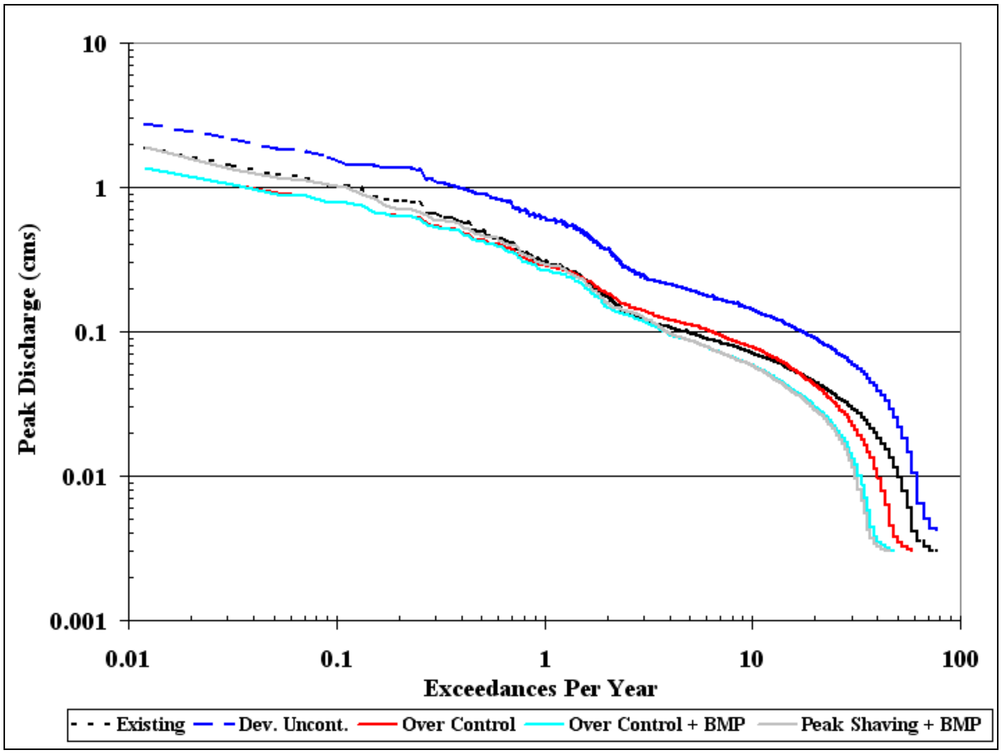

Figure 4-4. Peak flow exceedance frequency, full watershed: Atlanta.

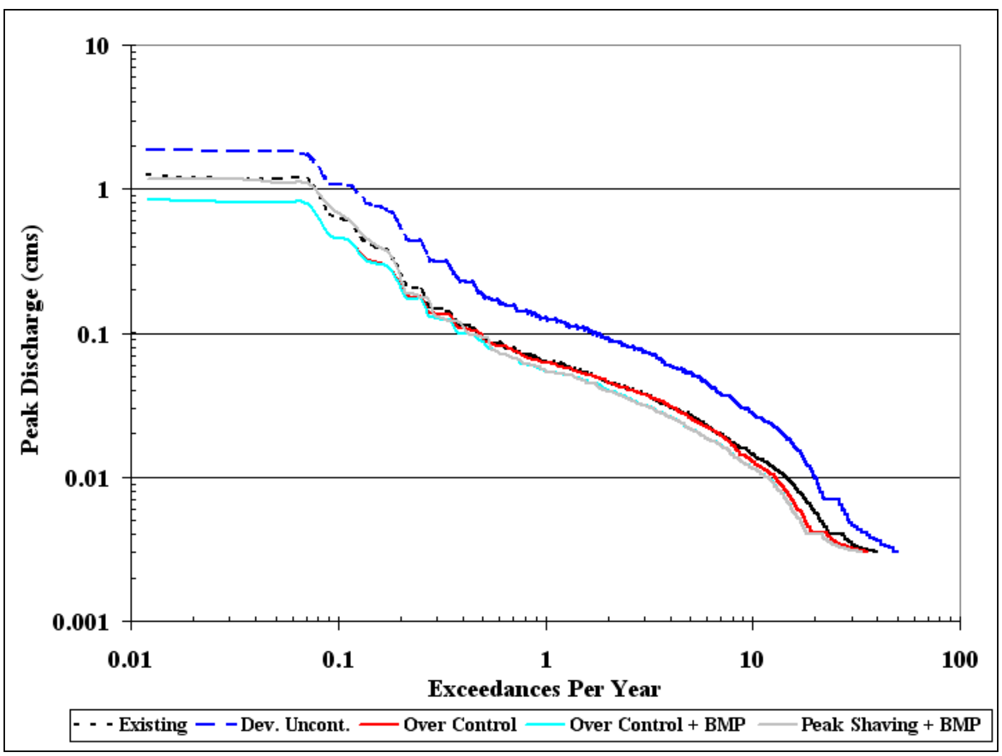

Figure 4-5. Peak flow exceedance frequency, full watershed: Fort Collins.

year design storm peak discharge value of 0.07 cms. These peak discharge values are equaled or exceeded 0.012 times a year or at an 84-year return interval.

Tables 4.2 and 4.3 summarize downstream peak discharges at various return intervals. These peak discharges were obtained from the partial duration series of peak flows generated by SWMM Transport. Note that these peak discharges are different from

Table 4.2. Summary of Design Storm Peak Discharges: Atlanta.

Return Interval (years)	Peak Discharge (cms)				
	Existing	Developed Uncontrolled	Over Control	Over Control + BMP	Peak Shaving + BMP
83.6	1.92	2.78	1.36	1.36	1.89
50	1.61	2.39	1.16	1.16	1.53
25	1.31	2.02	0.97	0.97	1.24
10	1.01	1.54	0.78	0.78	1.02
2	0.49	0.90	0.43	0.42	0.47
1.5	0.42	0.79	0.38	0.36	0.40
1	0.30	0.61	0.29	0.27	0.29
0.5	0.17	0.37	0.18	0.15	0.16
0.25	0.11	0.21	0.12	0.09	0.10
0.1	0.07	0.14	0.08	0.06	0.06

Table 4.3. Summary of Design Storm Peak Discharges: Fort Collins.

Return Interval (years)	Peak Discharge (cms)				
	Existing	Developed Uncontrolled	Over Control	Over Control + BMP	Peak Shaving + BMP
83.6	1.26	1.93	0.86	0.86	1.18
50	1.20	1.86	0.82	0.82	1.17
25	1.17	1.82	0.80	0.80	1.13
10	0.64	1.09	0.46	0.46	0.69
2	0.09	0.18	0.09	0.09	0.09
1.5	0.08	0.15	0.08	0.07	0.07
1	0.06	0.13	0.06	0.05	0.05
0.5	0.05	0.09	0.05	0.04	0.04
0.25	0.03	0.06	0.03	0.03	0.03
0.1	0.01	0.03	0.01	0.01	0.01

peak discharges reported in Tables 4.1, which were obtained from simulations of design storm events in Subarea 1300 only.

Figures 4-6 and 4-7 show that the duration of various downstream discharges are exceeded during the 50-year continuous simulation. Figure 4-6 shows that, in Atlanta, flows are greater than baseflow approximately 10 percent of the time in the Existing and Developed Uncontrolled scenarios, but flows are greater than baseflow approximately 50 percent of the time for the postdevelopment scenarios where stormwater is controlled (i.e., the Over Control scenario). Figure 4-7 shows that in Fort Collins flows are greater than baseflow approximately five percent of the time in the Existing and Developed Uncontrolled scenarios, but flows are greater than baseflow approximately 12 percent of the time for the postdevelopment scenarios where stormwater is controlled. Quantitative sediment transport implications due to these flow duration changes are examined in Sections 4.3 and 4.4. Discharge duration curves in log- and semi-log-scale for Subarea 1300 are given in Appendix B. Semi-log discharge duration curves for the full watershed are presented in Appendix C.

4.2 Indicators of Sediment Transport

As discussed in Section 3.5, incipient motion is said to occur when the fluid flow around a sediment particle exerts a force that is greater than the resisting force of the particle weight. This condition can be evaluated by comparing a *calculated* Shield's Parameter (Equation 3-7) to a *critical* Shield's Parameter for the bed material examined. By association, incipient motion of sediment can also be evaluated by comparing shear stress to a critical shear stress (τ_c). Figures 4-8 and 4-9 display shear stress durations in

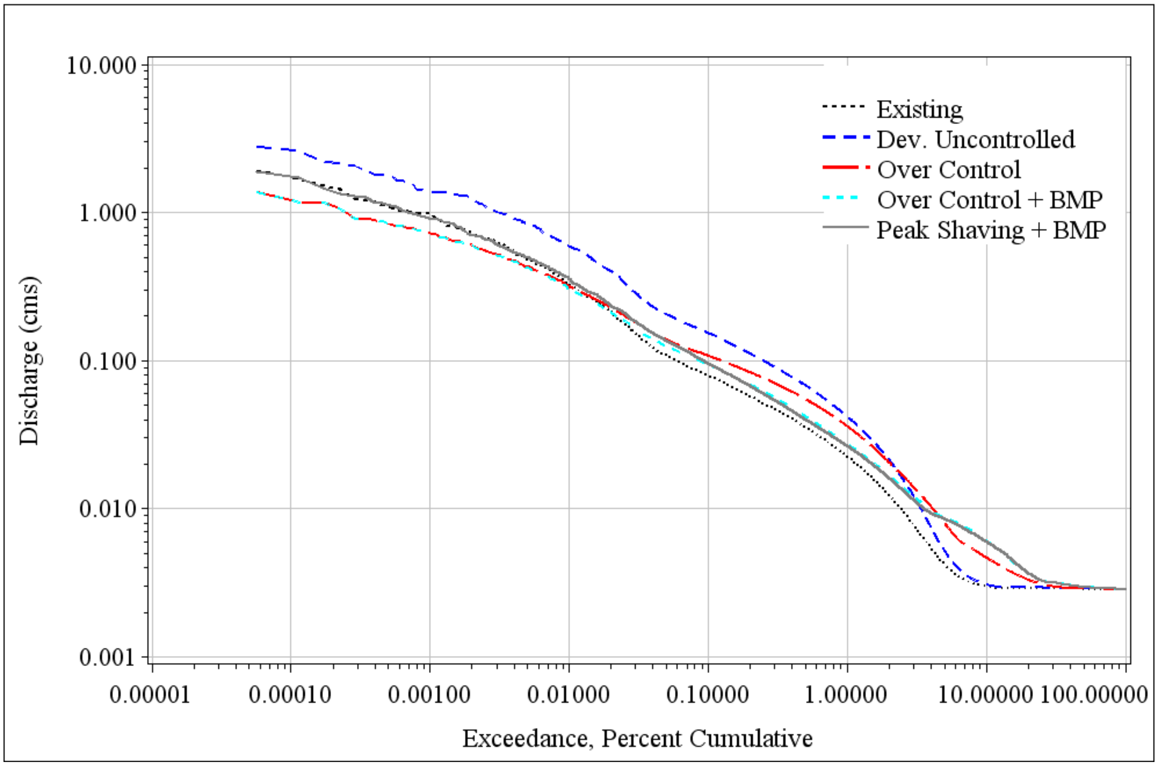

Figure 4-6. Discharge duration, full watershed: Atlanta.

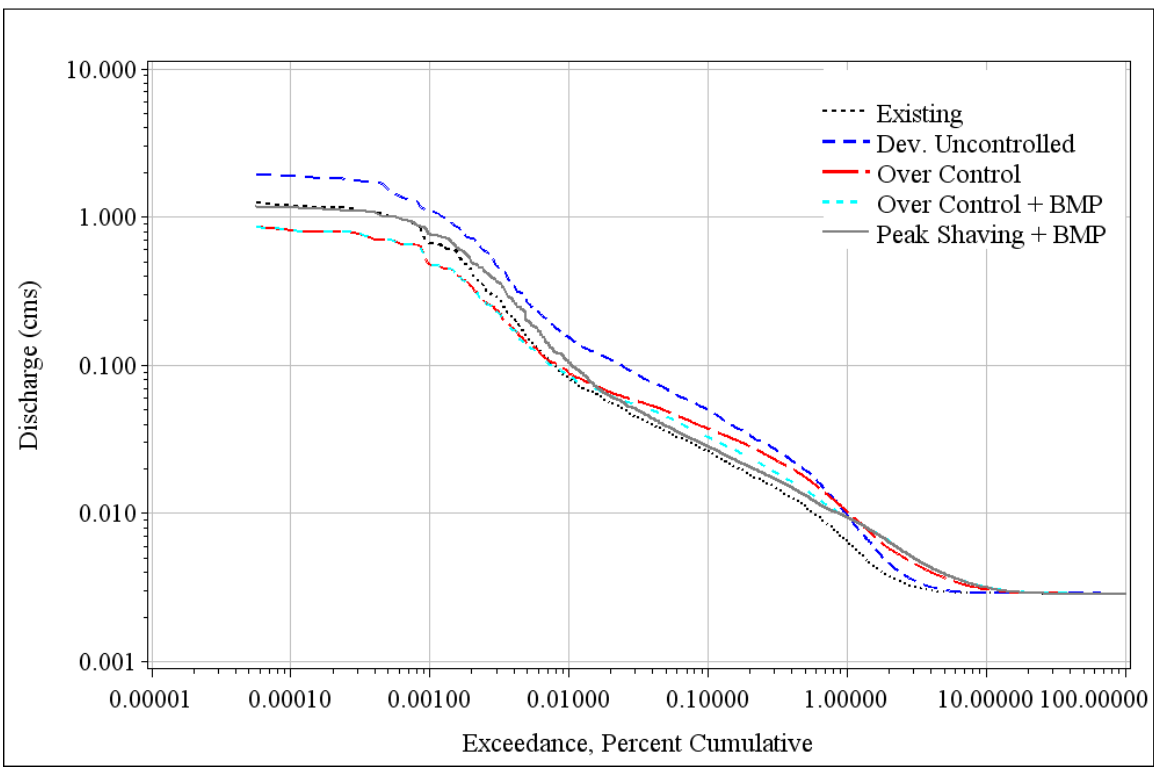

Figure 4-7. Discharge duration, full watershed: Fort Collins.

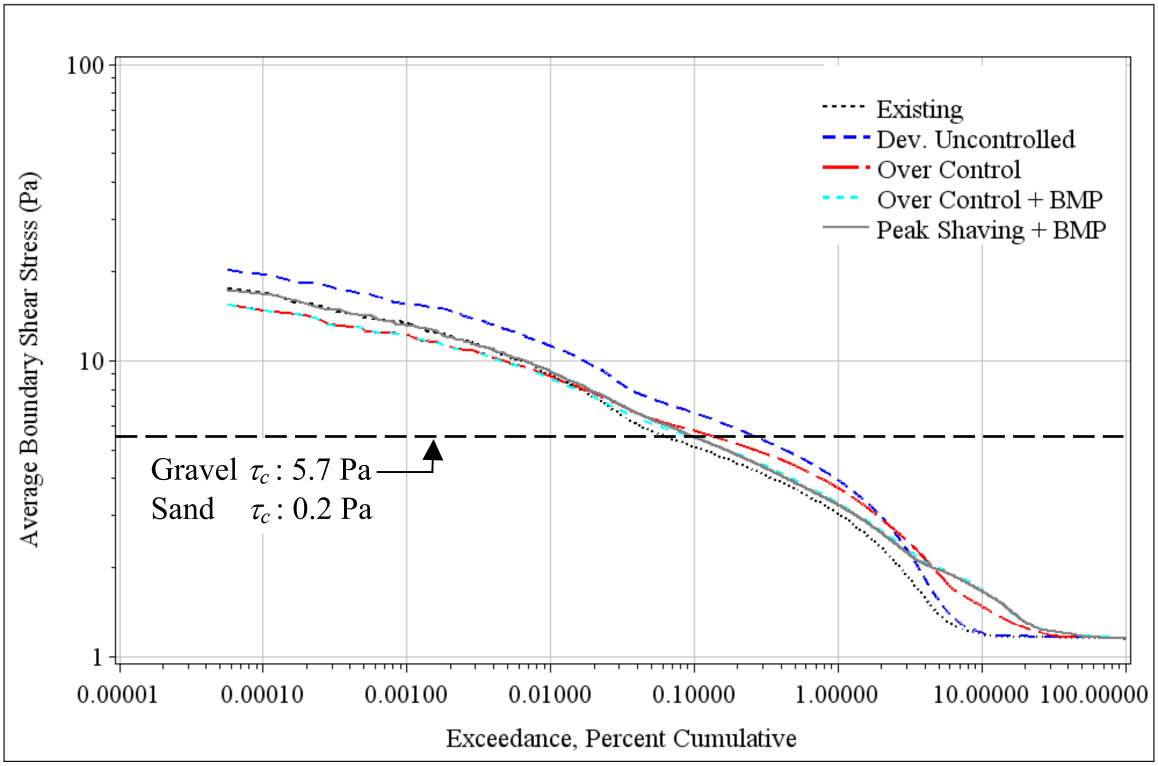

Figure 4-8. Average boundary shear stress, full watershed: Atlanta.

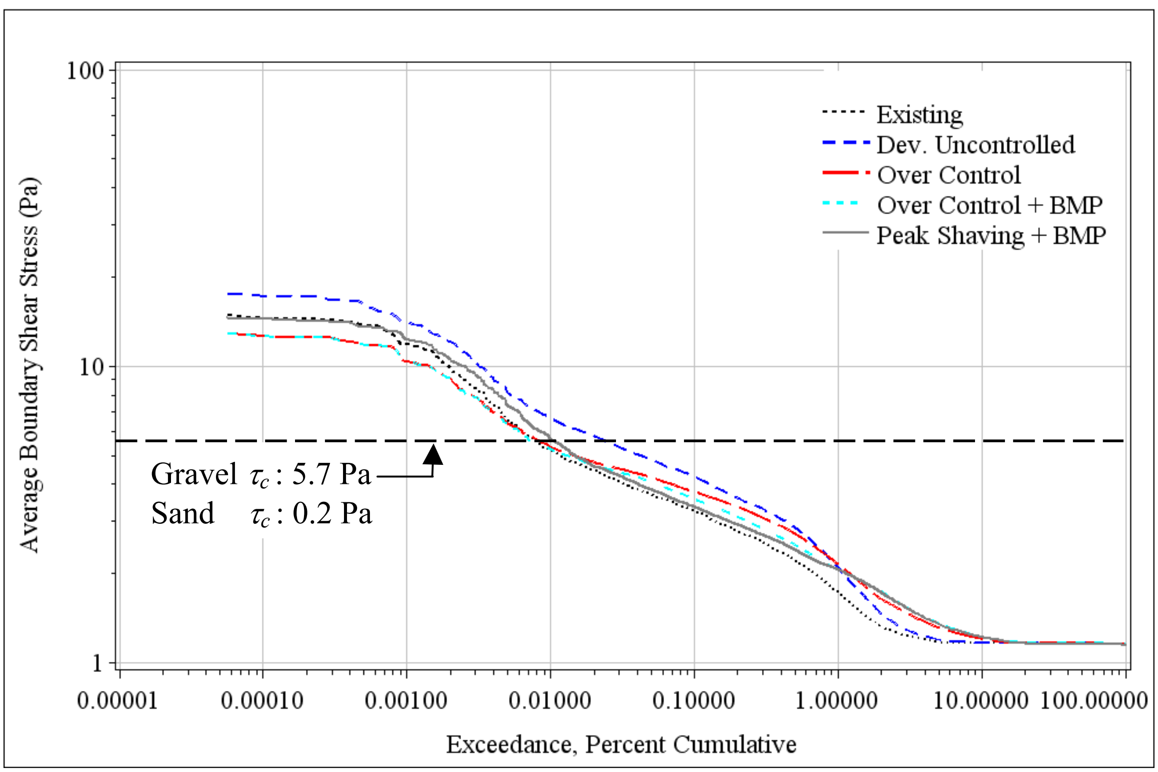

Figure 4-9. Average boundary shear stress, full watershed: Fort Collins.

the channel downstream of the full watershed. These shear stress durations were computed using the five scenarios examined over the 50-year record.

Table 3.14 showed that critical shear stress values for medium sand and medium gravel are 0.2 Pa and 5.7 Pa, respectively. From Figures 4-8 and 4-9 it can be seen that the average boundary shear stress in the channel is greater than the critical shear stress for medium sand 100 percent of the time. This indicates that sediment transport is taking place continuously in both the Atlanta and Fort Collins analyses. Incipient motion thresholds for medium gravel are exceeded 0.07 – 0.3 percent of the time (10 – 41 days over 50 years) in Atlanta and 0.007 – 0.02 percent of the time (1 – 4 days over 50 years) in Fort Collins, depending on which scenario is examined. It is important to note, however, that the duration over medium gravel transport would decrease if the channel allowed overbank flow. The duration of gravel transport is increased approximately 300 percent in Atlanta from existing conditions to developed uncontrolled conditions. In Fort Collins, the duration of gravel transport is increased approximately 200 percent.

Average boundary shear stress is approximately the same between existing and developed uncontrolled conditions 90 percent of the time in Atlanta, and 94 percent of the time in Fort Collins. The rest of the time, average boundary shear stress values are 15 to 30 percent higher in developed uncontrolled conditions than they are under existing conditions. The Over Control scenarios result in average boundary shear stresses that are less than existing conditions for approximately 0.02 percent of the time (88 hours over 50 years) in Atlanta and 0.007 percent of the time (31 hours over 50 years) in Fort Collins. Shear stresses between existing conditions and the Over Control scenarios are nearly the same approximately 50 percent of the time in Atlanta, and approximately 20 percent of the time in Fort Collins. The rest of the time, over control shear stresses are greater than existing conditions to varying degrees, with larger shear stresses occurring when the BMP is not implemented. Shear stresses for the Peak Shaving + BMP scenario track closer to existing conditions than the Over Control scenarios, especially in the range of shear stresses that are above the critical shear stress for medium gravel.

Although Figures 4-7 and 4-8 graphically present differences in average boundary shear stress and the duration of time these values occur, it is difficult to use this information to determine the total change in shear stress exerted in the channel over the 50-year duration. As an alternative to examining differences graphically, calculated differences in average boundary shear stress values between each scenario can be examined if the values for each time step are summed over the 50-year period of record. This method is most effective when excess shear is examined. Excess shear can be calculated by subtracting the *critical* shear from the *calculated* shear and then summing the excess shear values for each time step. This method is more useful than examining cumulative raw shear values because excess shear is the amount of shear affecting sediment transport in the channel. Rather than calculating cumulative excess shear differences between scenarios, this study calculated relative differences in sediment transport using sediment transport equations. Results for these analyses are presented in Section 4.3 and 4.4.

Because specific stream power is often thought of as a descriptor of overall conditions and sedimentation processes in stream channels (Bledsoe, 1999), downstream stream power durations are presented in Figures 4-10 and 4-11. It is important to note that this study did not evaluate excess specific stream power relative to sedimentation

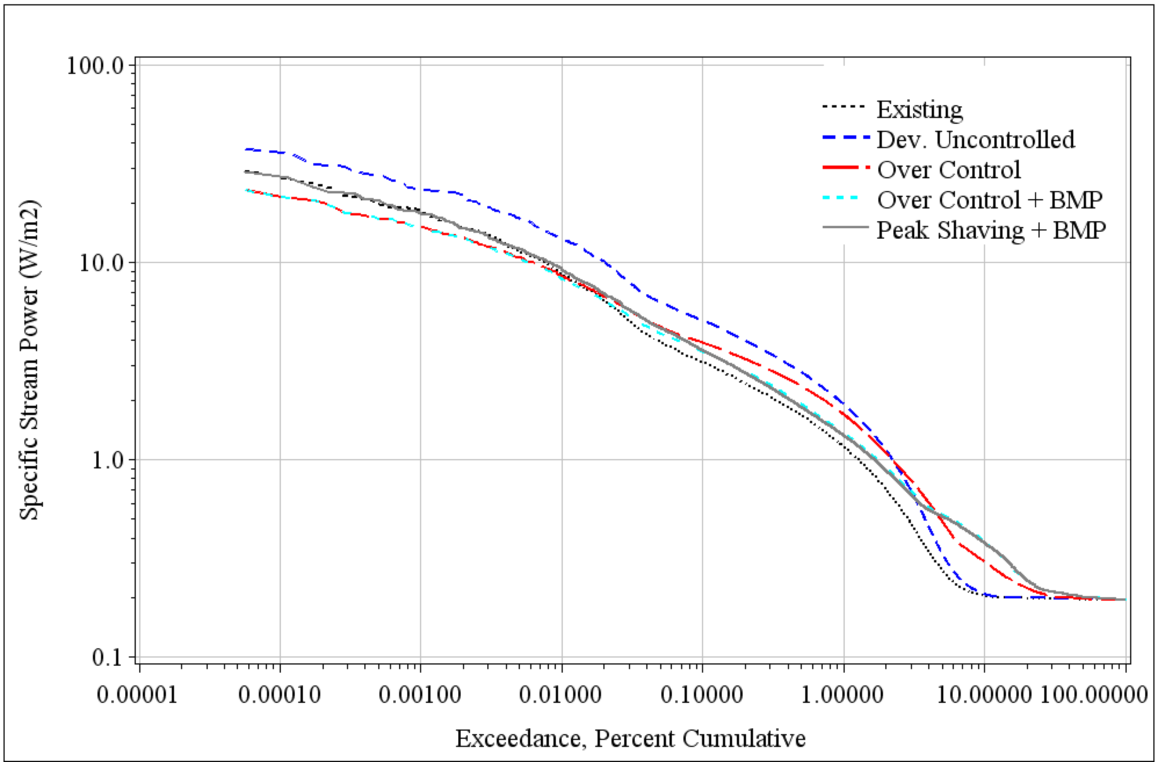

Figure 4-10. Specific stream power, full watershed: Atlanta.

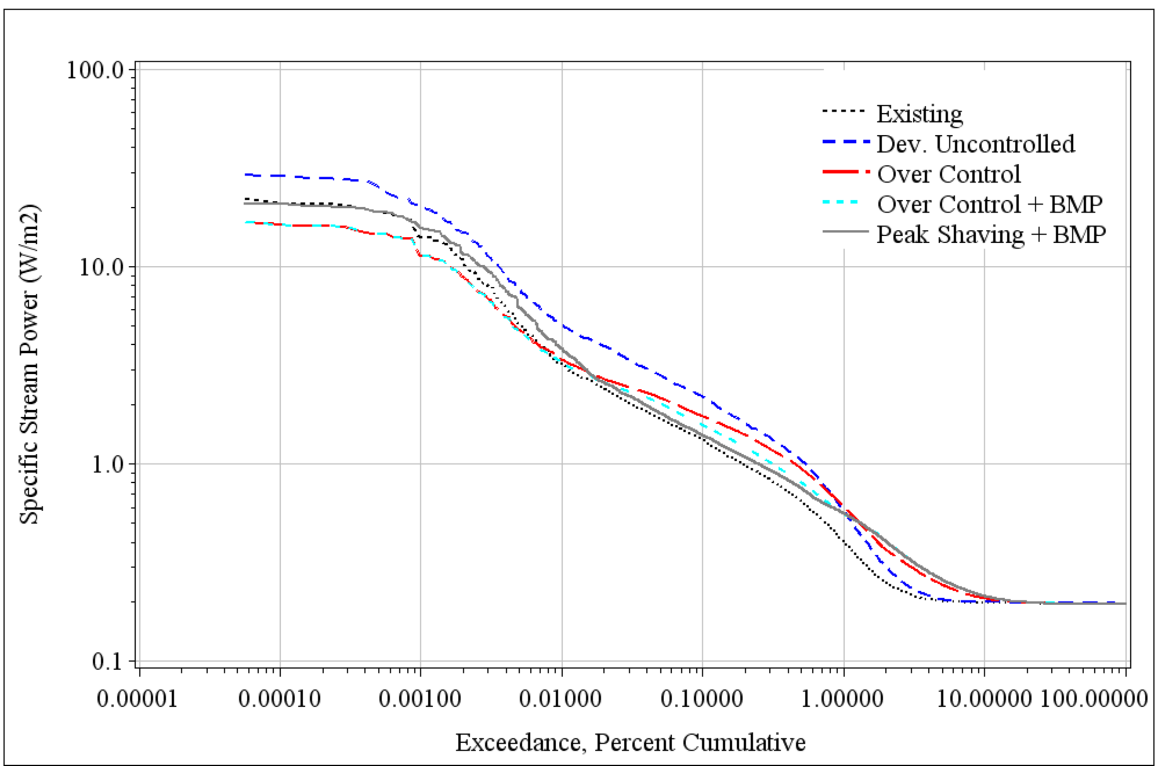

Figure 4-11. Specific stream power, full watershed: Fort Collins.

characteristics; rather, stream power results are presented simply for comparison between scenarios. It can be noted that these results follow the same patterns discussed for average boundary shear stresses. Again, quantitative sediment transport effects of various stormwater controls are presented in sections 4.3 and 4.4.

Shear stress and specific stream power duration curves in log- and semi-log-scale for the developing portion of the watershed only, are given in Appendix B. Downstream semi-log shear stress and specific stream power duration curves are presented in Appendix C.

4.3 Sediment Transport of Medium Sand

The transport potential of medium sand (d_s = 0.25 mm) was evaluated using total load equations suggested by Brownlie, as described in Section Three. Tables 4.4 and 4.5 summarize the mass of medium sand transported over the 50-year continuous simulations conducted for Atlanta and Fort Collins. Results are tabulated in discharge bins that represent peak discharge return intervals from the Existing scenario. The results of the sediment transport analysis are for conditions in the stream channel downstream of the watershed, including discharge from Subareas 1100, 1200, and 1300. These results evaluate which scenarios most closely mimic sediment transport *potential* under existing conditions and do not take into account geomorphic changes that would occur within the stream channel over the 50-year period. For this exercise, only sediment loads transported by flows greater than baseflow are reported. Baseflow is represented by discharges less than 0.0029 cms. The percent of change in total sediment load between existing conditions and each level of postdevelopment stormwater control is presented for each scenario. This evaluation method is similar to the erosion potential indices suggested by

Table 4.4. Sediment load for medium sand: Atlanta.

Discharge Bin (cms)	Peak Discharge Return Interval (years)	Sediment Load Over 50 Years (tons)									
		Existing		Developed Uncontrolled		Overcontrol		Overcontrol + BMP		Peak Shaving + BMP	
$0.07 \geq Q > 0.0029$	> baseflow	223.1	43%	366.8	27%	406.6	49%	359.2	53%	345.8	48%
$0.11 \geq Q > 0.07$	> 0.1	61.1	12%	199.9	15%	148.4	18%	89.6	13%	78.9	11%
$0.17 \geq Q > 0.11$	> 0.25	32.6	6%	189.4	14%	84.3	10%	55.7	8%	64.2	9%
$0.30 \geq Q > 0.17$	> 0.5	50.5	10%	148.6	11%	71.9	9%	60.2	9%	68.4	9%
$0.42 \geq Q > 0.30$	> 1	31.0	6%	67.2	5%	37.7	5%	33.8	5%	39.0	5%
$0.49 \geq Q > 0.42$	> 1.5	22.2	4%	36.3	3%	18.2	2%	15.4	2%	21.1	3%
$1.01 \geq Q > 0.49$	> 2	71.9	14%	202.6	15%	52.6	6%	51.3	8%	78.2	11%
$1.31 \geq Q > 1.01$	> 10	16.7	3%	61.4	4%	6.4	1%	6.4	1%	15.9	2%
$1.61 \geq Q > 1.31$	> 25	6.5	1%	40.7	3%	2.8	0%	2.8	0%	5.6	1%
$1.92 \geq Q > 1.61$	> 50	8.4	2%	24.8	2%	0.0	0%	0.0	0%	8.7	1%
$Q > 1.92$	> 83.6	0.0	0%	31.2	2%	0.0	0%	0.0	0%	0.0	0%
	Sum:	523.9		1369.0		829.1		674.4		725.7	
	% Change from Undeveloped:	0%		161%		58%		29%		39%	

Table 4.5. Sediment load for medium sand: Fort Collins.

Discharge Bin (cms)	Peak Discharge Return Interval (years)	Sediment Load Over 50 Years (tons)									
		Existing		Developed Uncontrolled		Overcontrol		Overcontrol + BMP		Peak Shaving + BMP	
$0.01 \geq Q > 0.0029$	> baseflow	7.1	7%	8.3	3%	15.4	12%	21.2	18%	21.6	15%
$0.03 \geq Q > 0.01$	> 0.1	19.2	18%	34.1	14%	36.2	28%	28.9	25%	27.2	19%
$0.05 \geq Q > 0.03$	> 0.25	8.8	8%	27.2	11%	22.0	17%	14.7	13%	10.2	7%
$0.06 \geq Q > 0.05$	> 0.5	2.6	3%	12.1	5%	7.1	5%	5.7	5%	3.5	2%
$0.08 \geq Q > 0.06$	> 1	3.5	3%	14.8	6%	6.8	5%	5.1	4%	3.5	2%
$0.09 \geq Q > 0.08$	> 1.5	1.1	1%	4.7	2%	1.7	1%	1.3	1%	1.2	1%
$0.64 \geq Q > 0.09$	> 2	27.2	26%	58.3	24%	25.1	19%	23.9	21%	35.7	25%
$1.17 \geq Q > 0.64$	> 10	30.3	29%	38.1	15%	15.5	12%	15.5	13%	38.7	27%
$1.20 \geq Q > 1.17$	> 25	2.2	2%	0.0	0%	0.0	0%	0.0	0%	2.2	2%
$1.26 \geq Q > 1.20$	> 50	2.5	2%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
$Q > 1.26$	> 83.6	0.0	0%	50.1	20%	0.0	0%	0.0	0%	0.0	0%
	Sum:	104.7		247.7		130.0		116.4		143.8	
	% Change from Undeveloped:	0%		136%		24%		11%		37%	

MacRae (1993) and Bledsoe (2002), although it uses the mass of sediment transported rather than sediment transport capacities and compares these values as a percent change rather than an index value. Results in Tables 4.4 and 4.5 demonstrate that for medium sand the Over Control + BMP scenarios yield a total sediment load that is closest to the existing conditions. However, the Over Control + BMP scenarios still create a 29 percent increase in sediment transport in Atlanta and an 11 percent increase in Fort Collins.

Traditional geomorphic estimates (Leopold et al., 1964) indicate that storms at a 1.5- to two-year recurrence interval are responsible for the form of an active channel in humid, temperate climates; however, this estimate is less likely to occur in semi-arid climates. In the Atlanta analysis, at least 88 percent of the total sediment load in each scenario is transported by storms with an Existing peak discharge return interval of two years. From Table 4.4, it can be seen that storms with an Existing peak discharge return interval of less than 0.25 years account for 55 percent or more of the sand transported over the 50-year continuous simulation in all Atlanta scenarios. This indicates that the high duration of low discharge values account for more than half of all sediment transported. This also indicates that in a southeastern climate, smaller discharges may contribute more to erosion in sand bed streams than the traditional 1.5- to two-year storm. Continuous simulations by Bledsoe (1999) also indicate that the use of a single channel-forming discharge may oversimplify the effects of urbanization caused by flow events that are more frequent than the two-year storm. These more frequent events are significant in terms of sediment transport potential.

Results for Fort Collins in Table 4.5 show a different trend because peak discharges are smaller than those for Atlanta, but the size of the material being transported remains the same. Storms with larger return intervals (e.g., two- to 10-year) under existing conditions contribute significantly to the 50-year sediment load, with Existing peak discharge return intervals less than or equal to the 10-year storm contributing 95 percent of the 50-year sediment load in existing conditions. This indicates that the traditional 1.5- to two-year channel-forming estimate (Leopold et al., 1964) may be low for sand bed channels in a semiarid climate with convective storms.

Figures 4-12 and 4-13 show cumulative medium sand load by return intervals associated with discharge bins. From Figure 4-12 it can be seen that at discharges between baseflow and a return interval of 0.1 years, each postdevelopment scenario transports 1.5 to 2 times more sediment than existing conditions in Atlanta. For the Developed, Uncontrolled Atlanta scenario the cumulative sediment load increases at a rate much larger than the Existing scenario. In contrast, the Atlanta BMP scenarios closely follow the pattern of the Existing scenario, aside from the initial increase in sediment at the first discharge bin. Figure 4-13 shows that sediment loads between scenarios are fairly consistent for discharges in the baseflow bin for Fort Collins. For all Fort Collins postdevelopment scenarios, cumulative sediment load increases at a rate greater than that of existing conditions until the 1.5-year return interval, where the scenarios that control the 100-yr to the two-yr pre, cumulative sediment loads start to increase at a rate less than existing conditions.

Figures 4-14 and 4-15 show cumulative percent exceedance frequencies for medium sand, as calculated by the Brownlie equation for the downstream most point in the watershed. These results are displayed as daily transport values, averaged for the 50-year record. These figures show that medium sand is transported at all times. Appendix B presents cumulative percent exceedance frequencies for medium sand in log- and semi-log-scale for discharges from Subarea 1300 only. Semi-log cumulative percent exceedance frequencies for the transport of medium sand by discharges from the full watershed are presented in Appendix C.

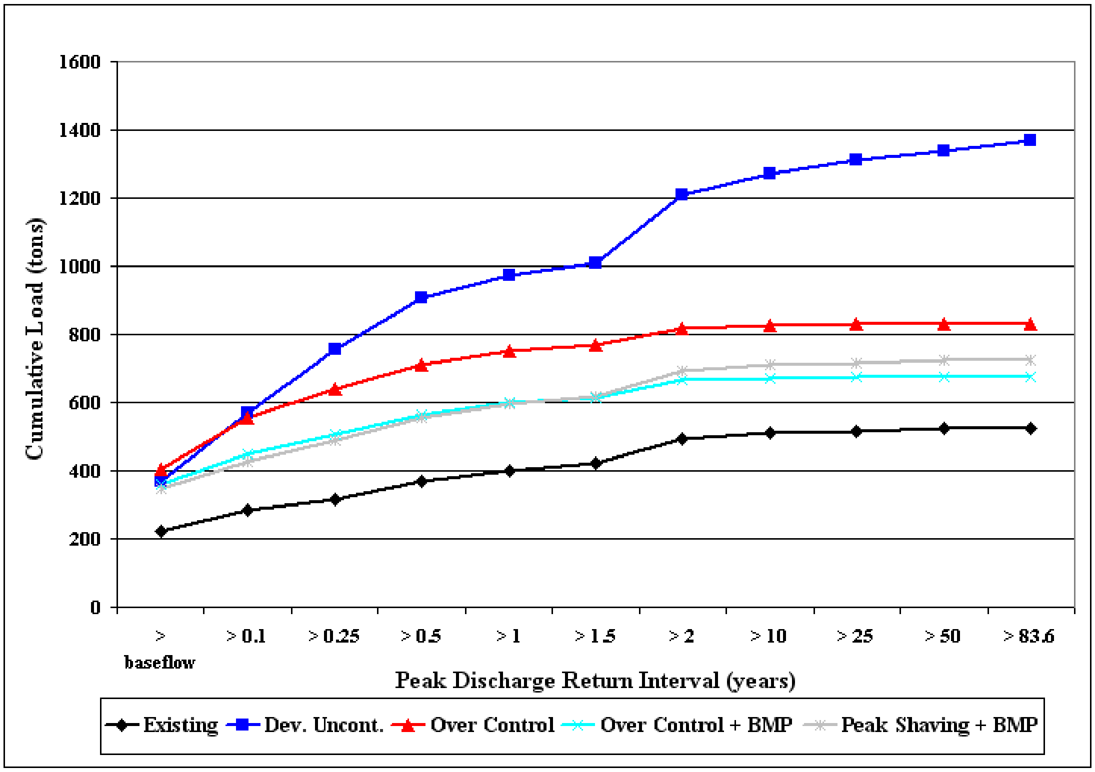

Figure 4-12. Cumulative medium sand load by return interval: Atlanta.

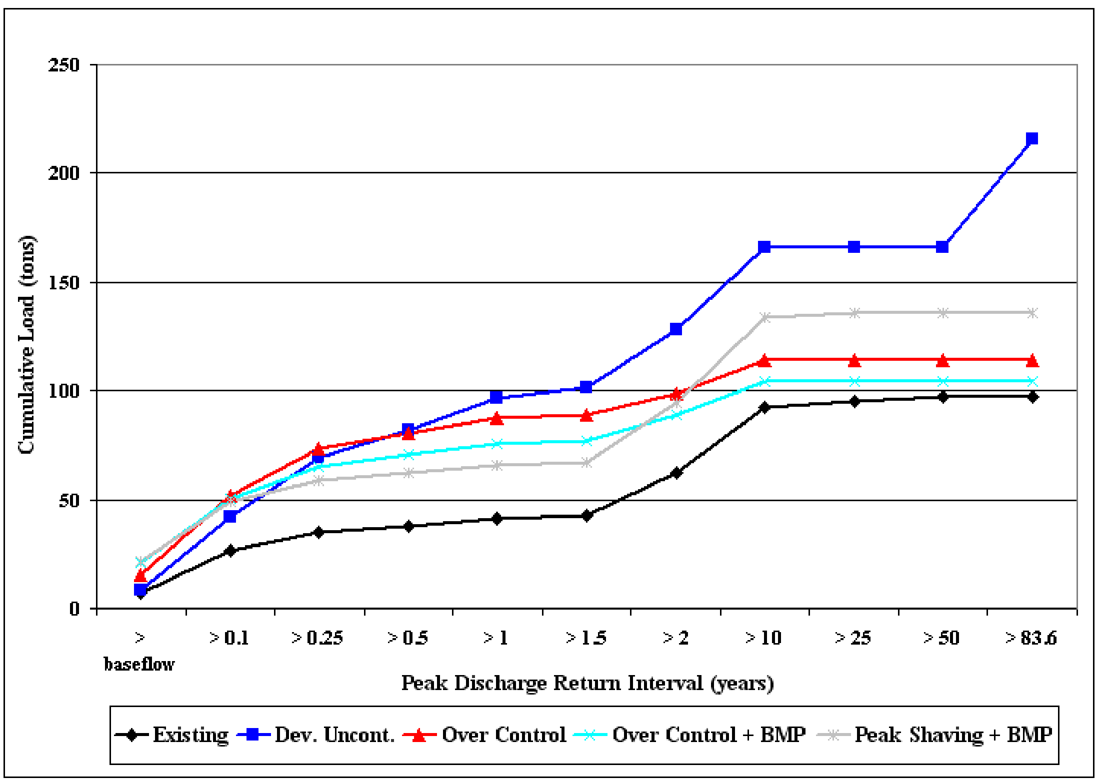

Figure 4-13. Cumulative medium sand load by return interval: Fort Collins.

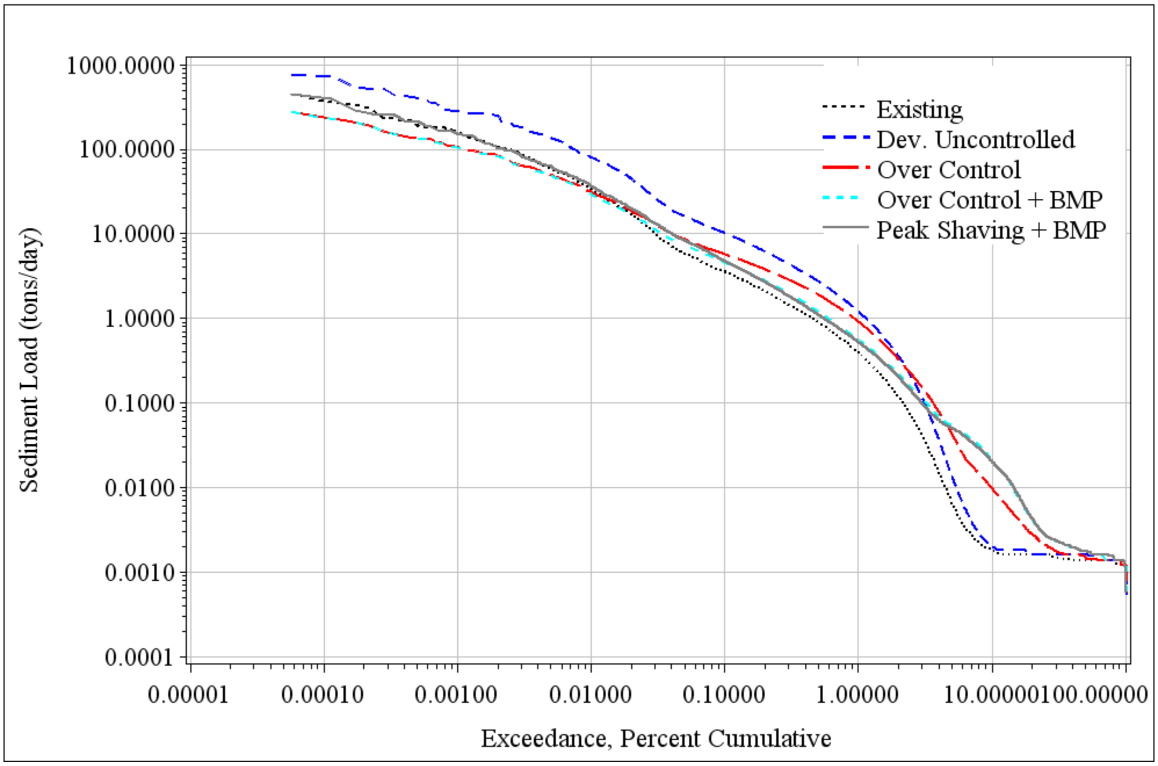

Figure 4-14. Brownlie sediment load for medium sand, full watershed: Atlanta.

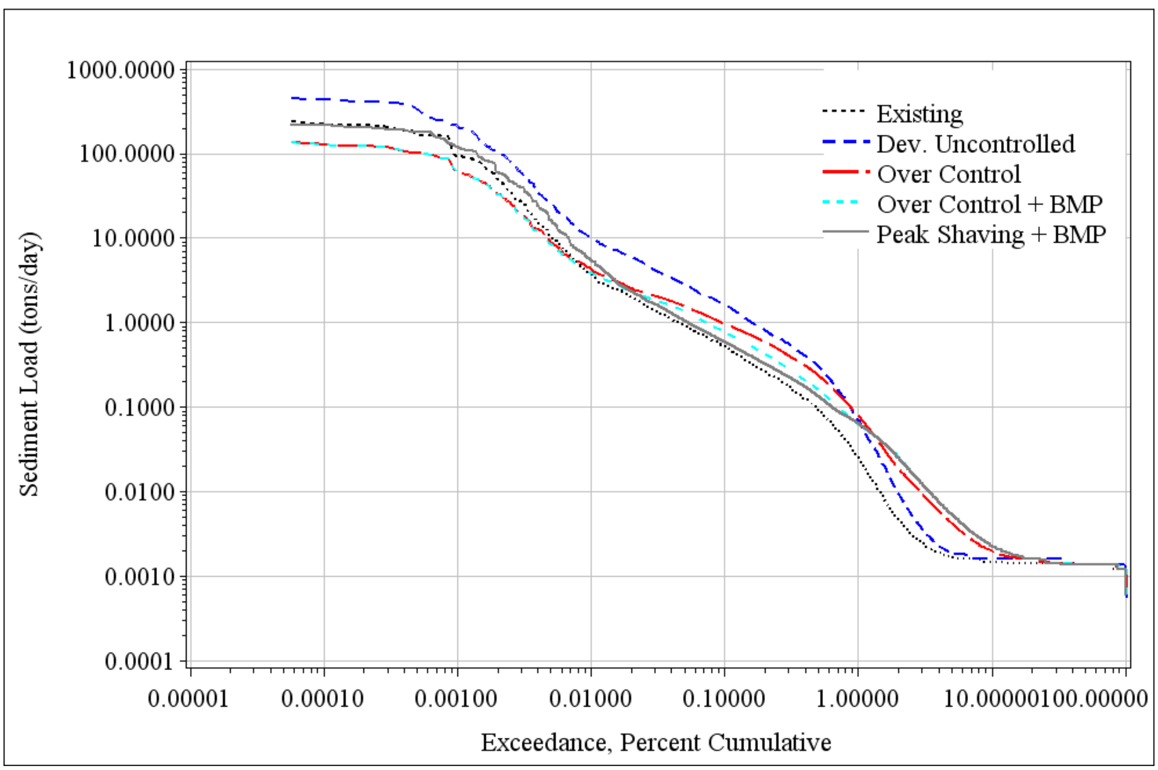

Figure 4-15. Brownlie sediment load for medium sand, full watershed: Fort Collins.

4.4 Sediment Transport of Medium Gravel

The transport potential of medium gravel (d_s = 8 mm) was evaluated using the Meyer-Peter and Müller bedload equation described in Section Three. Tables 4.6 and 4.7 summarize sediment transport potential for medium gravel. Comparison of existing condition sediment loads to the Over Control and Over Control + BMP scenario sediment loads shows that when over control detention is used, sediment transport potential is

Table 4.6. Sediment load for medium gravel: Atlanta.

Discharge Bin (cms)	Peak Discharge Return Interval (years)	Sediment Load Over 50 Years (tons)									
		Existing		Developed Uncontrolled		Overcontrol		Overcontrol + BMP		Peak Shaving + BMP	
0.07 ≥ Q > 0.0029	> baseflow	0.0	0%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
0.11 ≥ Q > 0.07	> 0.1	0.2	0%	0.9	0%	0.4	0%	0.2	0%	0.2	0%
0.17 ≥ Q > 0.11	> 0.25	9.8	4%	60.9	8%	22.4	10%	15.8	8%	19.2	7%
0.30 ≥ Q > 0.17	> 0.5	46.3	19%	125.4	17%	65.2	28%	54.7	26%	62.4	22%
0.42 ≥ Q > 0.30	> 1	36.4	15%	80.0	11%	45.3	19%	40.3	20%	46.3	16%
0.49 ≥ Q > 0.42	> 1.5	27.9	11%	44.4	6%	22.6	10%	19.3	9%	26.4	9%
1.01 ≥ Q > 0.49	> 2	90.3	37%	254.1	34%	66.9	29%	65.3	32%	98.8	34%
1.31 ≥ Q > 1.01	> 10	20.2	8%	71.1	10%	7.6	3%	7.6	4%	18.4	6%
1.61 ≥ Q > 1.31	> 25	6.9	3%	47.0	6%	3.2	1%	3.2	2%	6.9	2%
1.92 ≥ Q > 1.61	> 50	9.2	4%	26.1	4%	0.0	0%	0.0	0%	9.0	3%
Q > 1.92	> 83.6	0.0	0%	30.8	4%	0.0	0%	0.0	0%	0.0	0%
	Sum:	247.3		740.9		233.7		206.4		287.7	
	% Change from Undeveloped:	0%		200%		-5%		-17%		16%	

Table 4.7. Sediment load for medium gravel: Fort Collins.

Discharge Bin (cms)	Peak Discharge Return Interval (years)	Sediment Load Over 50 Years (tons)									
		Existing		Developed Uncontrolled		Overcontrol		Overcontrol + BMP		Peak Shaving + BMP	
0.01 ≥ Q > 0.0029	> baseflow	0.0	0%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
0.03 ≥ Q > 0.01	> 0.1	0.0	0%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
0.05 ≥ Q > 0.03	> 0.25	0.0	0%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
0.06 ≥ Q > 0.05	> 0.5	0.0	0%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
0.08 ≥ Q > 0.06	> 1	0.0	0%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
0.09 ≥ Q > 0.08	> 1.5	0.0	0%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
0.64 ≥ Q > 0.09	> 2	26.2	38%	41.8	29%	21.7	52%	21.1	51%	33.7	40%
1.17 ≥ Q > 0.64	> 10	36.9	54%	45.4	32%	19.9	48%	19.9	49%	47.5	57%
1.20 ≥ Q > 1.17	> 25	2.7	4%	0.0	0%	0.0	0%	0.0	0%	2.7	3%
1.26 ≥ Q > 1.20	> 50	2.9	4%	0.0	0%	0.0	0%	0.0	0%	0.0	0%
Q > 1.26	> 83.6	0.0	0%	55.3	39%	0.0	0%	0.0	0%	0.0	0%
	Sum:	68.7		142.5		41.6		41.0		83.9	
	% Change from Undeveloped:	0%		107%		-39%		-40%		22%	

decreased in gravel bed channels. This over control may lead to streambed aggradation. The Peak Shaving + BMP scenario causes a 16 percent increase in sediment transport potential for Atlanta and a 22 percent increase in sediment transport potential for Fort Collins. For Atlanta, the five percent decrease in cumulative sediment load in the Over Control scenario most closely matches the cumulative sediment load in the Existing scenario. For Fort Collins, the cumulative sediment load in the Peak Shaving + BMP scenario most closely matches the cumulative sediment load in the Existing scenario, with a 22 percent increase in sediment transport.

Table 4.6 shows that a 0.1-year Existing peak discharge return interval is required to initiate sediment transport in a gravel bed channel in Atlanta. The largest percentage of sediment is transported in the two-year Existing peak discharge return interval bin across all scenarios examined. Seventy-eight to 87 percent of all sediment is transported in the 0.5- to two-year Existing peak discharge return intervals.

Table 4.7 shows that sediment transport is not initiated in a gravel bed channel for Fort Collins storms smaller than those with a peak discharge return interval of 2 years for existing conditions. Ninety-two percent of the sediment transport occurs with storms between the two-year and 10-year return interval under existing conditions. One-hundred percent of the sediment transport occurs between the two- and 10-year Existing peak discharge return intervals for the Over Control and Over Control + BMP scenarios and between the two- and 25-years for the Peak Shaving + BMP scenario.

Figures 4-16 and 4-17 show the cumulative medium gravel load by return interval. Figure 4-16 shows that particle motion is not initiated for storms less than the 0.1 year return interval. When any type of stormwater detention is used in Atlanta, the cumulative sediment load follows closely the loads estimated for existing conditions. Figure 4-16 also shows that over control in Atlanta allows less sediment load than the Existing scenario. This over control may potentially lead to aggradation of the stream channel. Figure 4-17 shows that motion of gravel is not initiated in the Fort Collins analysis for storms less than a 1.5 year return interval. Again, the over control detention restricts sediment load to below existing levels.

Figures 4-18 and 4-19 show cumulative percent exceedance frequencies for medium gravel downstream of the entire watershed, as calculated by the Meyer-Peter and Müller bedload equation. These figures show that medium gravel is transported no more than 0.2 percent of the time in Atlanta and 0.02 percent of the time in Fort Collins. Appendix B presents cumulative percent exceedance frequencies for medium gravel in log- and semi-log-scale for discharge from Subarea 1300. Semi-log cumulative percent exceedance frequencies for the transport of medium gravel by discharges from the full watershed are presented in Appendix C.

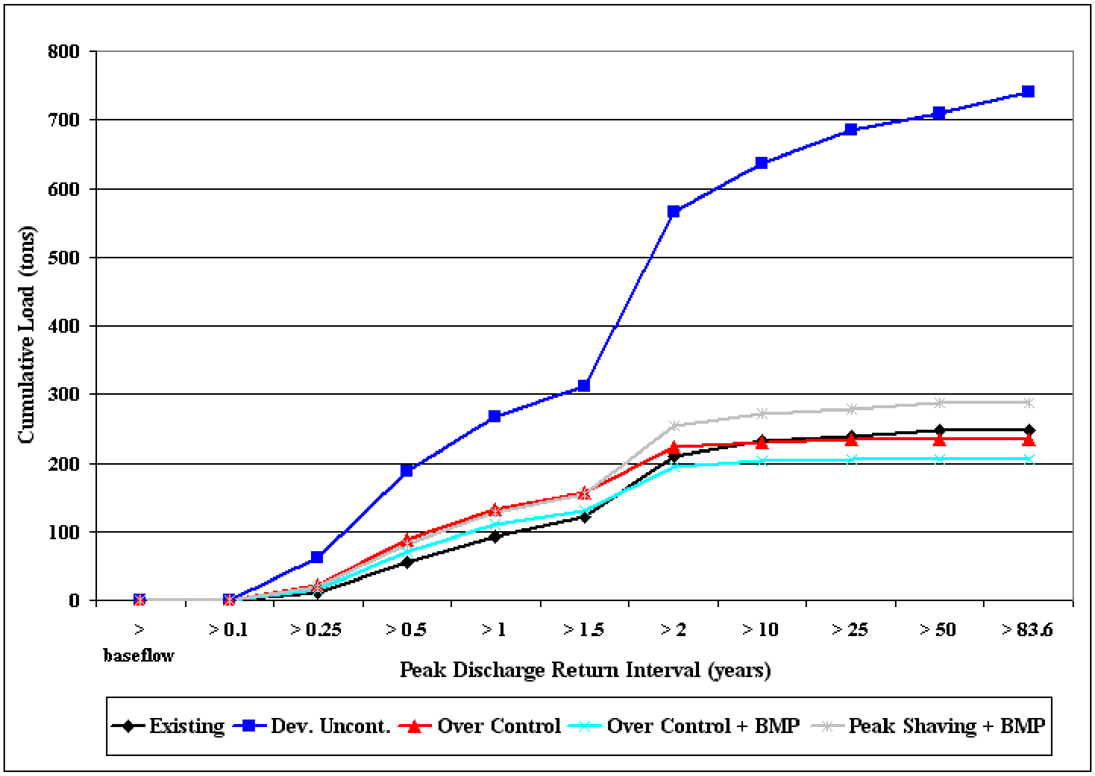

Figure 4-16. Cumulative medium gravel load by return interval: Atlanta.

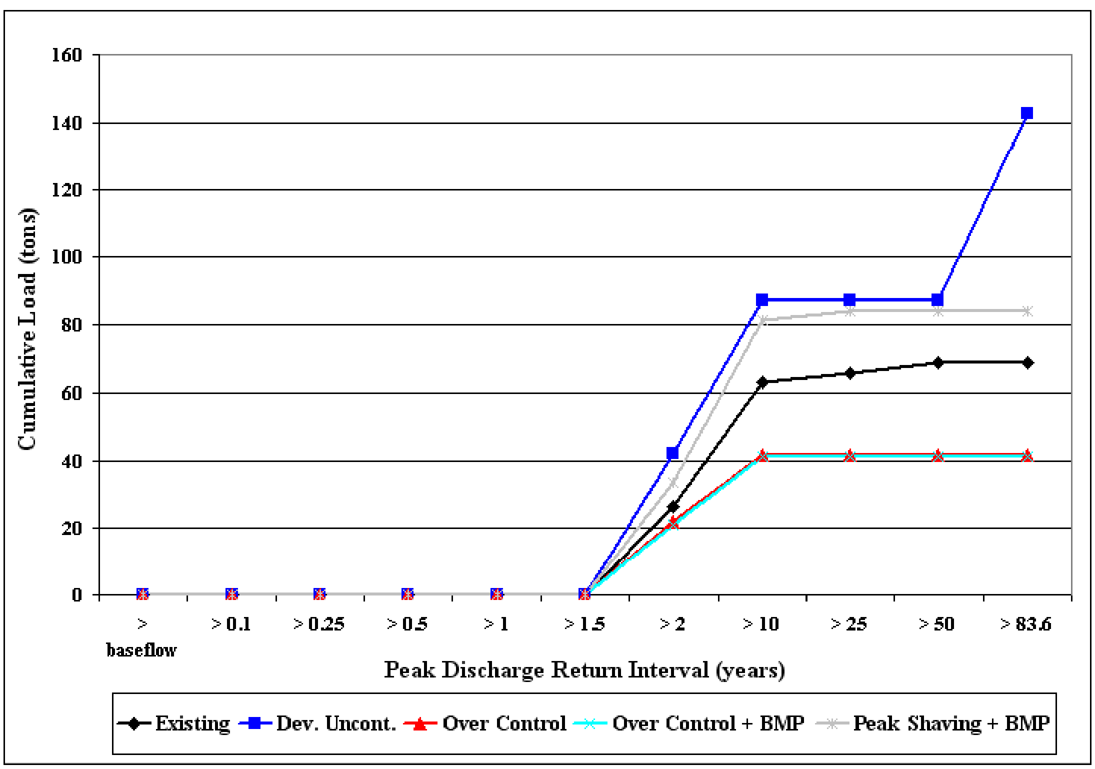

Figure 4-17. Cumulative medium gravel load by return interval: Fort Collins.

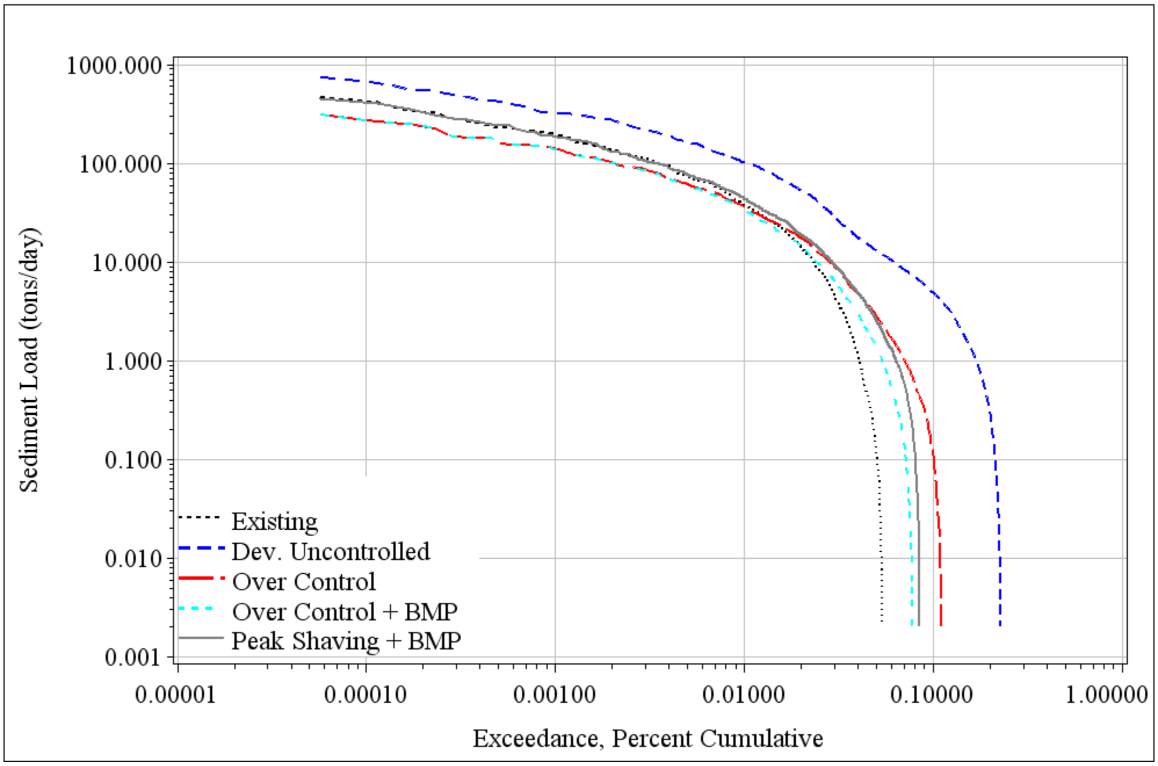

Figure 4-18. MPM sediment load for medium gravel, full watershed: Atlanta.

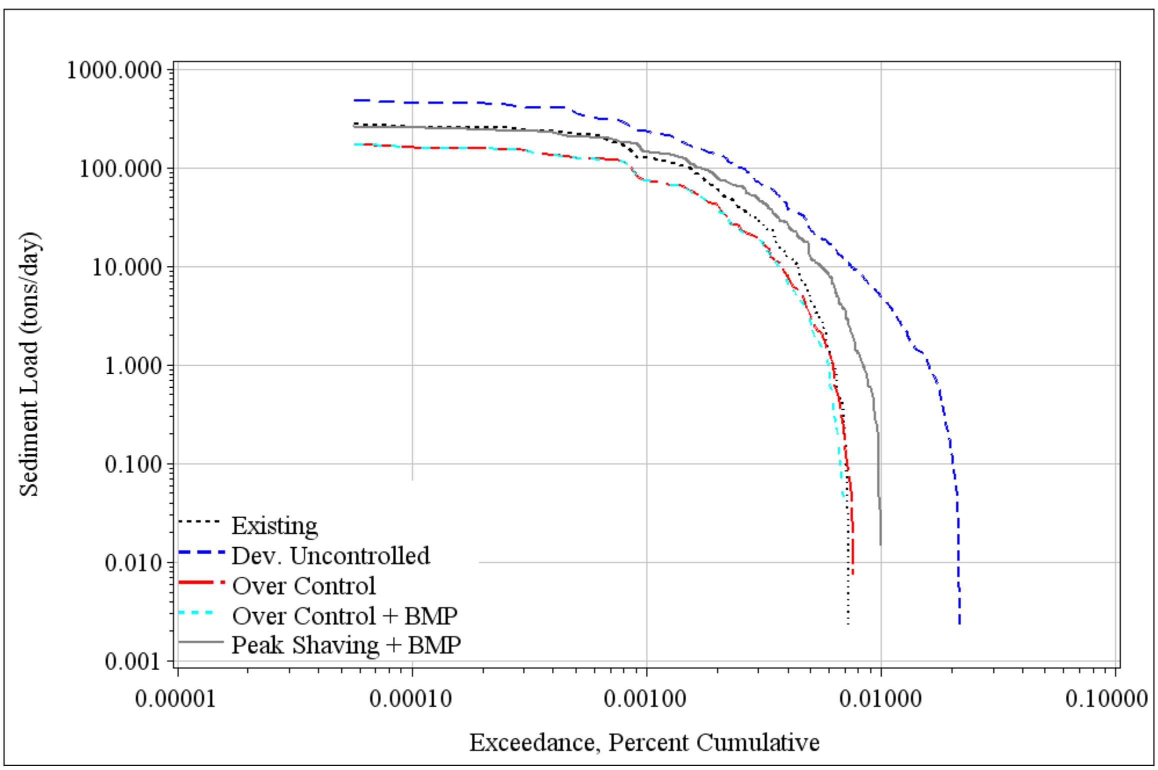

Figure 4-19. MPM sediment load for medium gravel, full watershed: Fort Collins.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The results in Section 4 suggest that streambed material and climate are both important considerations in estimating the potential response of a stream to urbanization and stormwater management scenarios. A "one size fits all" solution to stormwater control is not adequate for the protection of urban streams. In the sensitivity analysis performed in this study, cumulative sediment loads from the Over Control + BMP stormwater management scenarios most closely match existing conditions for streambeds consisting of sand. For gravel streambeds and banks, one stormwater management scenario was not optimal for both the Atlanta and Fort Collins climates. In this analysis, cumulative sediment loads from the Atlanta Over Control scenario most closely matched existing conditions. For the Fort Collins analysis, cumulative loads generated using the Peak Shaving + BMP scenario most closely matched cumulative loads from existing conditions.

Under existing conditions in this study, 76 percent of the total cumulative sand bed load was transported by storms of less than the one-year return interval in Atlanta, but in Fort Collins, only 39 percent of the total cumulative load was transported by storms of less than the one-year return interval. When watershed development occurred uncontrolled, increased peak discharges in Atlanta caused the percent of total cumulative load transported by storms less than a one-year return interval to decrease. The percent of total cumulative sand load transported by storms less than a one-year return interval remained the same for Fort Collins. Conversely, when stormwater controls were applied, the percentage of cumulative sand load transported by storms less than the one-year return interval increased due to lengthened durations of mid- to low-range flows. Cumulative loads of sand transported by discharges less than the one-year return interval under the three stormwater control scenarios ranged from 82 to 90 percent in Atlanta and 46 to 67 percent in Fort Collins. If overbank flow were allowed, it is likely that the percentage of sediment transports by storms of less than a one-year return interval would increase. As was found by Bledsoe (2002), from a geomorphic perspective, the potential impacts of urbanization may be significant for sand bed channels due to stormwater practices that increase flow durations.

In Atlanta, over half of the total cumulative gravel loads were transported by storms in the one- to two-year recurrence interval range. However, in Fort Collins, motion of gravel was not initiated by storms less than a two-year peak discharge return interval, and in existing conditions two-thirds of the sediment was transported by storms with a peak discharge greater than a 10-year return interval. It is important to note, however, that the channel modeled in this analysis did not allow overbank flow and in non-entrenched systems, storms greater than a one- to two-year return interval are likely to flow overbank, preventing sediment transport from increasing significantly past the bankfull depth. Although important from a water quality perspective, this sensitivity analysis also showed that control of the WQCV or the two-year storm had no effect on gravel bed transport in Fort Collins. This sensitivity analysis also showed that over control of stormwater discharges to coarse bed streams decreased sediment transport. Decreases in sediment transport may potentially result in channel aggradation.

Results from the analysis performed in this study indicate that coarse bed or armored channels are not as sensitive to changes in the duration of low flows as are sand bed channels. This study also demonstrates the importance of regional analysis for determining stormwater criteria. While it is not feasible for continuous modeling to be performed on a development-by-development basis, these methods can be applied to develop stormwater control criteria at the local or regional level, based on local precipitation, hydrologic and hydrogeomorphic data. Although not popular in practice when compared to event-based simulation, it is critical that regional or local stormwater control criteria be developed using continuous simulations, or some other methodology that includes analysis of flow durations.

Further study needs to take place to expand the research described in this thesis. This study evaluated the sensitivity of sediment transport to various stormwater controls in two climates, for two types of stream bed materials In the future, the effects of stormwater controls should be evaluated for watersheds with different hydrologic characteristics including varied size, shape, slope, infiltration parameters and additional climate areas such as those with frequent, low-intensity storms such as those found in the Pacific Northwest. Additional types of stormwater controls such as low impact development (LID), infiltration basins, and peak shaving without control of the WQCV should also be evaluated. Non-entrenched receiving streams that allow overbank flows need to be evaluated, and additional streambed materials need to be evaluated. Lastly, methods to simulate the transport of noncohesive bed materials such as clays should be developed and evaluated.

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APPENDIX A: ADDITIONAL WATERSHED DATA

Table A.1. Design Storm Hyetographs.

Time (min)	Depth (mm)			
	2-Year		100-Year	
	Atlanta	Fort Collins	Atlanta	Fort Collins
0	0.00	0.00	0.00	0.00
5	0.76	0.61	0.77	2.12
10	1.51	0.70	2.32	2.41
15	3.18	0.80	3.56	2.82
20	6.06	1.36	6.20	4.72
25	9.46	1.71	10.85	6.01
30	5.30	3.32	19.37	11.62
35	2.38	6.03	10.85	21.06
40	1.89	2.50	6.20	8.72
45	1.14	1.50	4.80	5.25
50	1.14	0.89	3.87	3.09
55	1.14	0.74	3.10	2.58
60	1.14	0.64	3.10	2.24
65	1.14	0.42	3.10	2.12
70	0.76	0.40	1.55	2.01
75	0.76	0.38	1.55	1.93
80	0.76	0.36	0.93	1.84
85	0.76	0.36	0.93	1.78
90	0.76	0.34	0.93	1.71
95	0.76	0.32	0.93	1.65
100	0.76	0.32	0.93	1.59
105	0.76	0.30	0.93	1.55
110	0.76	0.30	0.93	1.50
115	0.38	0.28	0.93	1.46
120	0.38	0.28	0.93	1.42
125	0.00	0.00	0.00	0.00
Total:	43.79	24.85	89.56	93.20

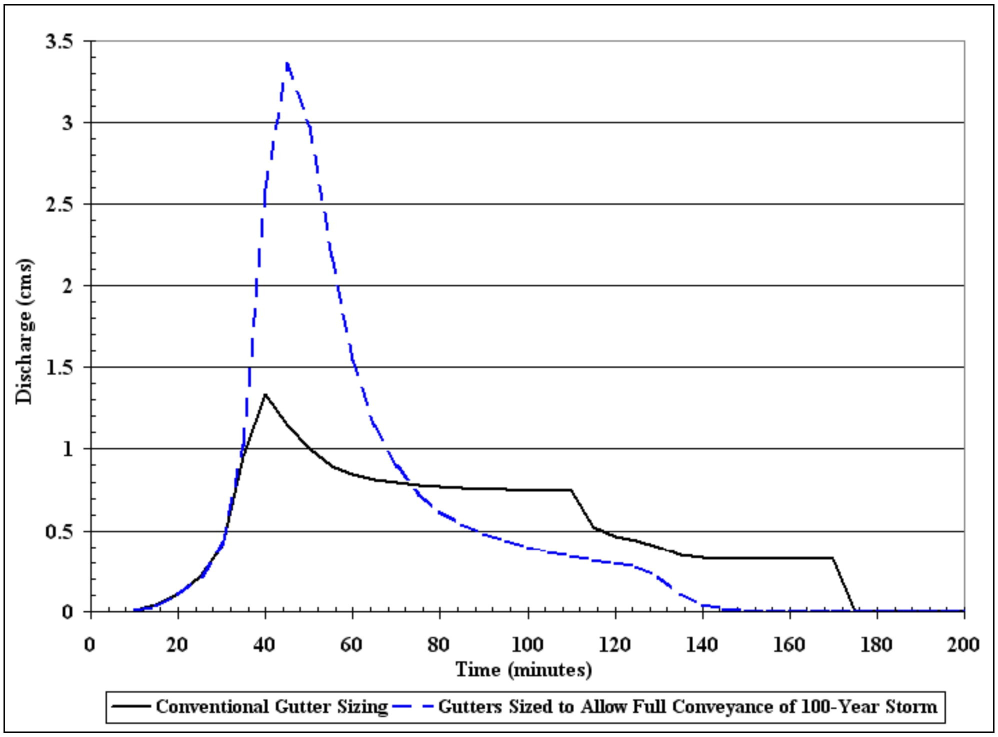

Figure A-1. 100-Year Design Storm Outflow Hydrograph: Fort Collins.

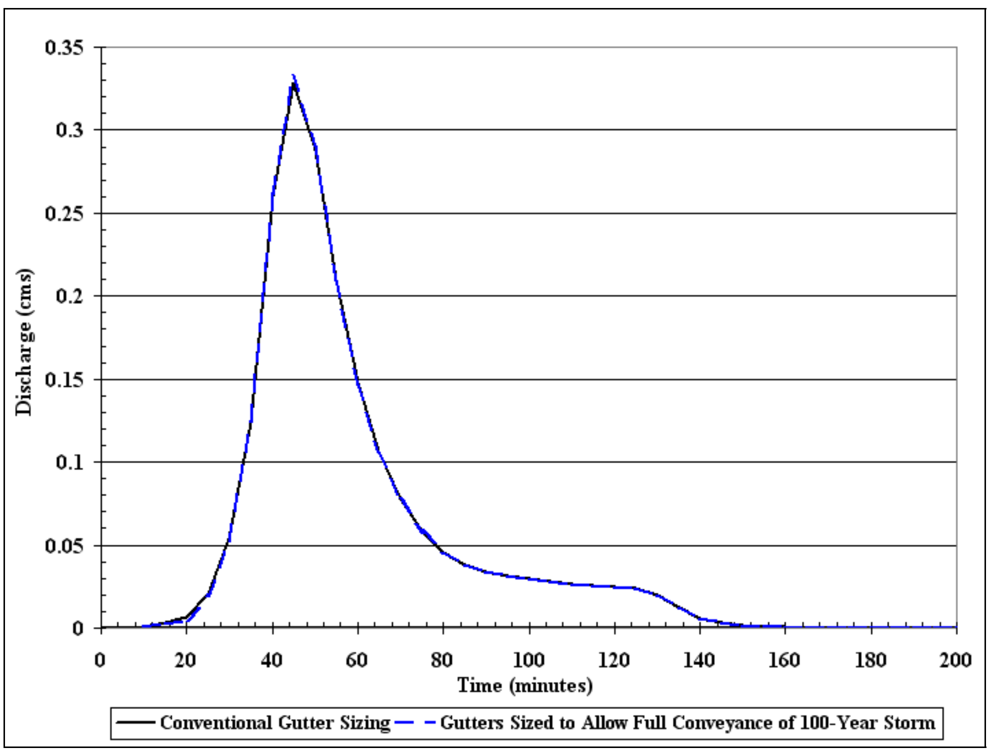

Figure A-2. 2-Year Design Storm Outflow Hydrograph: Fort Collins.

APPENDIX B: ADDITIONAL RESULTS DATA - DEVELOPING WATERSHED

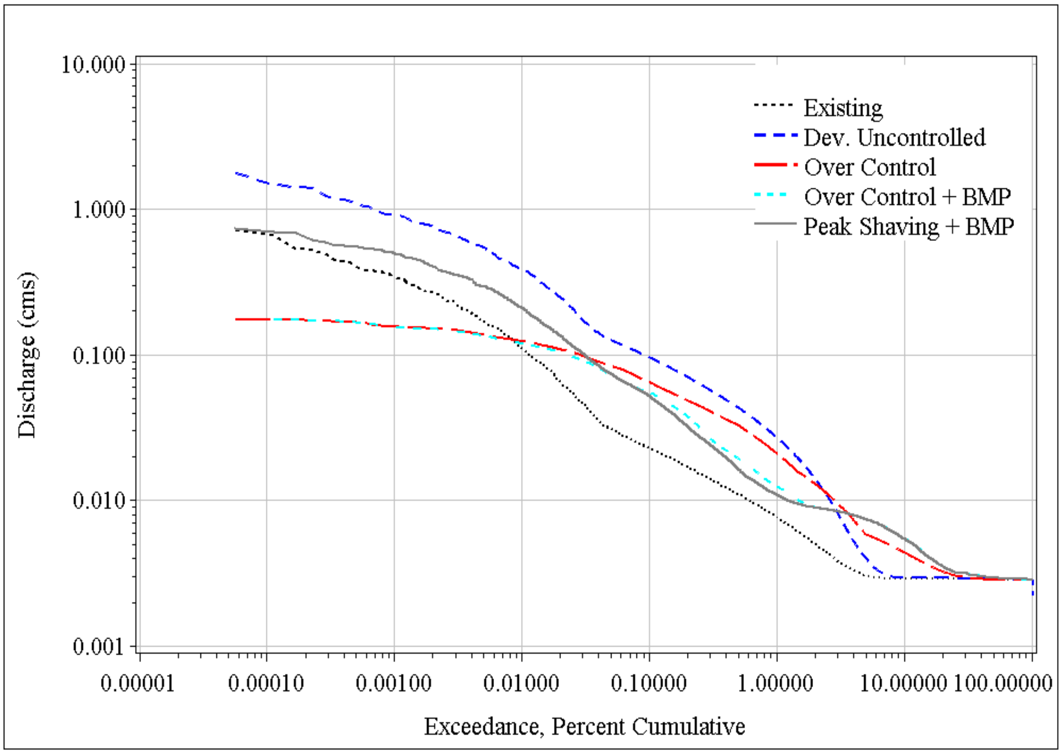

Figure B-1. Discharge Duration: Atlanta, Subarea 1300.

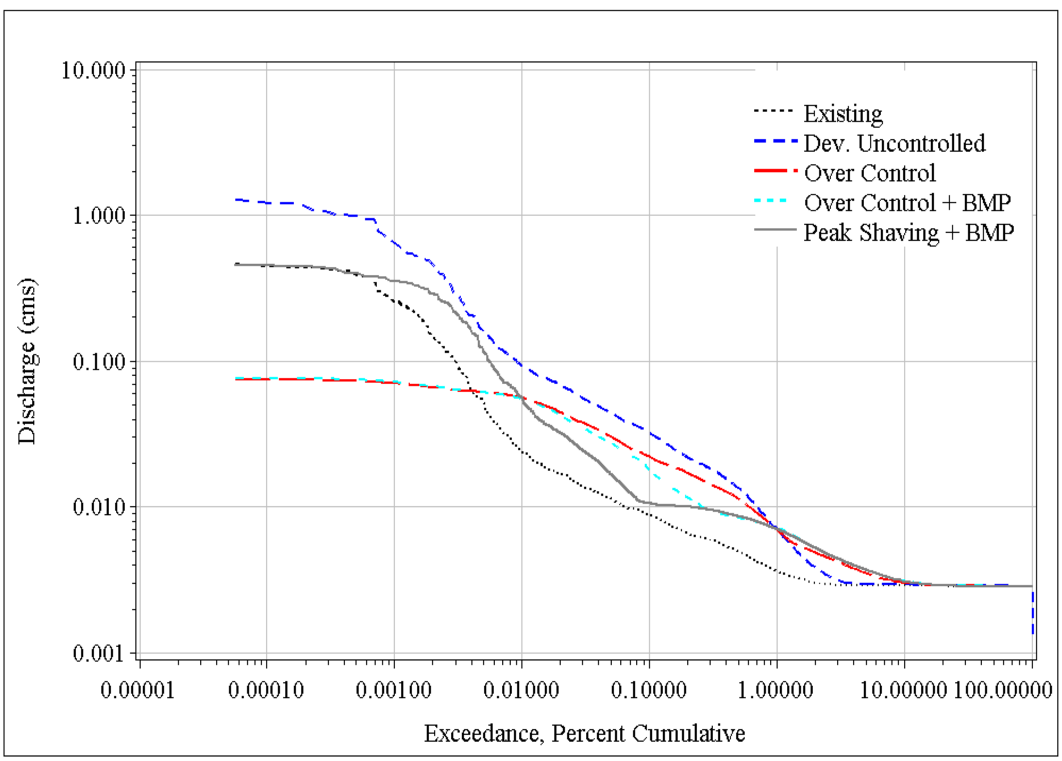

Figure B-2. Discharge Duration: Fort Collins, Subarea 1300.

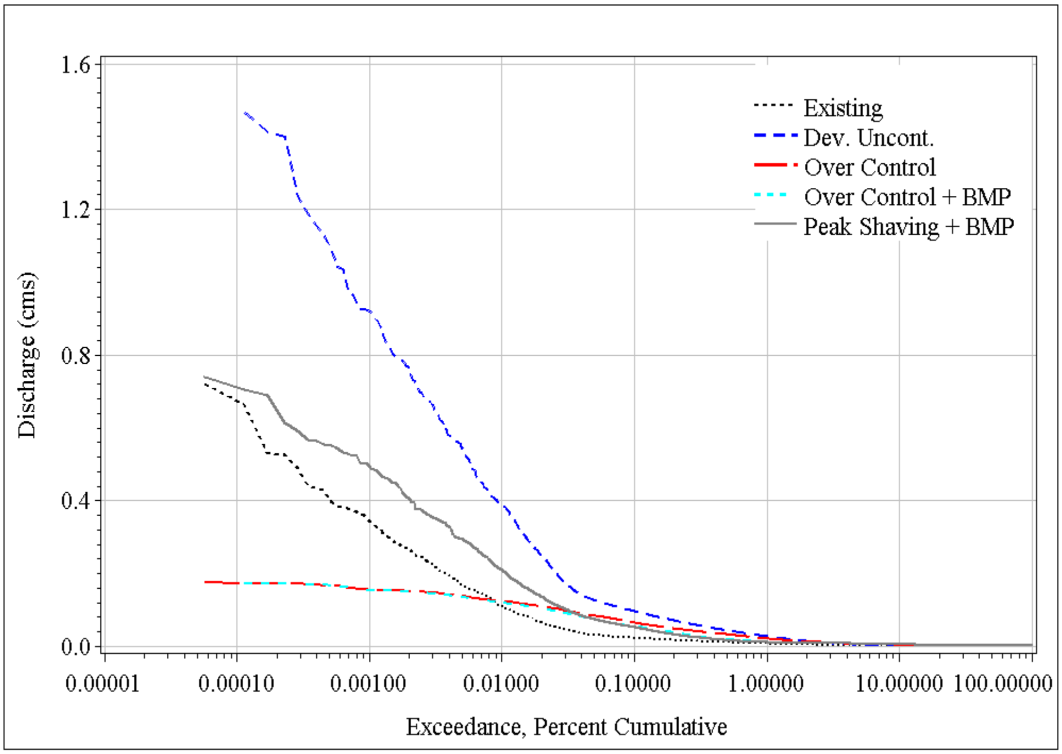

Figure B-3. Discharge Duration: Atlanta, Subarea 1300, Semi-log.

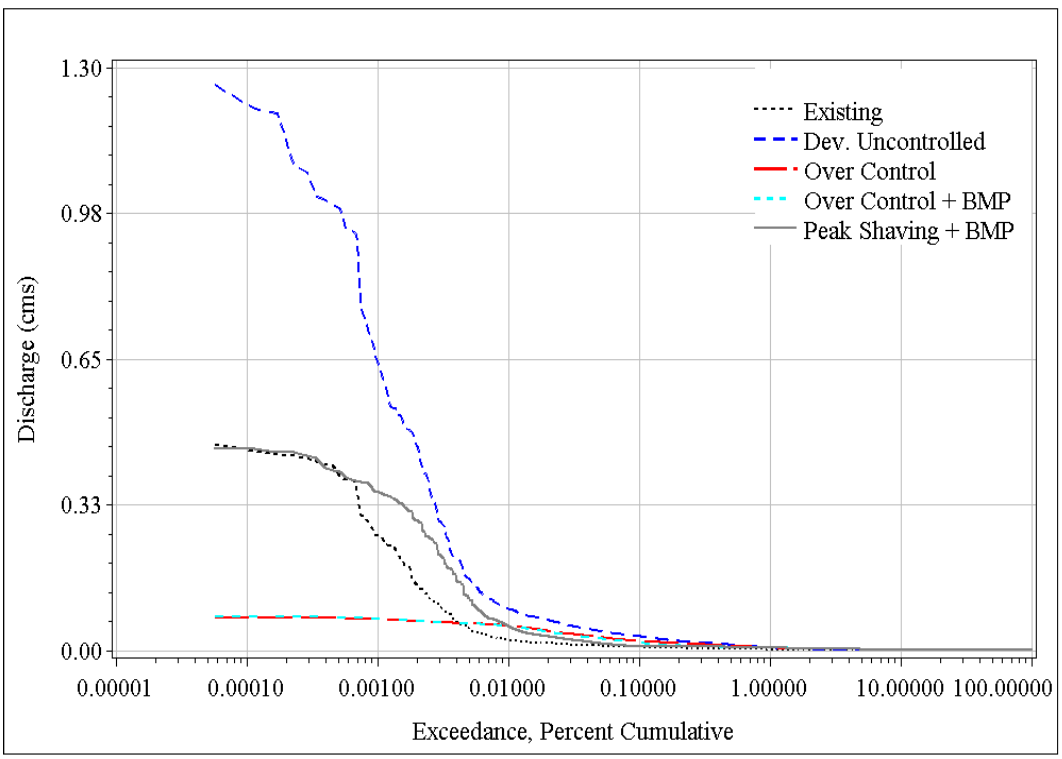

Figure B-4. Discharge Duration: Fort Collins, Subarea 1300, Semi-log.

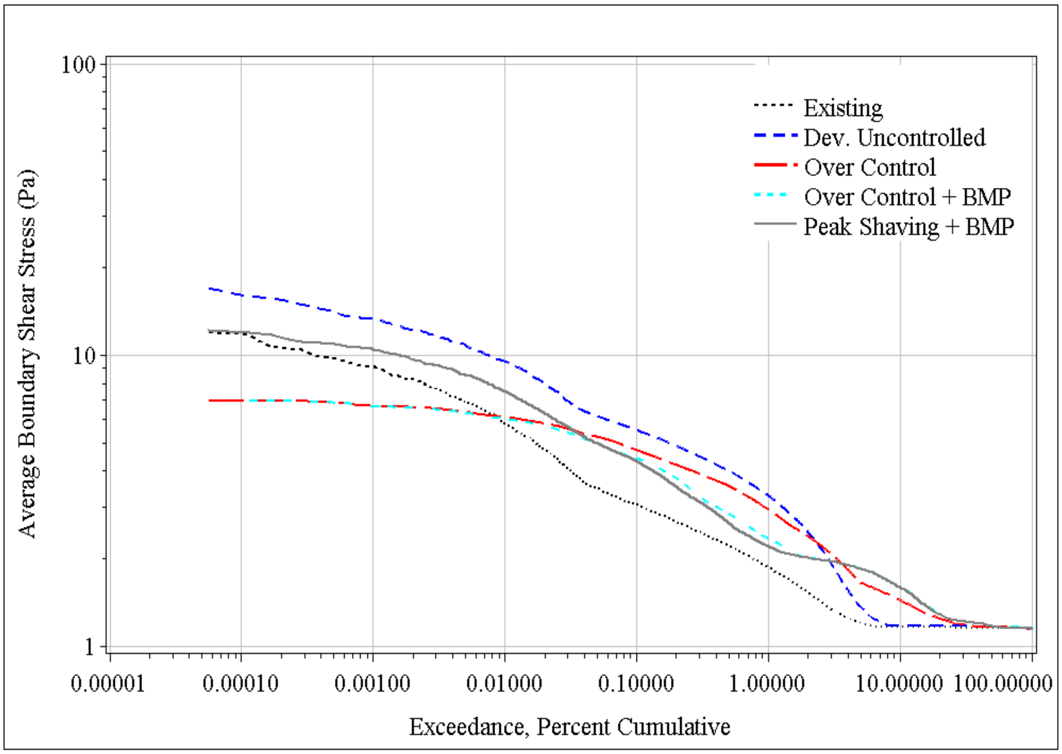

Figure B-5. Average Boundary Shear Stress: Atlanta, Subarea 1300.

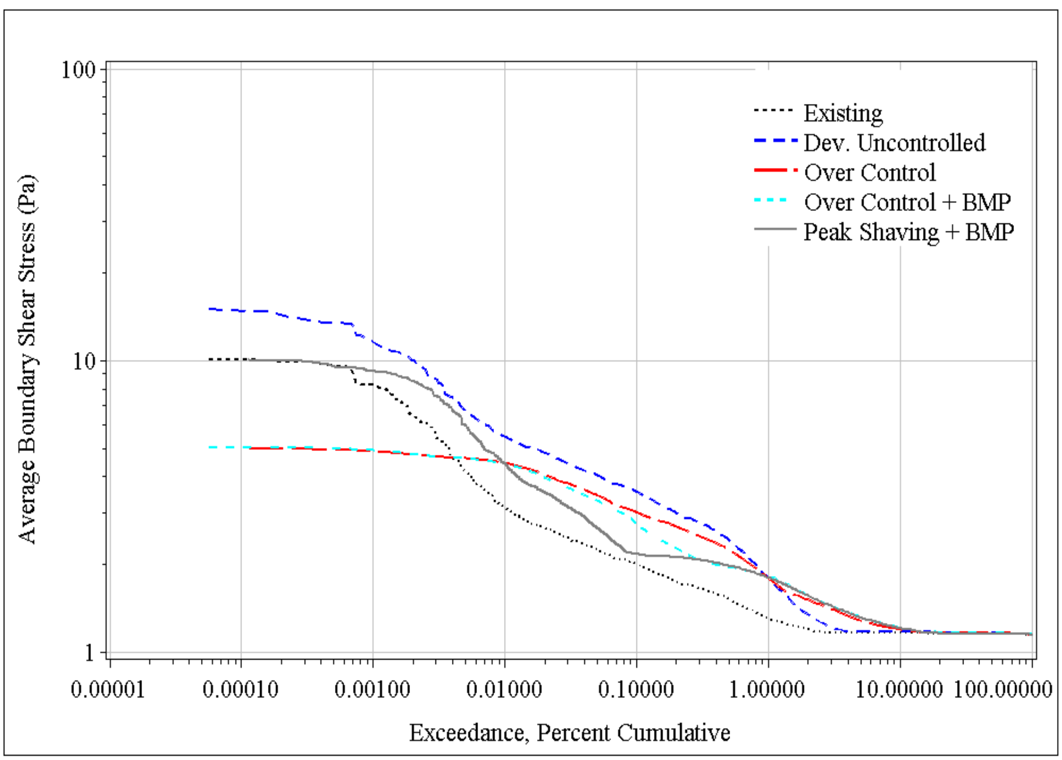

Figure B-6. Average Boundary Shear Stress: Fort Collins, Subarea 1300.

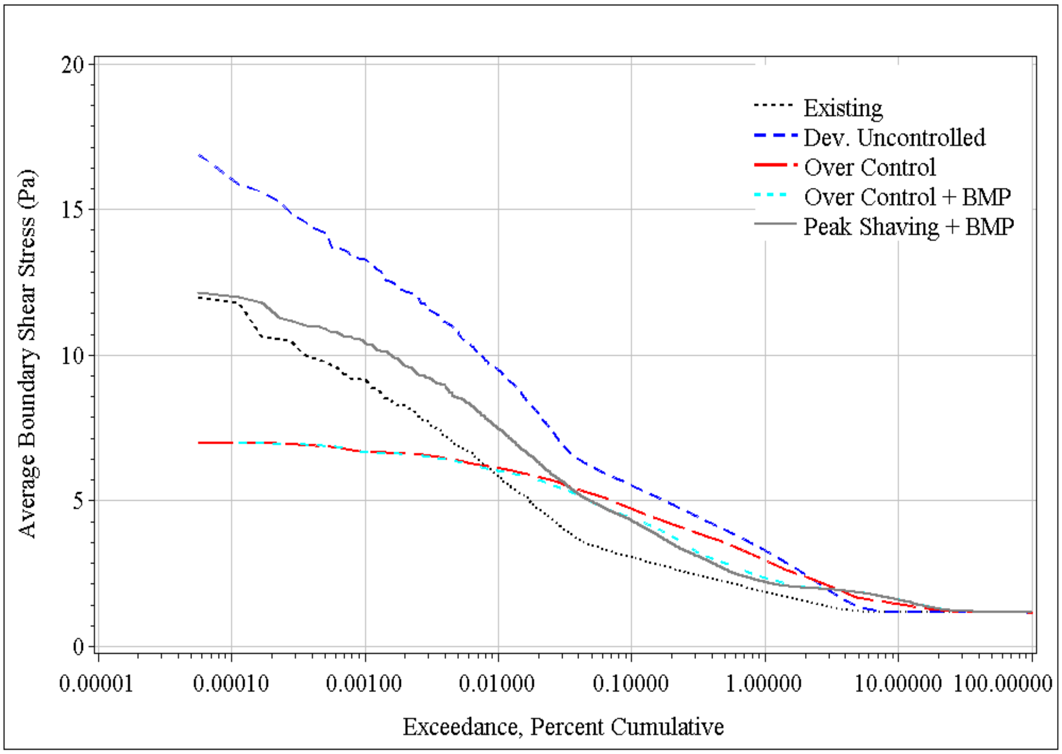

Figure B-7. Average Boundary Shear Stress: Atlanta, Subarea 1300, Semi-log.

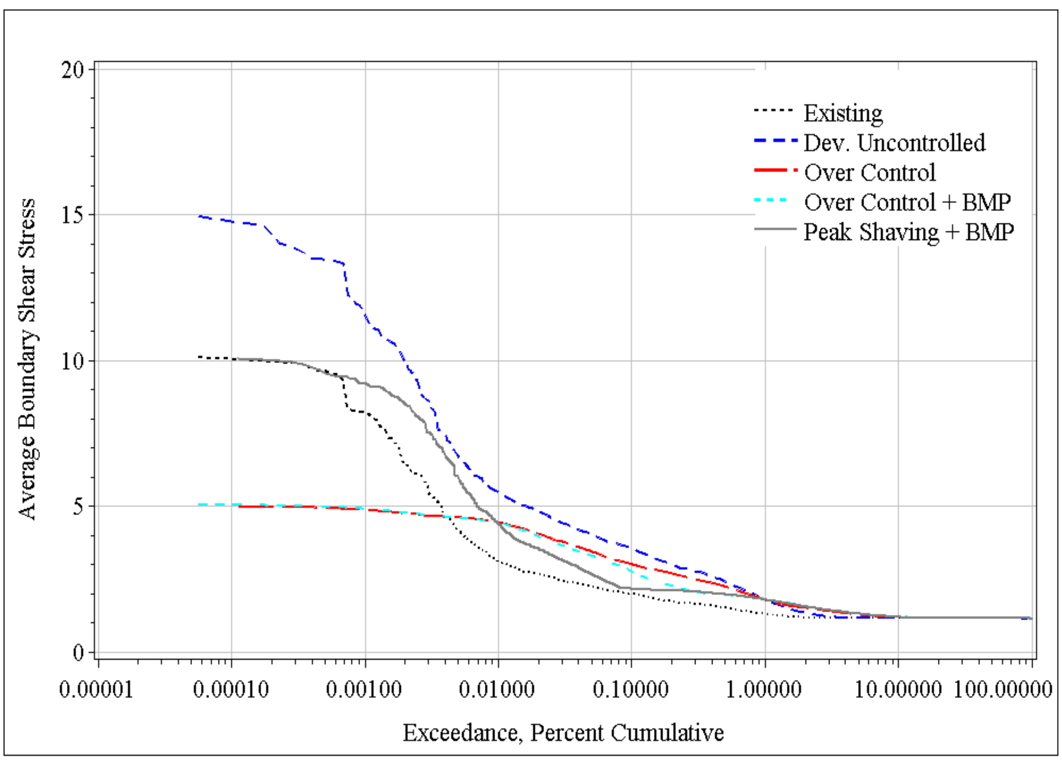

Figure B-8. Average Boundary Shear Stress: Fort Collins, Subarea 1300, Semi-log.

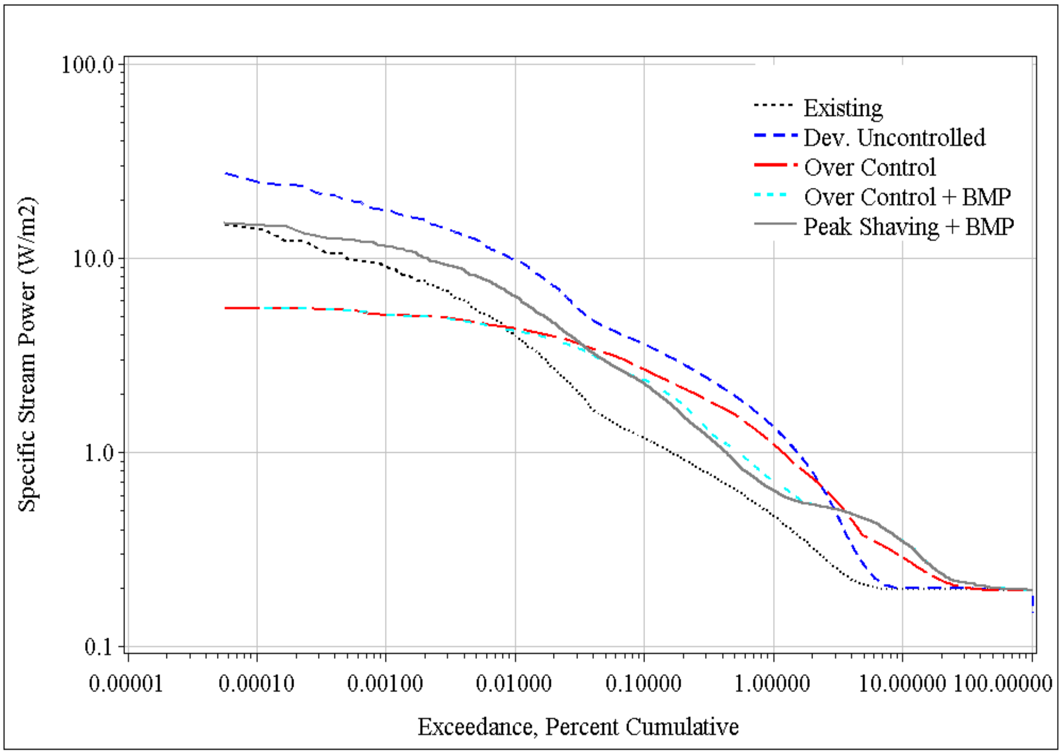

Figure B-9. Specific Stream Power: Atlanta, Subarea 1300.

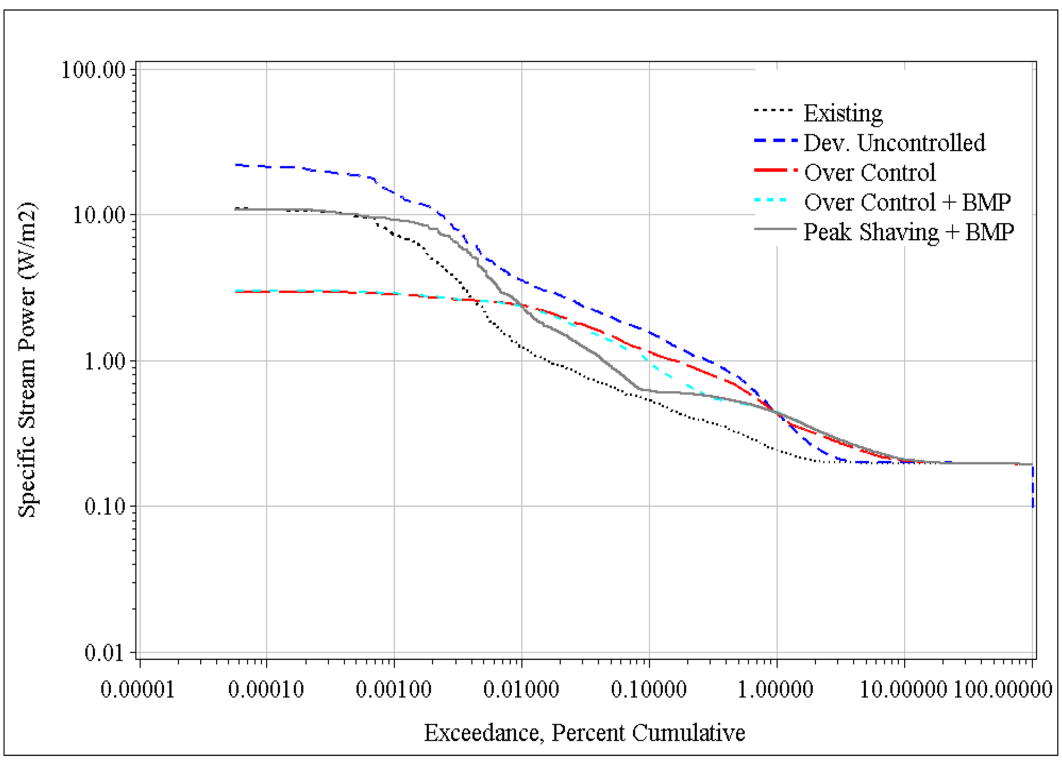

Figure B-10. Specific Stream Power: Fort Collins, Subarea 1300.

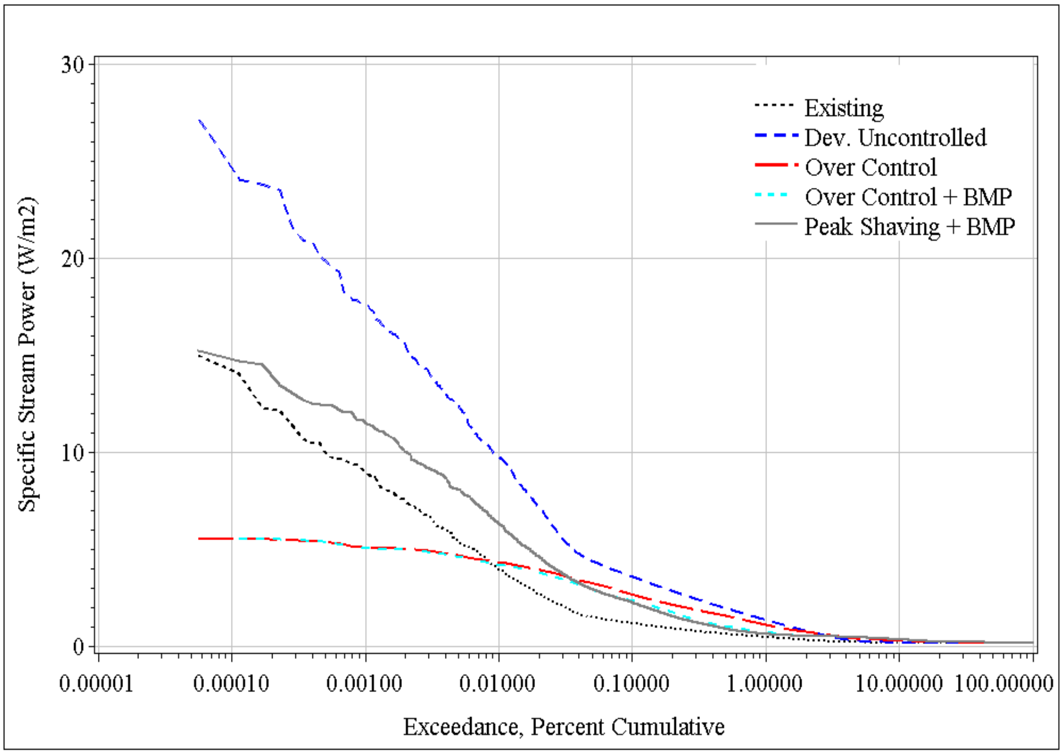

Figure B-11. Specific Stream Power: Atlanta, Subarea 1300, Semi-log.

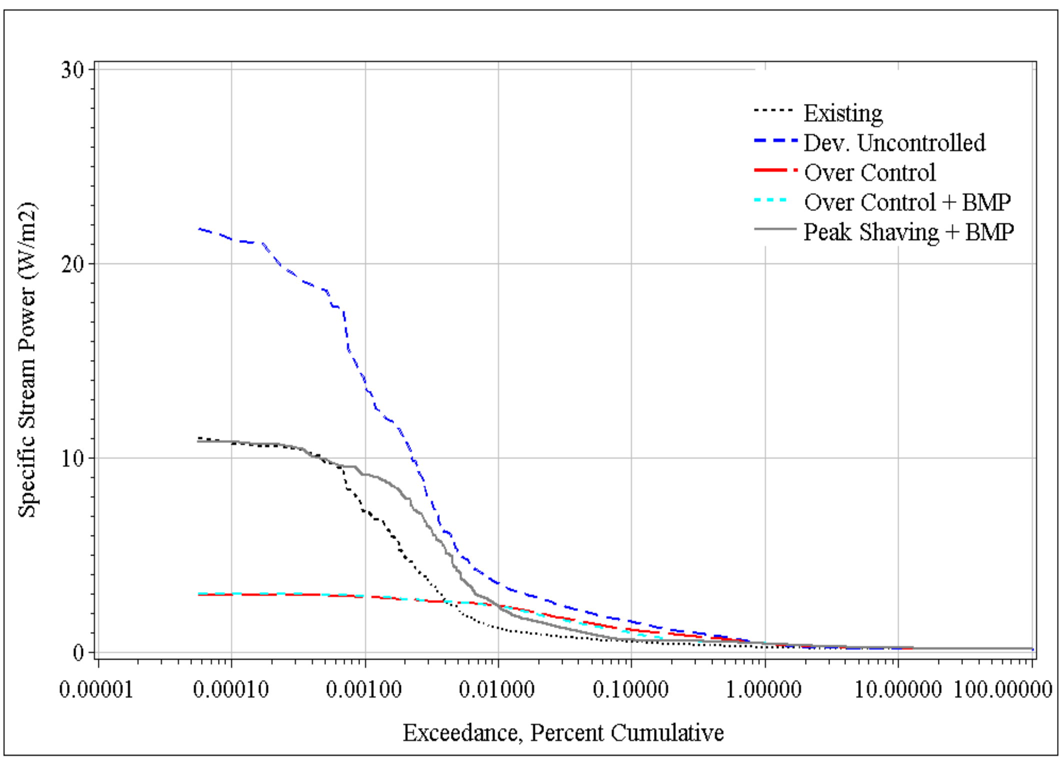

Figure B-12. Specific Stream Power: Fort Collins, Subarea 1300, Semi-log.

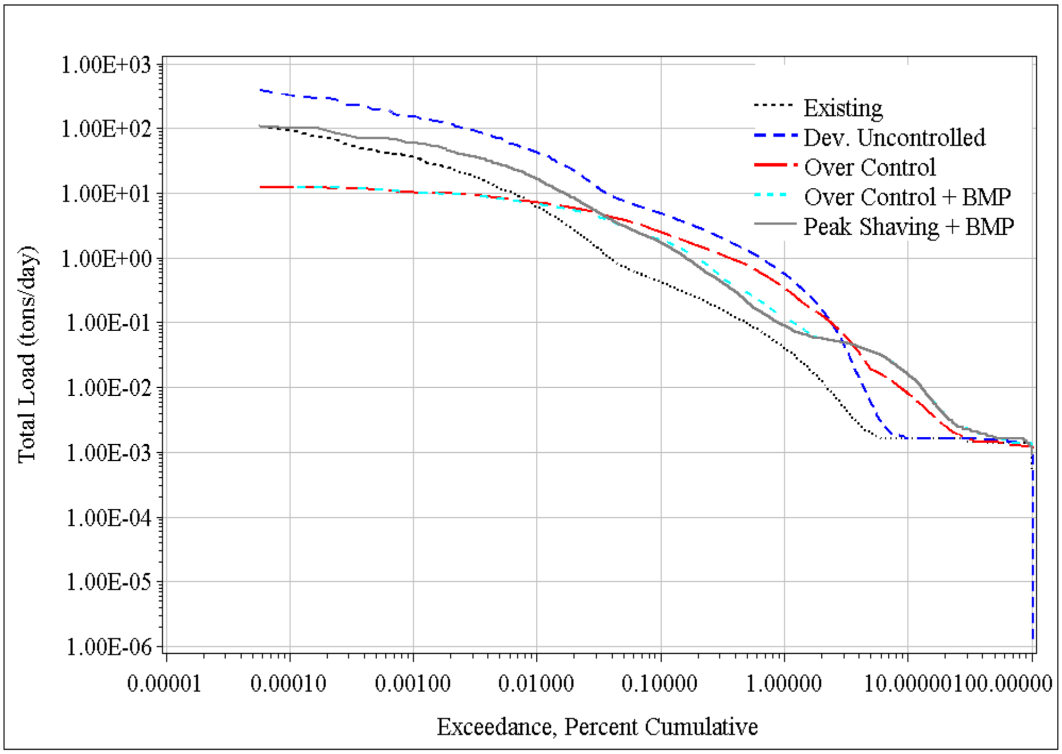

Figure B-13. Brownlie Sediment Load – Medium Sand: Atlanta, Subarea 1300.

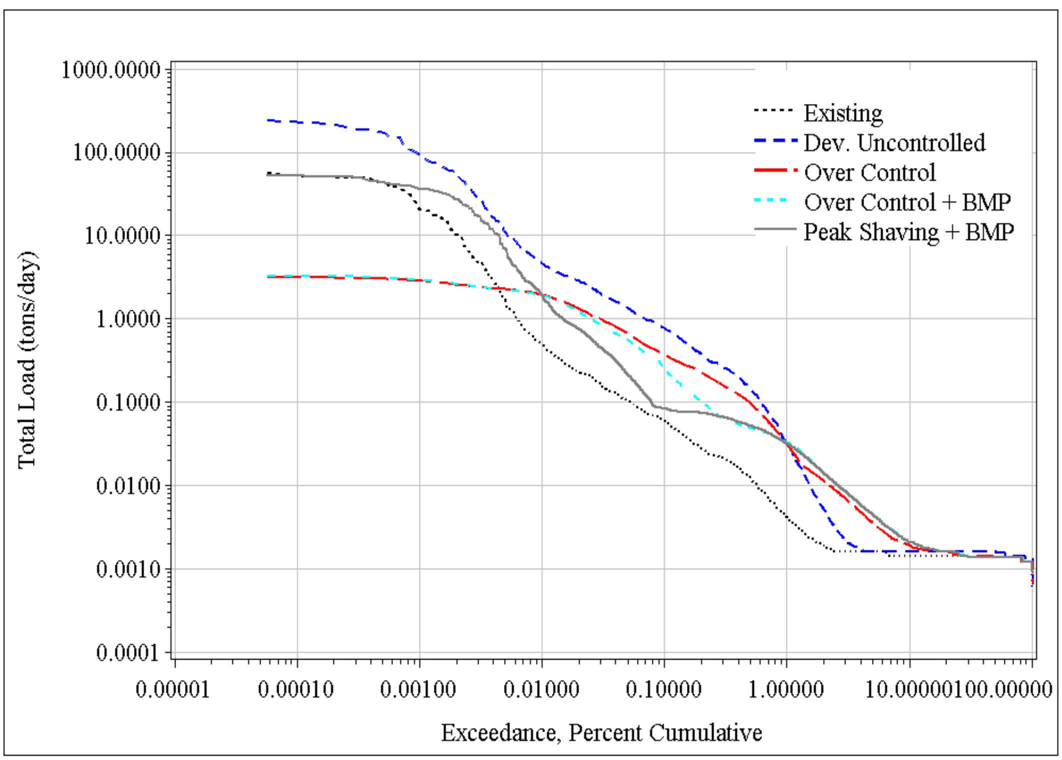

Figure B-14. Brownlie Sediment Load – Medium Sand: Fort Collins, Subarea 1300.

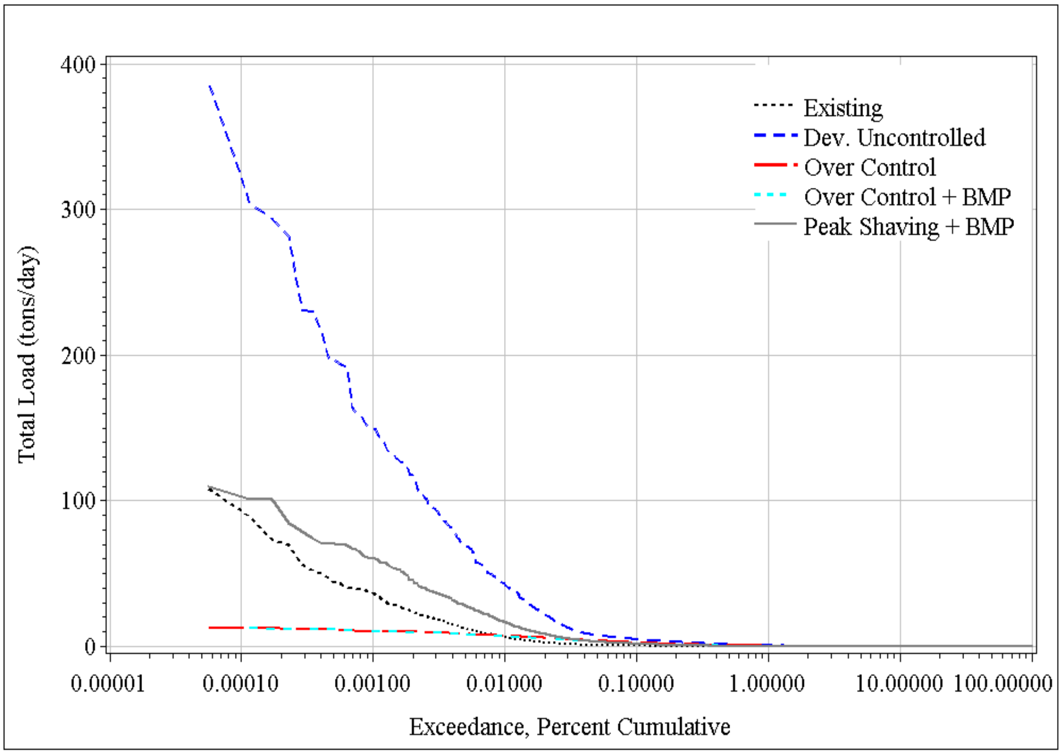

Figure B-15. Brownlie Sed. Load–Med. Sand: Atlanta, Subarea 1300, Semi-log.

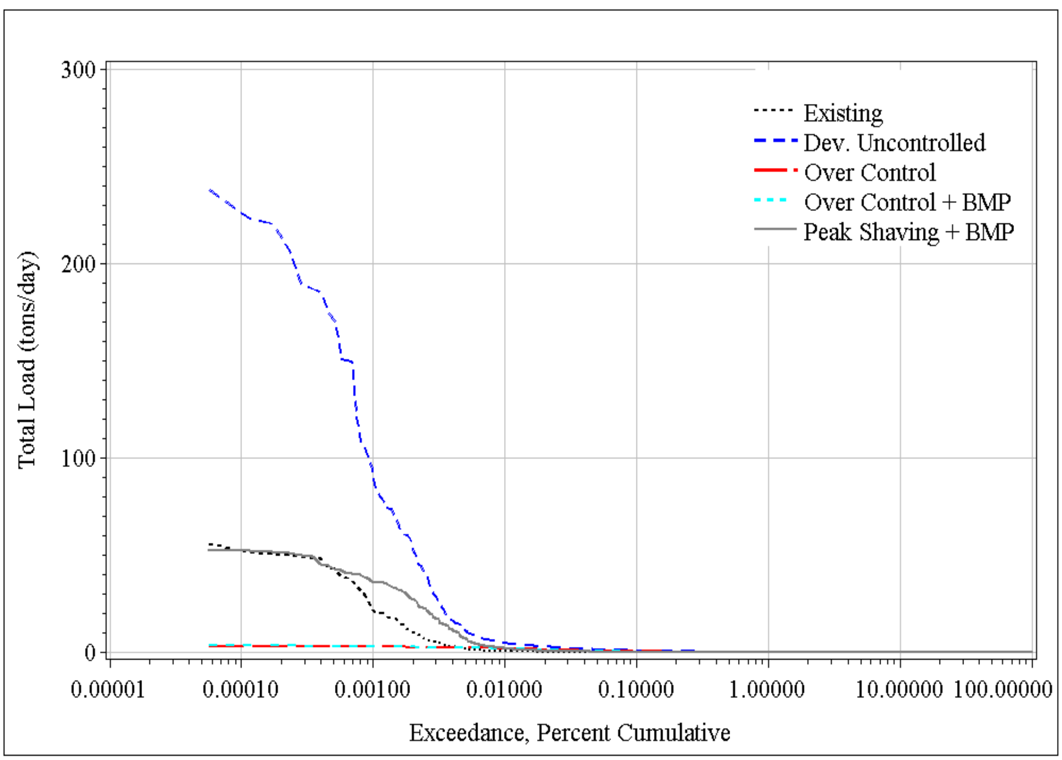

Figure B-16. Brownlie Sed. Load–Med. Sand: Fort Collins, Subarea 1300, Semi-log.

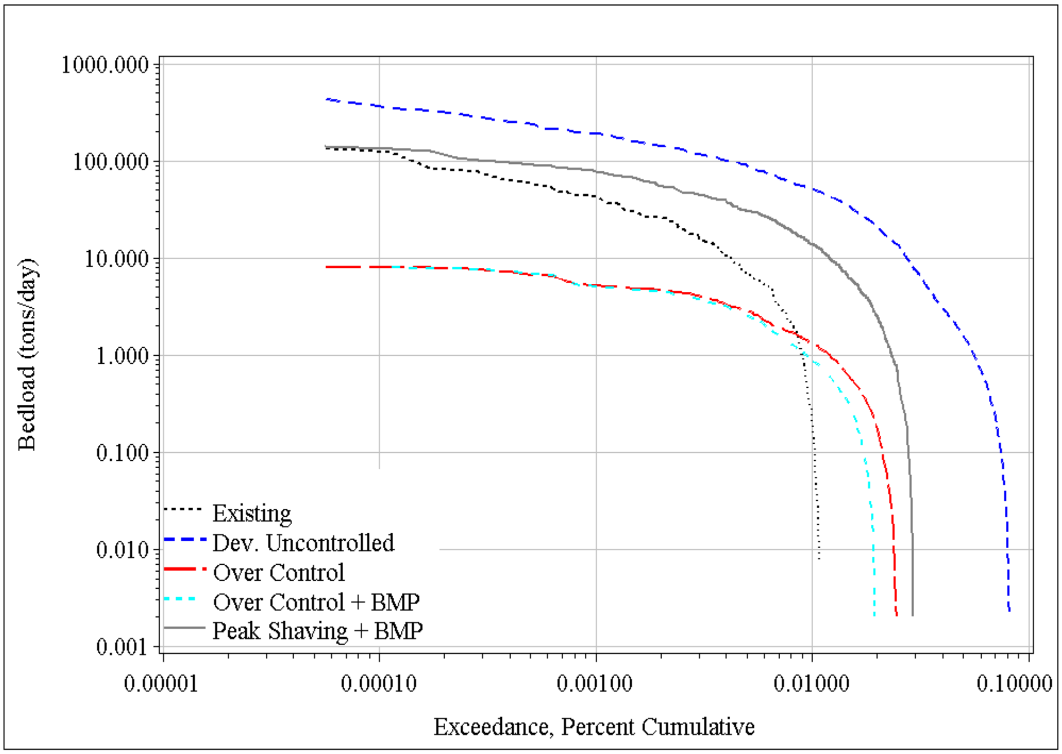

Figure B-17. MPM Sediment Load – Medium Gravel: Atlanta, Subarea 1300.

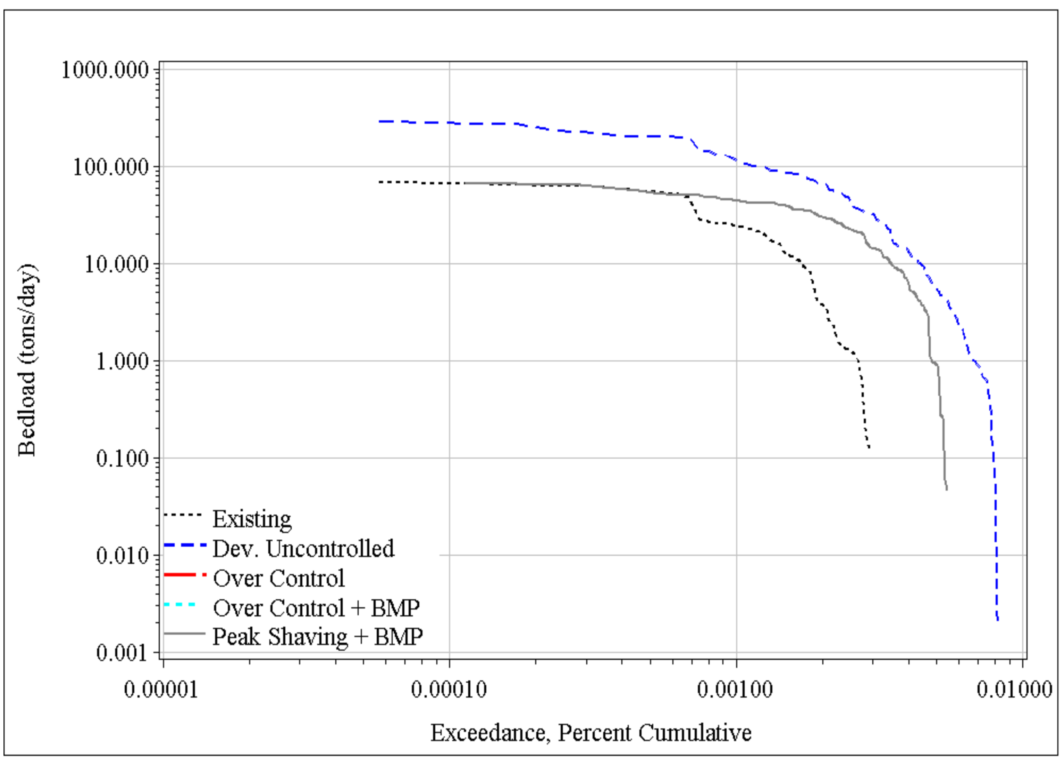

Figure B-18. MPM Sediment Load – Medium Gravel: Fort Collins, Subarea 1300.

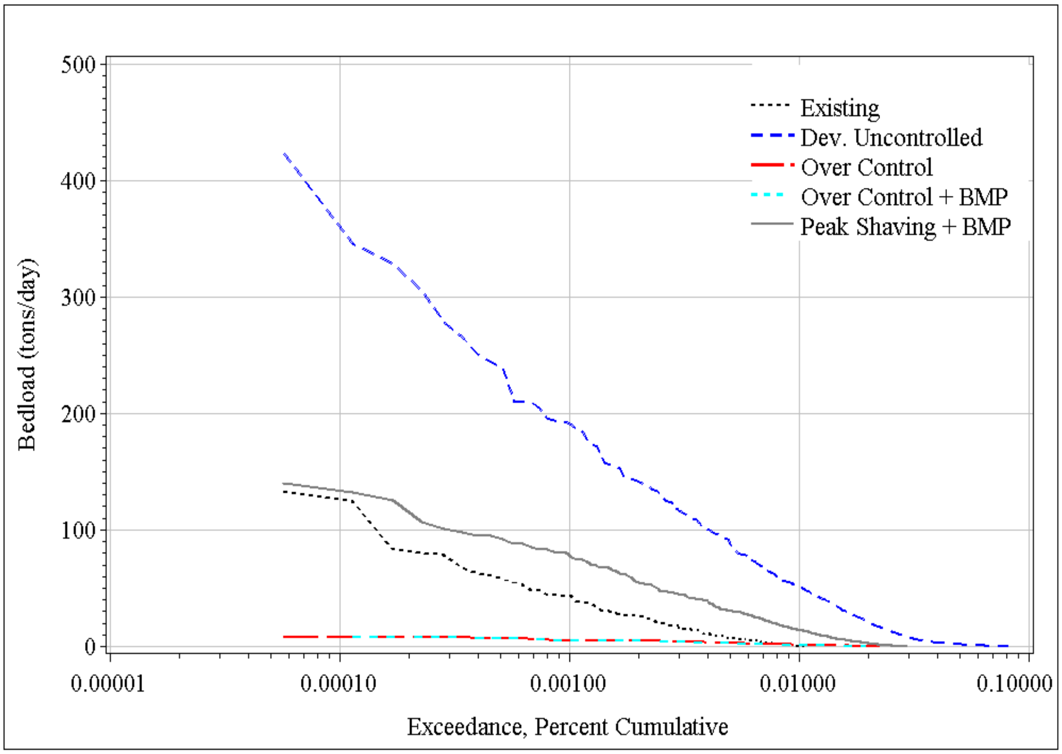

Figure B-19. MPM Sed. Load–Med. Gravel: Atlanta, Subarea 1300, Semi-log.

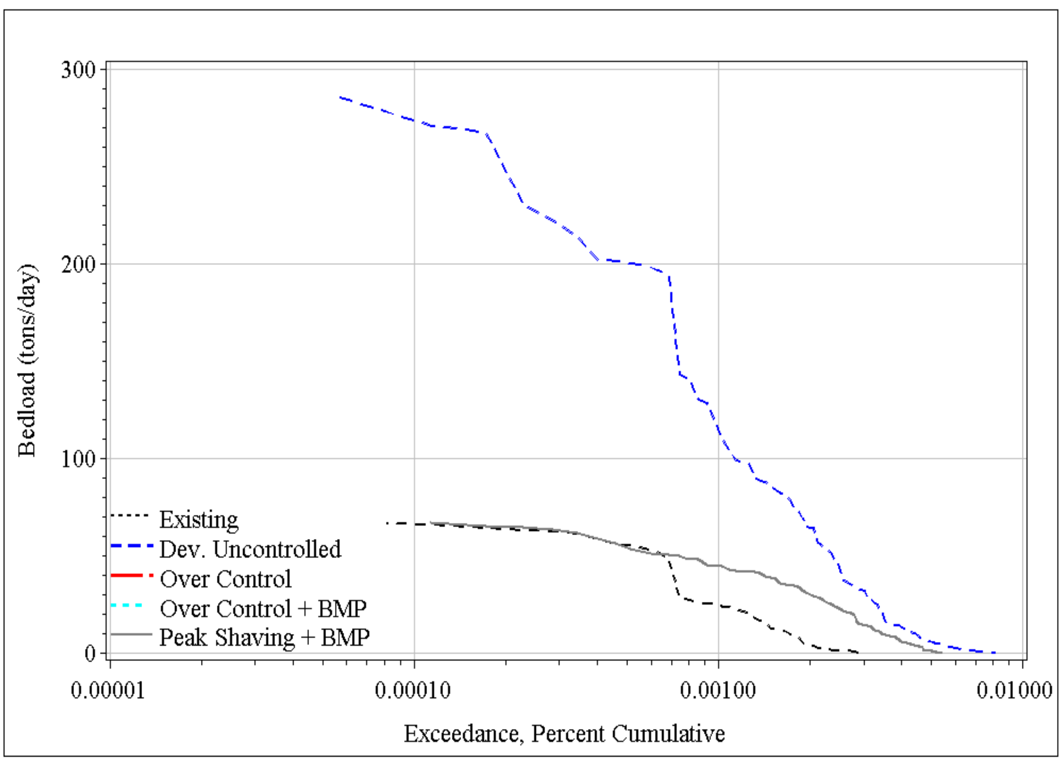

Figure B-20. MPM Sed. Load–Med. Gravel: Fort Collins, Subarea 1300, Semi-log.

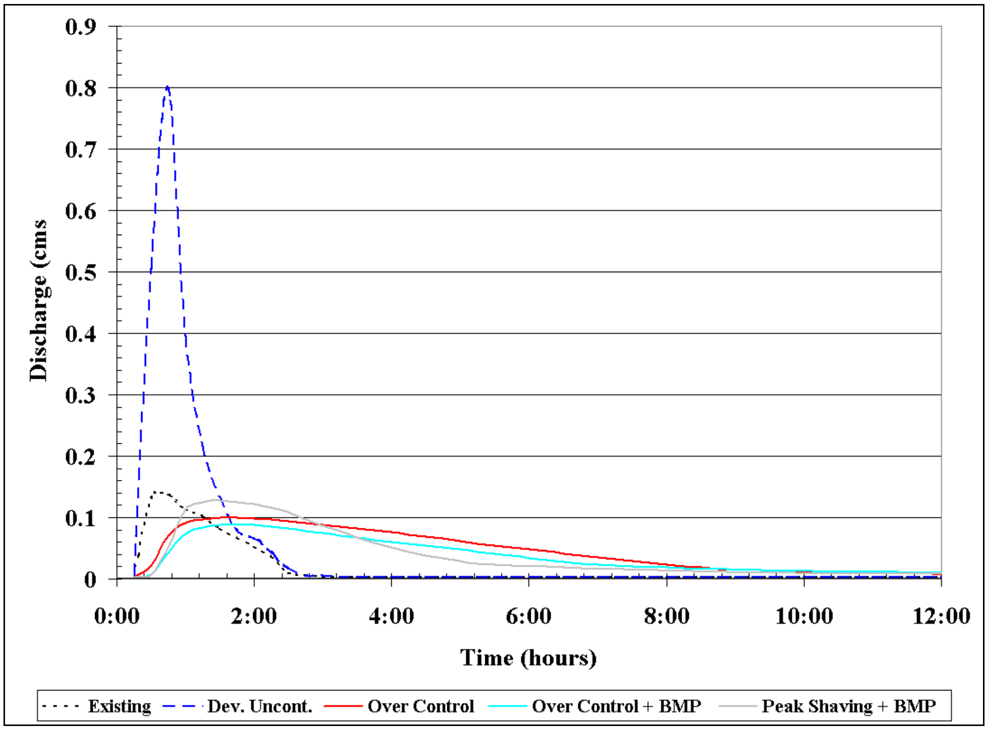

Figure B-21. Hydrograph, Subarea 1300, 2-Year Design Storm, Atlanta.

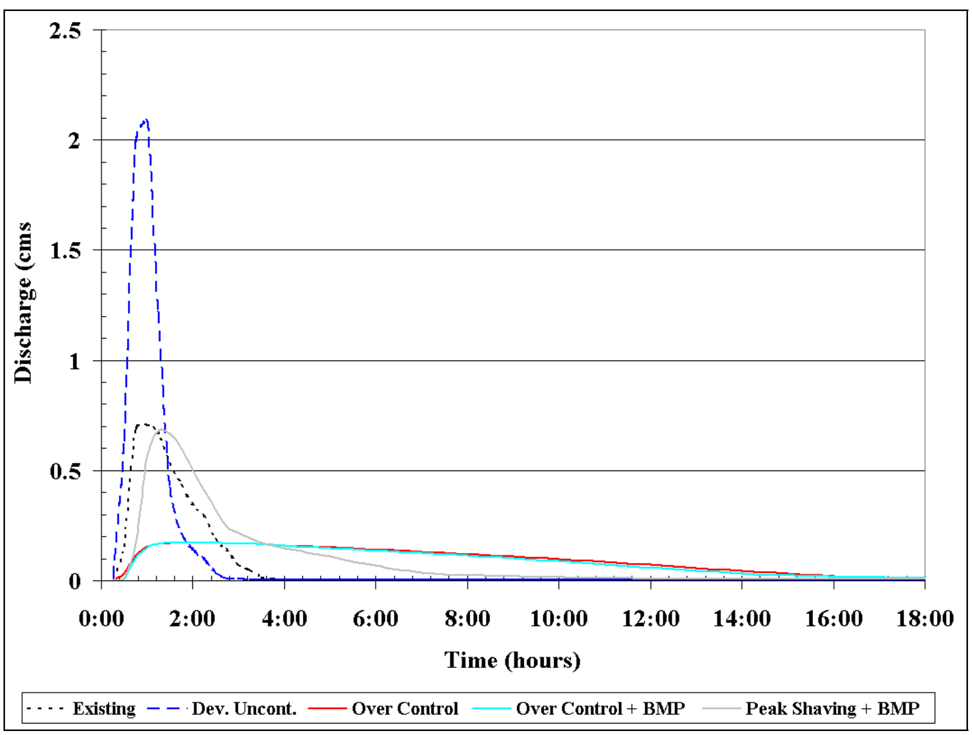

Figure B-22. Hydrograph, Subarea 1300, 100-Year Design Storm, Atlanta.

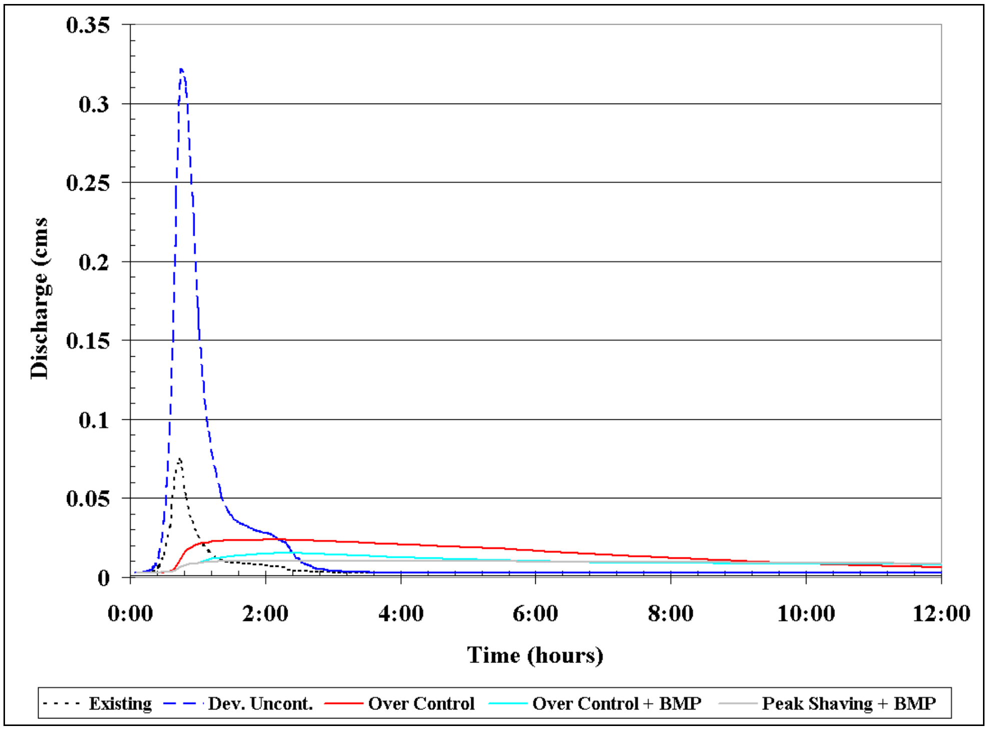

Figure B-23. Hydrograph, Subarea 1300, 2-Year Design Storm, Fort Collins.

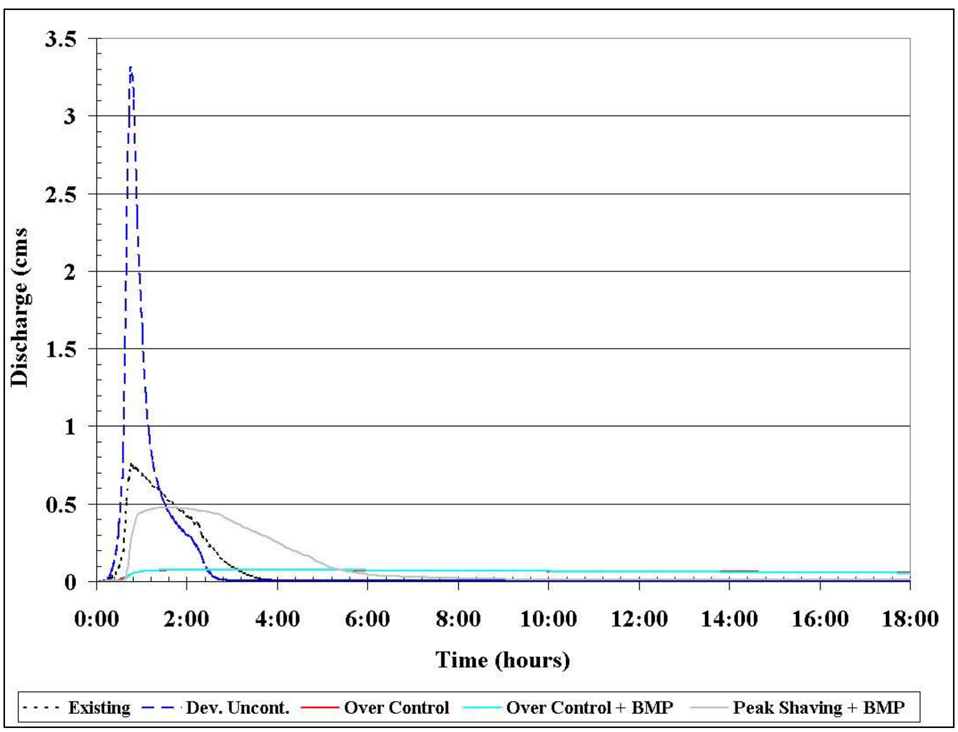

Figure B-24. Hydrograph, Subarea 1300, 100-Year Design Storm, Fort Collins.

APPENDIX C: ADDITIONAL RESULTS DATA – FULL WATERSHED

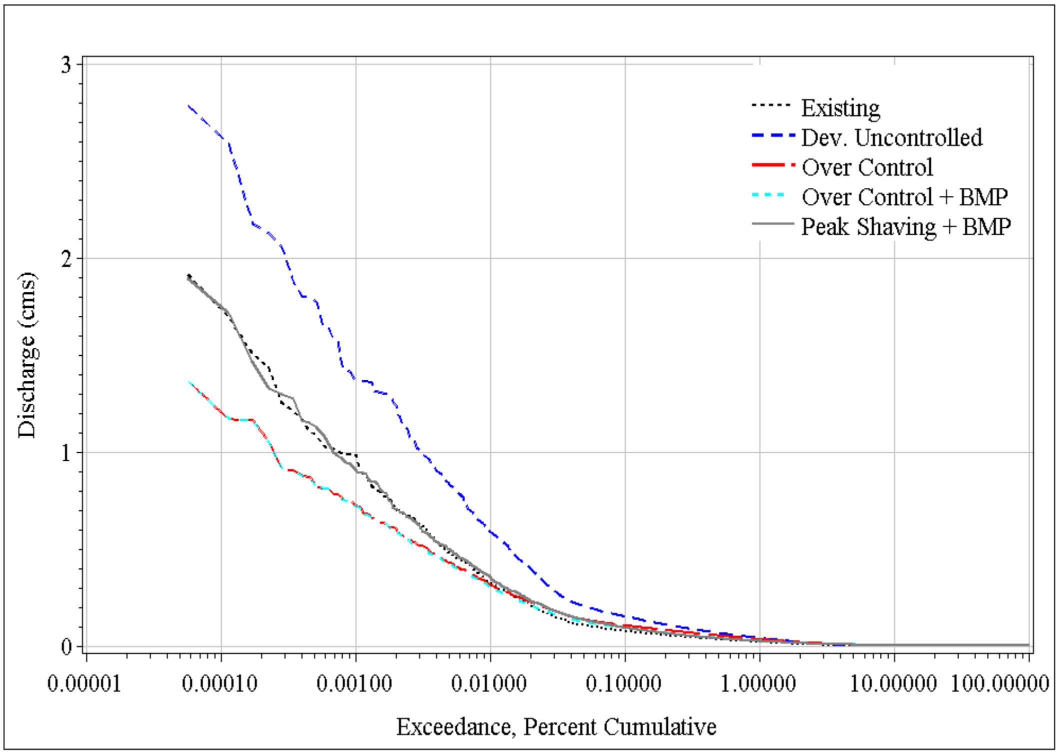

Figure C-1. Discharge Duration: Atlanta, Full Watershed.

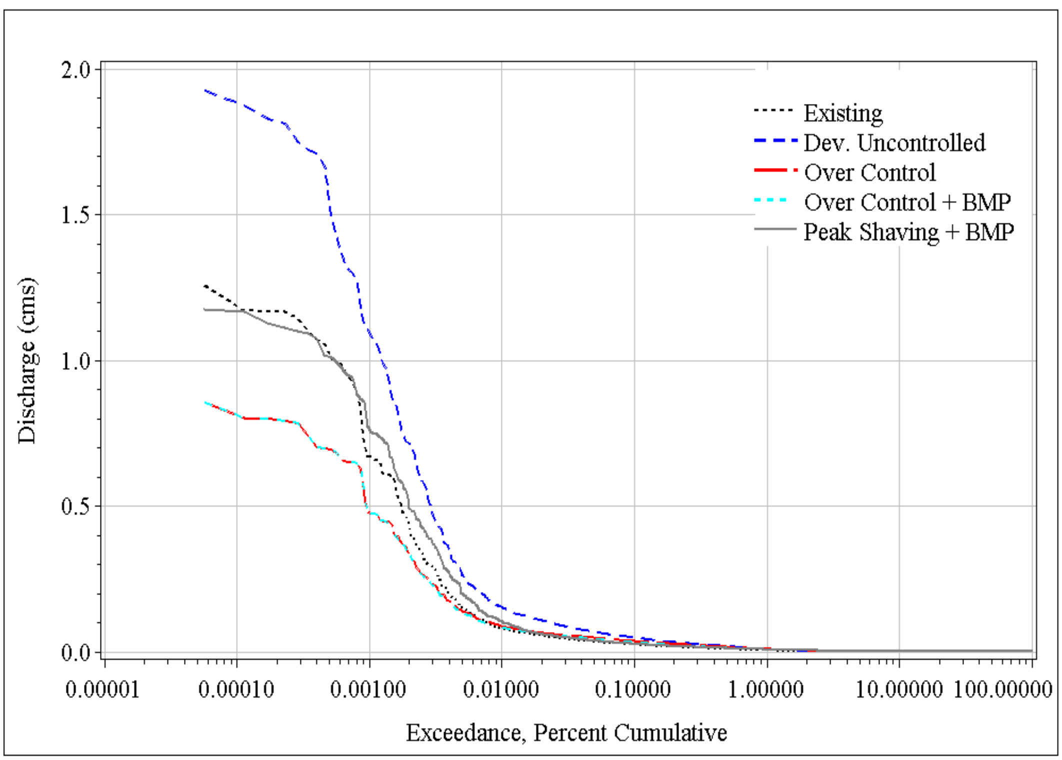

Figure C-2. Discharge Duration: Fort Collins, Full Watershed.

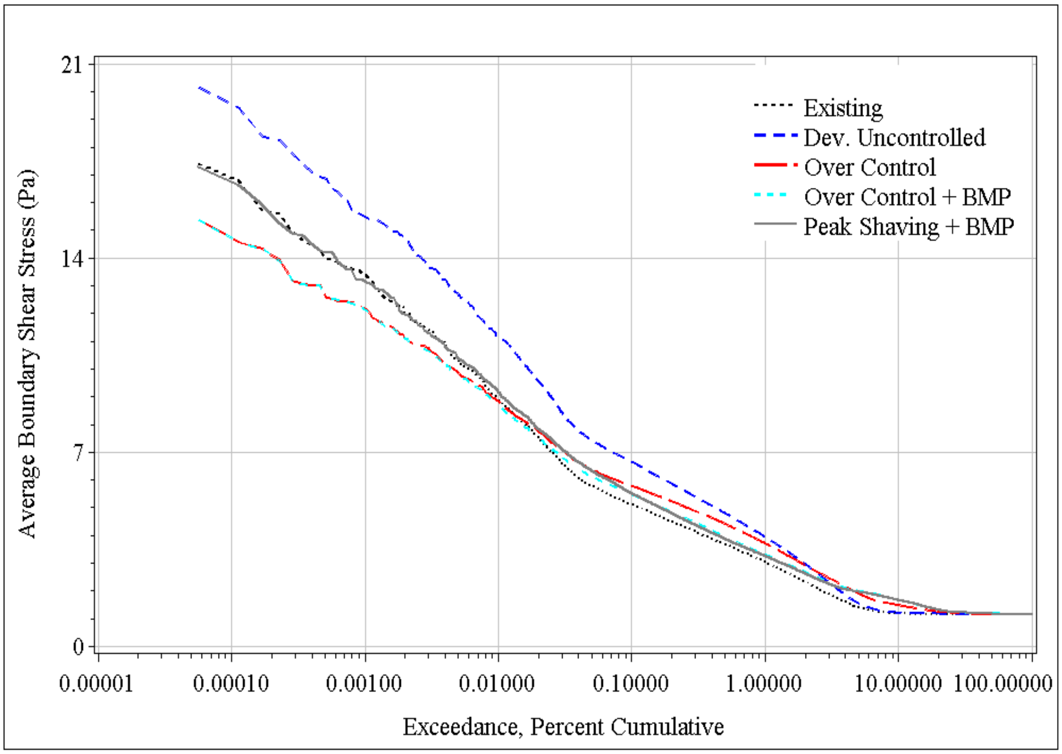

Figure C-3. Average Boundary Shear Stress: Atlanta, Full Watershed.

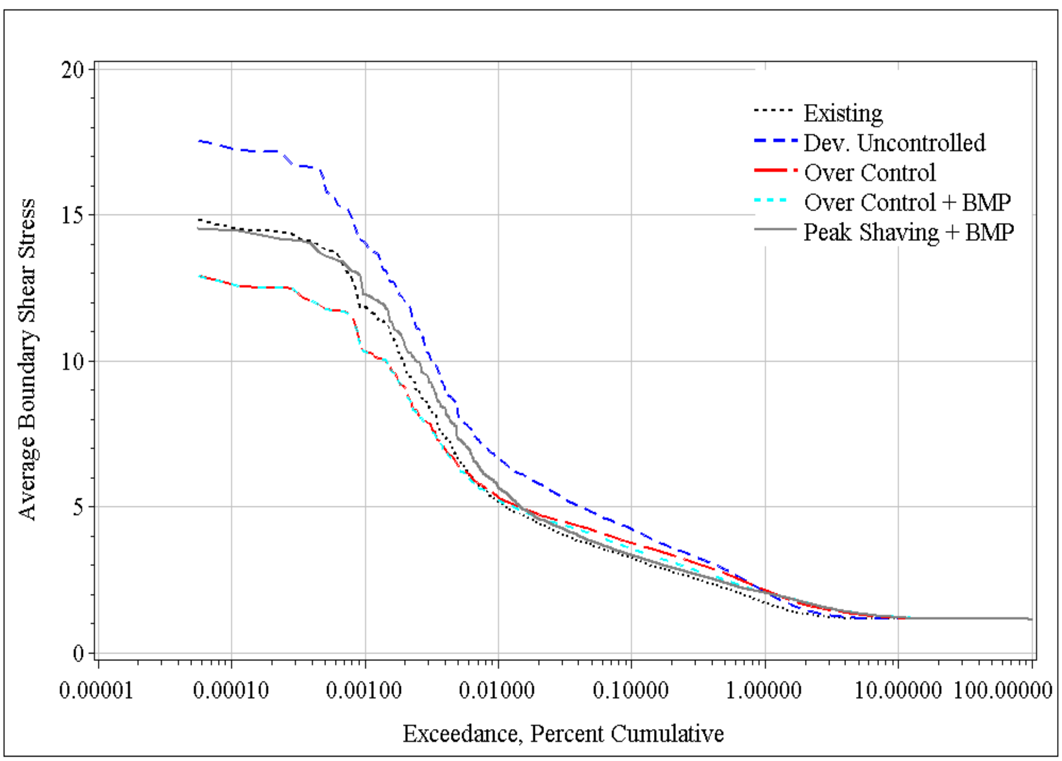

Figure C-4. Average Boundary Shear Stress: Fort Collins, Full Watershed.

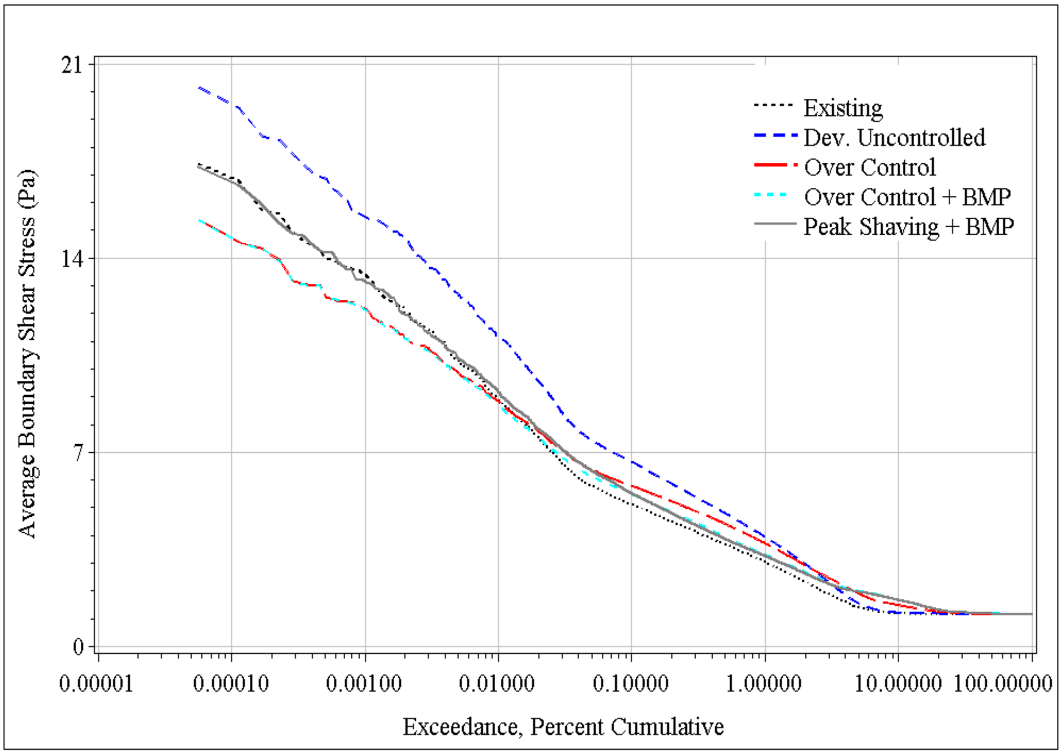

Figure C-5. Specific Stream Power: Atlanta, Full Watershed.

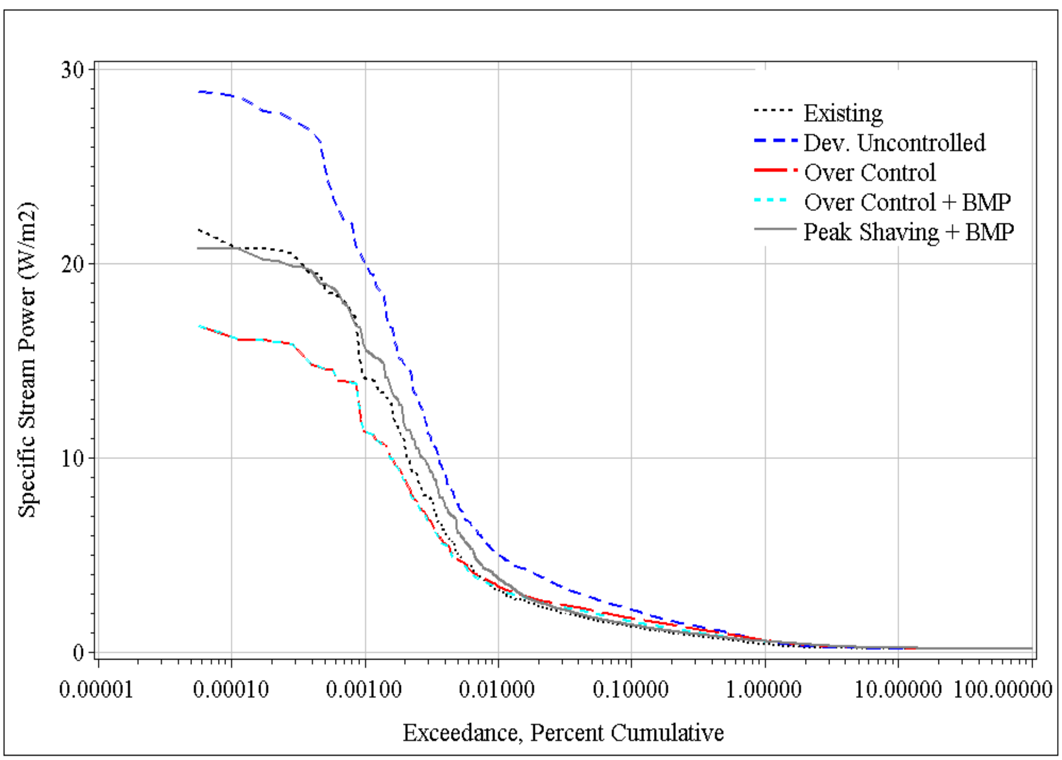

Figure C-6. Specific Stream Power: Fort Collins, Full Watershed.

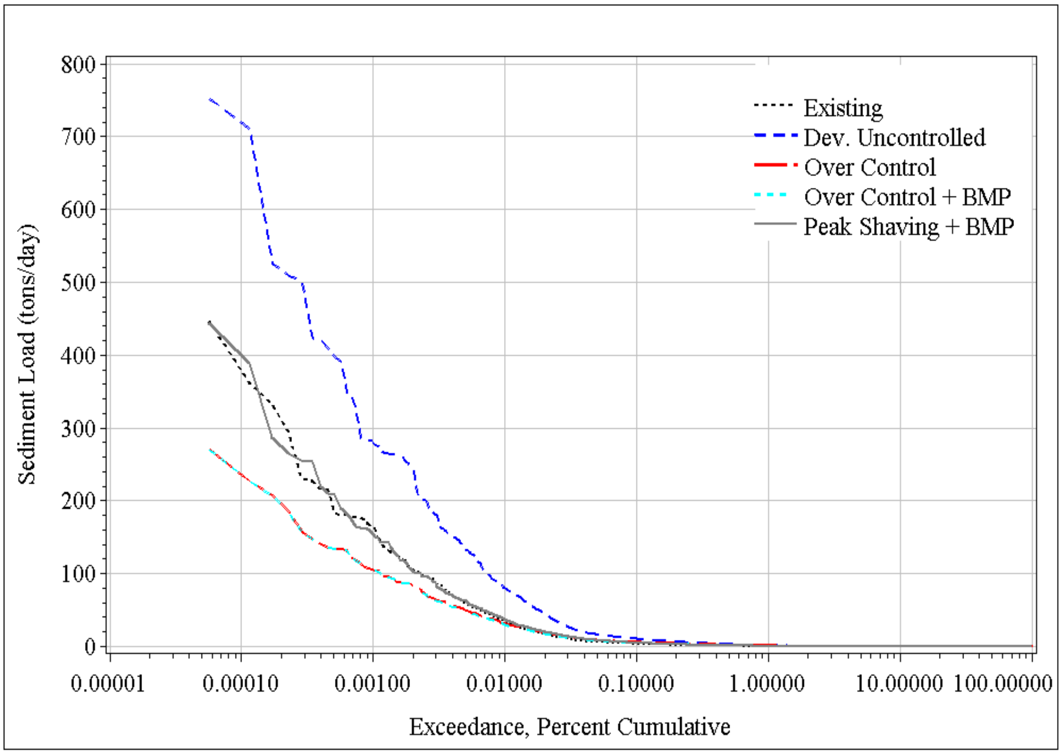

Figure C-7. Brownlie Sediment Load – Medium Sand: Atlanta, Full Watershed.

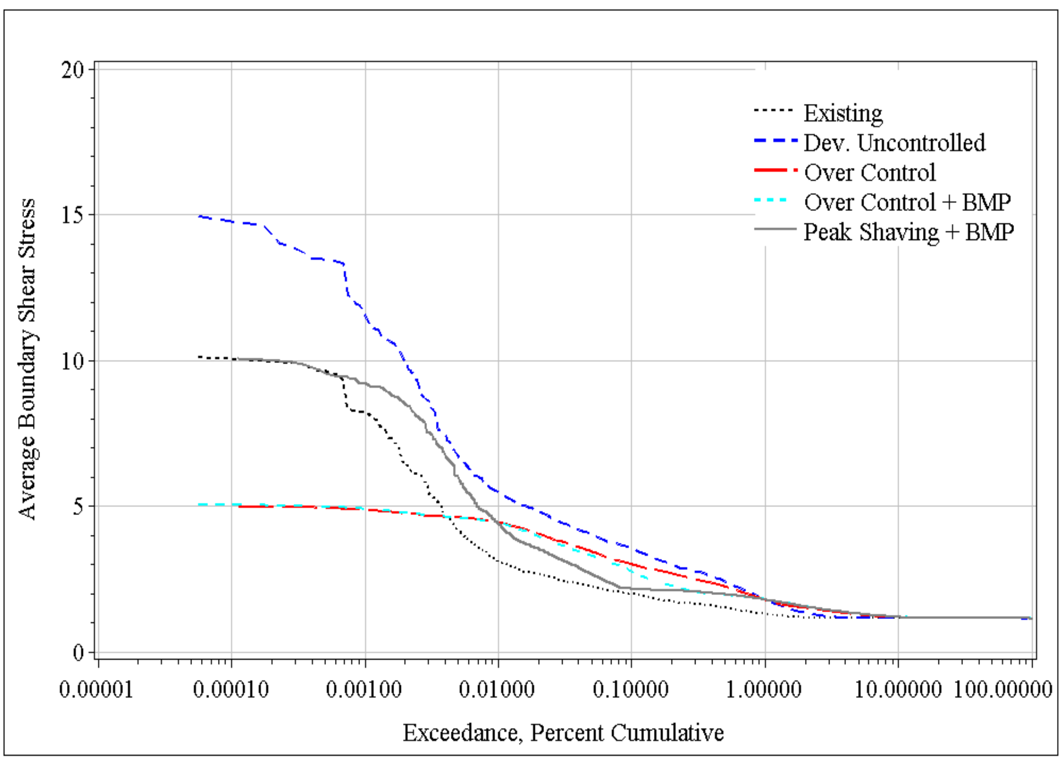

Figure C-8. Brownlie Sediment Load–Medium Sand: Fort Collins, Full Watershed.

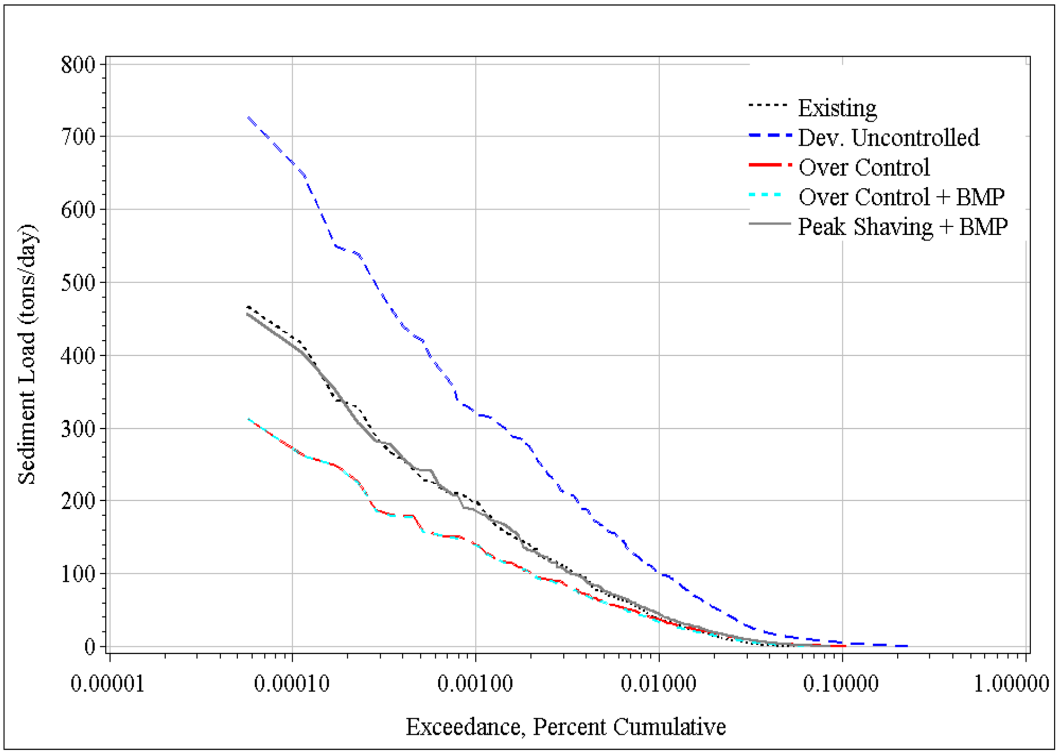

Figure C-9. MPM Sediment Load – Medium Gravel: Atlanta, Full Watershed.

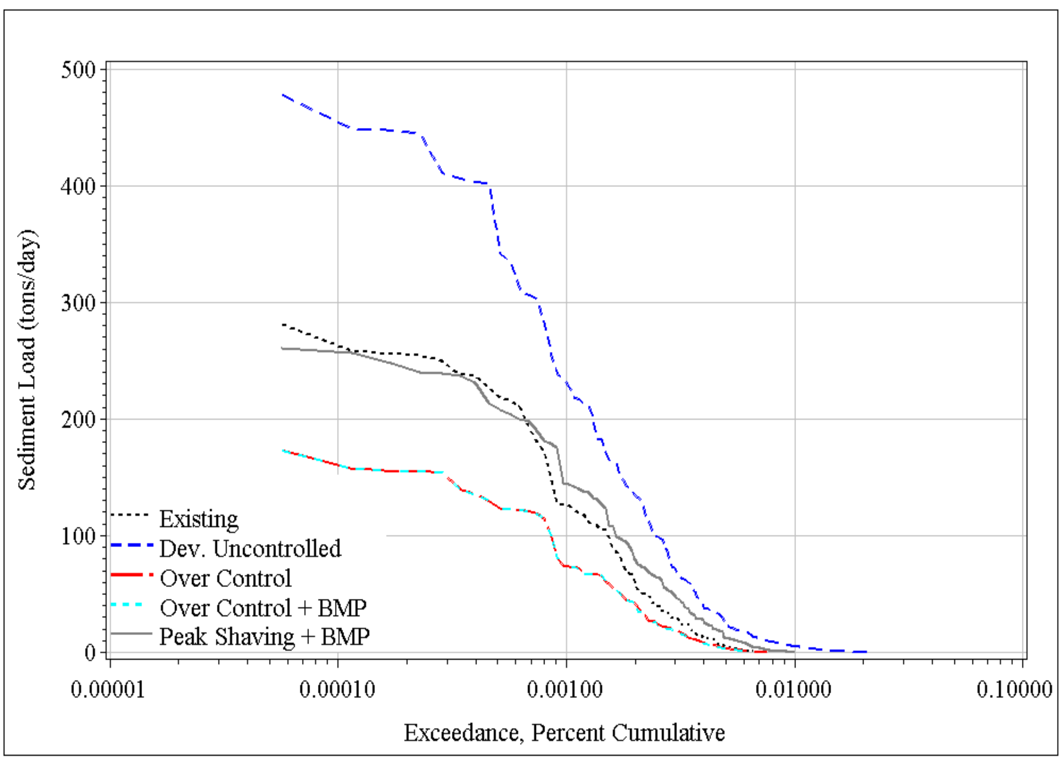

Figure C-10. MPM Sediment Load–Medium Gravel: Fort Collins, Full Watershed.

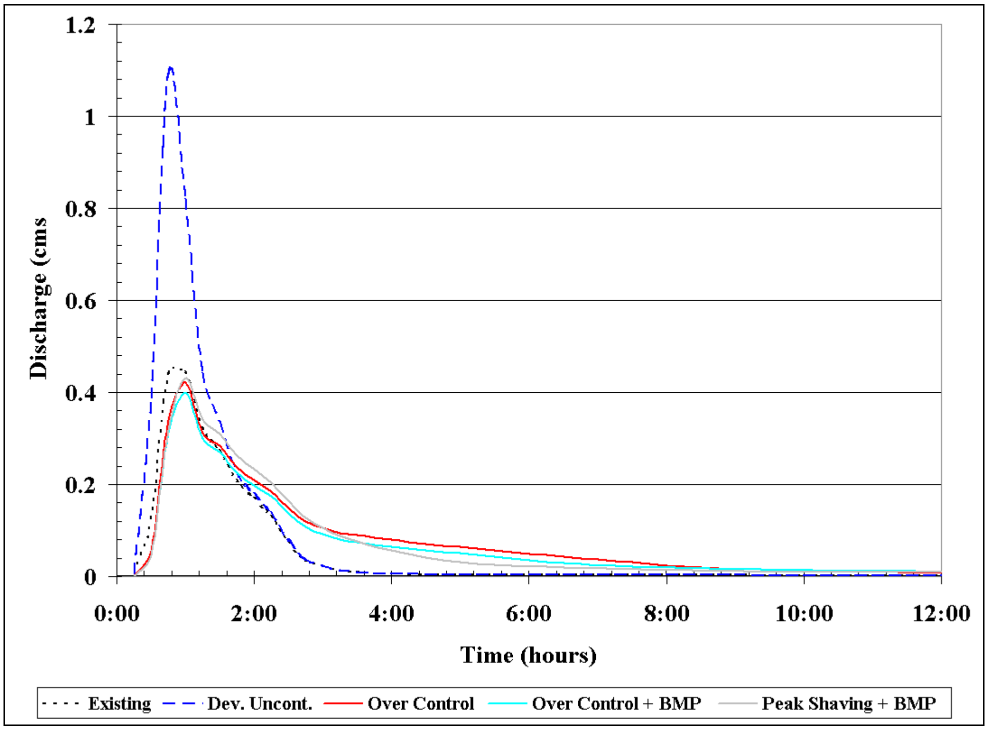

Figure C-11. Hydrograph, Full Watershed, 2-Year Design Storm, Atlanta.

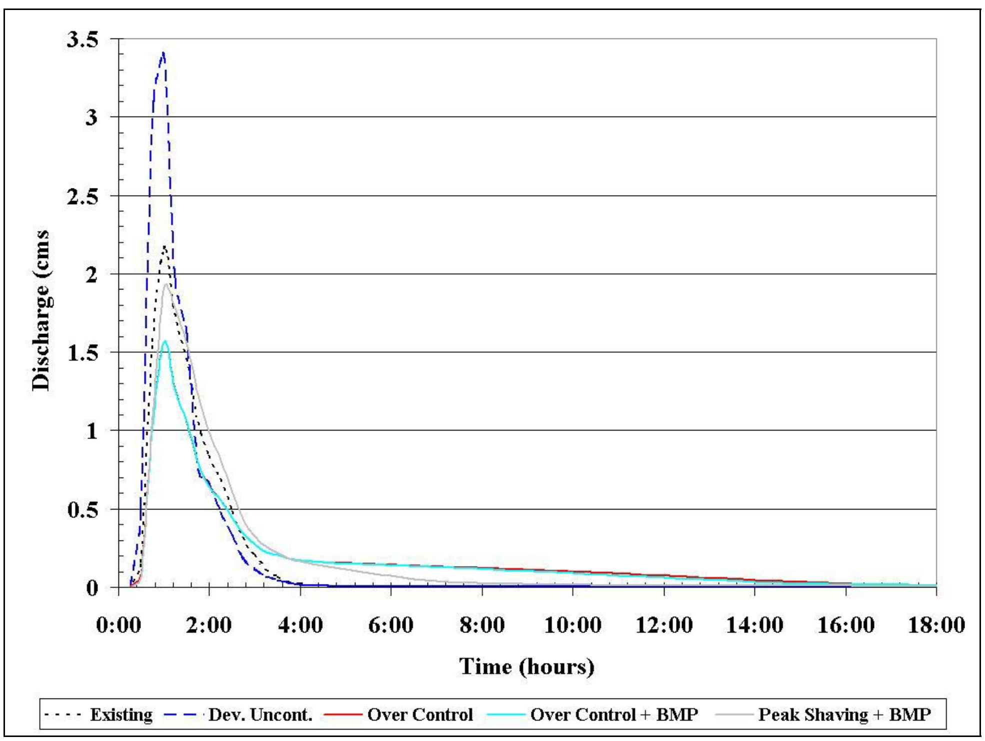

Figure C-12. Hydrograph, Full Watershed, 100-Year Design Storm, Atlanta.

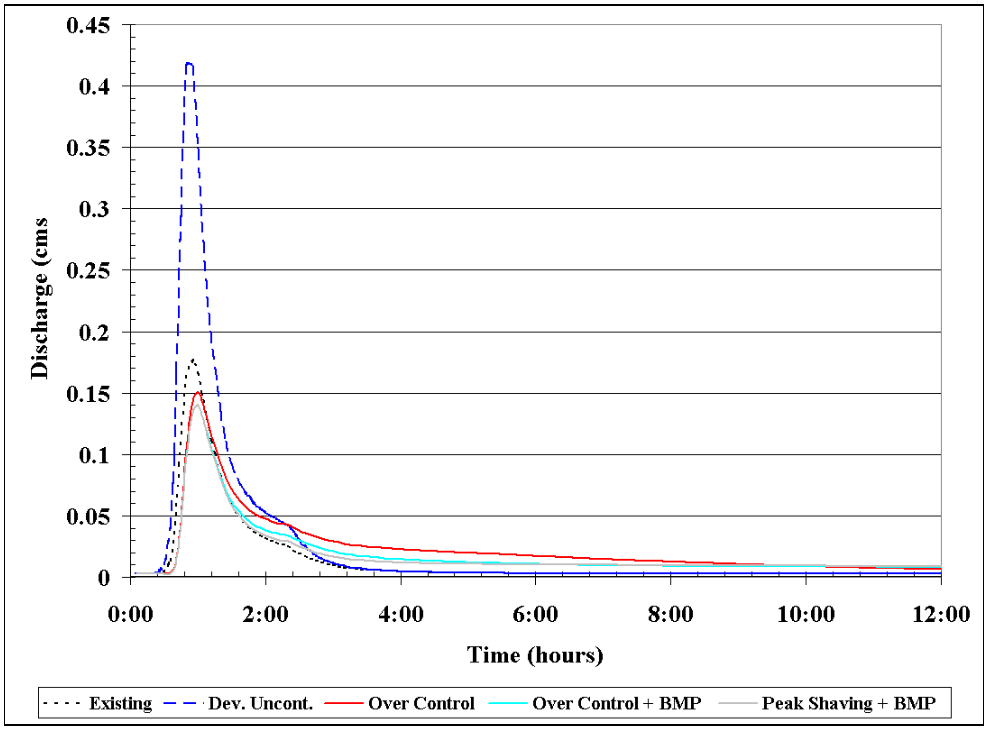

Figure C-13. Hydrograph, Full Watershed, 2-Year Design Storm, Fort Collins.

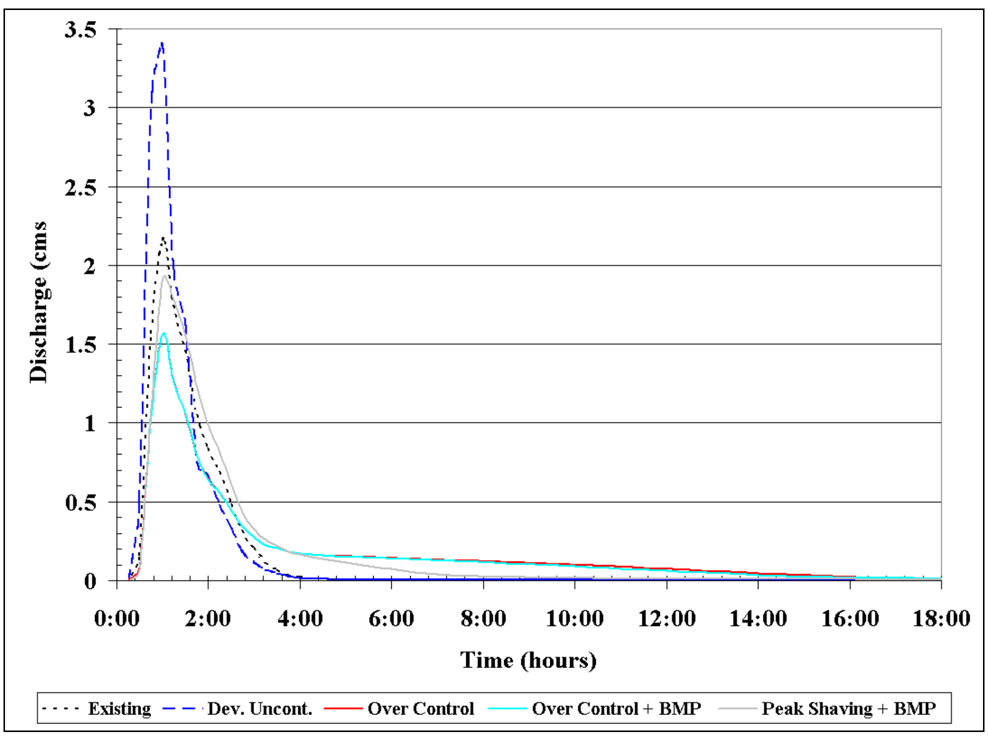

Figure C-14. Hydrograph, Full Watershed, 100-Year Design Storm, Fort Collins.

APPENDIX D: DESCRIPTION OF MODEL FILES

Table D.1. SWMM File Names and Continuity Error Values for Each Run.

Description	Design Storm	Atlanta Runoff	Atlanta Extran	Atlanta Transport	Fort Collins Runoff	Fort Collins Extran	Fort Collins Transport
Existing Conditions	continuous	ra00b -0.128 0.652	N.A.	ta00b 0.609	rfc00b -0.143 0.190	N.A.	tfc00b 0.630
	2-year	ra01b -0.073 0.115	N.A.	ta01b 0.981	rfc01b -0.061 0.217	N.A.	tfc01b 0.612
	100-year	ra03b 0.486 0.775	N.A.	ta03b 0.204	rfc03b 0.648 0.981	N.A.	tfc03b -0.796
Developed Uncontrolled	continuous	ra10b -0.193 0.394	N.A.	ta10b 0.973	rfc10b -0.216 0.134	N.A.	tfc10b 1.071
	2-year	ra11b -0.316 0.166	N.A.	ta11b 0.756	rfc11b -0.121 0.223	N.A.	tfc11b 0.544
	100-year	ra13b -0.062 0.506	N.A.	ta13b -0.638	rfc13b 0.052 0.669	N.A.	tfc13b -0.130
Over Control	continuous	ra20b -0.194 0.344	N.A.	ta20f 0.422	rfc20b -0.216 -0.013	N.A.	tfc20f 0.600
	2-year	ra21b -0.316 0.172	ea21f -0.030	ta21f 0.190	rfc21b -0.121 0.203	efc21f -0.160	tfc21f 0.163
	100-year	ra23b -0.062 0.520	ea23f -0.020	ta23f 0.076	rfc23b 0.052 0.688	efc23f -0.030	tfc23f 0.053
Over Control + BMP	continuous	ra20b -0.194 0.344	N.A.	ta30f 0.461	rfc20b -0.216 -0.013	N.A.	tfc30f 0.677
	2-year	ra21b -0.316 0.172	ea31f -0.110	ta31f 0.216	rfc21b -0.121 0.203	efc31f -0.240	tfc31f 0.236
	100-year	ra23b -0.062 0.520	ea33f -0.040	ta33f 0.078	rfc23b 0.052 0.688	efc33f -0.060	tfc33f 0.049
Peak Shaving + BMP	continuous	ra20b -0.194 0.344	N.A.	ta40g 0.708	rfc20b -0.216 -0.013	N.A.	tfc40g 0.680
	2-year	ra21b -0.316 0.172	ea41g -0.130	ta41g 0.339	rfc21b -0.121 0.203	efc41g -0.250	tfc41g 0.241
	100-year	ra23b -0.062 0.520	ea43g -0.030	ta43g 0.129	rfc23b 0.052 0.688	efc43g -0.030	tfc43g -0.416

N.A. = Not Applicable