Stormwater Retrofit Analysis and Recommendations for Juanita Creek Basin in the Lake Washington Watershed

AUGUST 2012

King County
Department of
Natural Resources and Parks
Water and Land Resources Division

DEPARTMENT OF ECOLOGY
State of Washington

CITY OF KIRKLAND WASHINGTON

Washington State Department of Transportation

"A society grows great when its elders plant trees whose shade they know they shall never sit in."

Greek Proverb

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Ecology Grant: G0800618

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Ecology Grant: G0900618

Prepared for:

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Washington State Department of Ecology
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Submitted by:

King County Science and Stormwater Services Sections
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Photos on cover taken by Jeff Burkey and City of Kirkland.

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ABSTRACT

Untreated stormwater runoff delivers high flow and pollutants (bacteria, petroleum based products, soil, etc.) that impair ecological functions in natural systems and contribute to localized flooding. In response, local governments have made large investments in constructing stormwater mitigation features (e.g., ponds and bio-swales) to ameliorate these impacts. In the heavily developed Juanita Creek basin, stormwater facilities are critically lacking and undersized relative to current standards (Washington State Department of Ecology 2005 Stormwater Management Manual for Western Washington—Ecology 2005 SMMWW). This is a consequence of the development regulations in effect during the middle to late 20th century when the majority of development was occurring in the basin. The effectiveness of seven potential stormwater mitigation scenarios was evaluated using a calibrated Hydrological Simulation Program-Fortran (HSPF) model. The objective of the evaluation was to identify a retrofit strategy or strategies that, if implemented, would achieve the goal of improved flow and water quality conditions supportive of aquatic beneficial uses (e.g., fish use) within the Juanita Creek stream system. A 'multiple lines of evidence' approach was used to evaluate the effectiveness of each mitigation scenario, using metrics that included a biological indicator (Benthic Index of Biotic Integrity–BIBI) estimated from regression relationships with hydrologic metrics, frequency of gravel disturbance, water-quality exceedances relative to Washington State standards (e.g., dissolved copper) and reductions in pollutant loads for water quality parameters with no regulatory thresholds (e.g., total suspended solids). The best performing mitigation scenario combined the flow control standards for low-flow (low impact development (LID) performance standard) and high-flow (standard requirement) proposed by Ecology in the draft Phase 1 Municipal National Pollutant Discharge Elimination System (NPDES) permit with effective date of August 1, 2013. To meet the standard requires not only LID best management practices applied to most impervious surfaces, but also conventional end-of-pipe flow control and treatment facilities. The modeling results indicate that this scenario, if implemented, would achieve the stated project goal. All other scenarios evaluated, including the scenario representing flow control standards from Ecology's 2005 SMMWW targeting high-flows only, were less effective and failed to achieve flow and water quality conditions supportive of aquatic beneficial uses. The estimated cost in current (2011) dollars to retrofit Juanita Creek basin to the best performing mitigation scenario is approximately $1.4 billion ($200 million per square mile).

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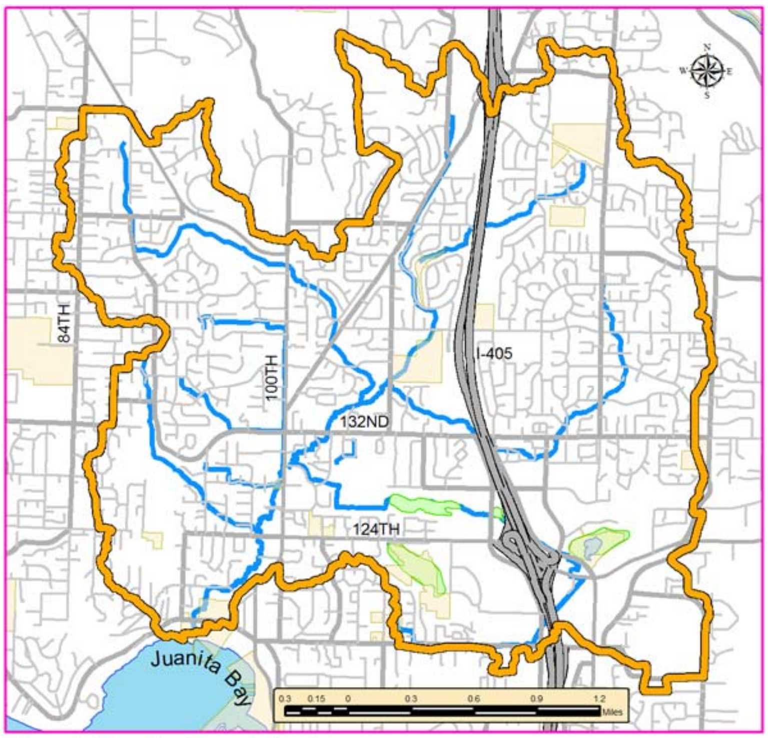
EXECUTIVE SUMMARY

Located northeast of Juanita Bay in the City of Kirkland, the Juanita Creek basin is comprised of an area of approximately 6.8 square miles that includes 15.7 miles of stream. The basin is heavily developed with impervious surfaces (pavement, roofs, etc.) covering approximately 68% of the land area. The primary land use is residential with some commercial development. Underlying soils in the basin are a mix of permeable outwash (45%) and less permeable till soils (55%). Facilities for detaining and treating stormwater are critically lacking and undersized relative to current stormwater standards, a consequence of the mid- and late 20th century development standards in effect when most development was occurring.



Map of Juanita Creek Basin

Runoff patterns and water quality in the Juanita Creek basin have changed dramatically as a result of development that occurred over many decades with ineffective or no stormwater controls. Consequently, the Juanita Creek stream system, whose designated/ beneficial use for aquatic life is core rearing and spawning habitat for coho salmon *Oncorhynchus kisutch*, has been severely degraded. Studies describe the Juanita Creek basin stream system as one where bank instability is a pervasive problem, habitat complexity is low, excessive fine sediments limit salmon spawning areas, frequency of large wood is critically low, and biological conditions are generally poor. Additionally, Juanita Creek is listed on the Washington State Department of Ecology (Ecology) 303(d) list of impaired water bodies for fecal coliform bacteria, dissolved oxygen and temperature.

To address the creek system's problems, King County (KC) partnered with Ecology, the City of Kirkland (COK) and the Washington State Department of Transportation (WSDOT) to recommend a plan of action for making significant systematic and cost-effective improvements in water quality and hydrologic conditions that would be supportive of designated beneficial uses for aquatic life specified as part of the state water quality standards (173-201A WAC).

Funding for the project was provided by Ecology Grant #G0800618 with matching funds and labor provided by KC Department of Natural Resources and Parks, COK, and WSDOT. The Ecology funding source was the Stormwater Management Implementation Grant Program, FY2008, 2007-09 Biennial Capital Budget. Total estimated project cost was $922,389; total eligible cost was $805,479; and the Ecology share was $604,109. The grant agreement had an effective start date of January 31, 2008 and an expiration date of April 15, 2012.

Methods

Although focus was on the Juanita Creek basin, this project demonstrates a quantitative, analytical approach to the development of stormwater retrofit strategies for other urbanized Puget Sound basins. It included extensive stormwater infrastructure mapping, flow gauging, habitat assessment, and water-quality testing, as well as the development of a Hydrological Simulation Program-Fortran (HSPF) model characterizing hydrology and water quality. Stormwater mitigation scenarios were evaluated to determine their effectiveness in achieving the project goal of restoring beneficial use. The scenarios modeled basin retrofits using various combinations and intensities of low-impact development techniques (e.g., rain gardens) and conventional stormwater ponds. For comparison purposes, benchmark scenarios were modeled for conditions including fully forested, 1977 land use, existing land use, future land use, and a hypothetical build-out land use. Cost estimates were prepared for each mitigation scenario in present (2011) dollars.

A calibrated hydrologic model (HSPF) was used to predict metric scores resulting from the benchmark and mitigation scenarios. Multiple metrics were used for evaluating the effectiveness of the mitigation scenarios to provide additional confidence in the predicted outcomes by acknowledging the uncertainty in any single metric's prediction. Metrics included BIBI (Biological Index of Benthic Integrity) based on correlations with hydrologic metrics, frequency of gravel disturbances, water-quality exceedances relative to Washington State standards (e.g., dissolved copper) and reductions in pollutant loads for water quality parameters with no regulatory thresholds (e.g., total suspended solids). The effectiveness of the mitigation scenarios was gauged on the degree to which identified targets for the selected metrics were achieved and also relative to simulated benchmark values.

For targeting, a BIBI of 35 was identified as a critical level based on "EPA's Review of Washington's Water Quality Criteria: An Evaluation of Whether Washington's Criteria

Proposal Protects Stream Health and Designated Uses (Karr et al., 2003[1])," which found that this is a score below which salmonid viability typically drops precipitously. In addition to determining whether a given scenario produced a predicted BIBI equal to or exceeding this critical score, scenarios were evaluated on a comparison of predicted BIBI scores with those of benchmark scenarios for forested, hypothetical build-out, and 1977 land use conditions.

The target range selected for gravel disturbances was 1 to 3 runoff events per year. Doyle (et al., 2000) found stable channel conditions in perennial, humid region streams commonly experience on average this number of events per year large enough to mobilize stream bed gravels.

Water quality targets were taken from WAC 173-201A, except for pollutants for which there are no surface water quality standards, in which case pollutant load reduction was used as a comparative metric. Pollutants evaluated with respect to water quality standards included dissolved copper, fecal coliform, water temperature, and dissolved oxygen. Pollutants evaluated with respect to load reduction included total suspended solids, dissolved copper, total copper, orthophosphate, total phosphorus, nitrate plus nitrite, ammonia, and total nitrogen.

Seven stormwater mitigation scenarios were evaluated, ranging from limited green infrastructure with rain gardens designed to infiltrate, detain, and treat 40% of the impervious surfaces in the basin (LID40), to an extensive retrofit where 80% of the impervious area was routed to rain gardens in combination with regional detention and water quality ponds (ECY08).

Mitigation Scenario Effectiveness

The ECY08 mitigation scenario, representing a combination of the flow control standards for low-flow (low impact development (LID) performance standard) and high-flow (standard requirement) proposed by Ecology in the draft Phase 1 Municipal National Pollutant Discharge Elimination System (NPDES) permit with effective date of August 1, 2013, was the best performing mitigation scenario of the seven evaluated, and the only one that achieved the goal of restoring flow and water quality conditions supportive of restored aquatic beneficial uses. The average predicted BIBI resulting from this scenario was 36—just above the identified critical level and at the high end of "Fair" on the narrative BIBI scale which assigns descriptors of "Very Poor, Poor, Fair, Good, and Excellent" for BIBI score ranges of 10-16, 17-26, 27-36, 37-44, and 45-50, respectively.

With the exception of the LID80 scenario (80% of impervious surface routed to rain gardens), which scored in the low end of the "Fair" range with a predicted BIBI of 28, the remainder of the scenarios had predicted scores in the "Poor" category, with BIBI scores ranging from 17 to 24. This includes the LVL2WET mitigation scenario, representing the

[1] Karr, J.R., R.R. Horner, and C.R. Horner. 2003. EPA's review of Washington's water quality criteria: an evaluation of whether Washington's criteria proposal protects stream health and designated uses. National Wildlife Federation, Seattle. 25 p.

flow control and water quality standards from Ecology's 2005 Stormwater Management Manual for Western Washington (SMMWW), which had a predicted BIBI score of 24.

The mitigation scenarios were generally effective in reducing the number of gravel disturbances from those calculated for existing (2002) conditions for the three sites evaluated. However, to achieve the target range of 1 to 3 disturbances per year, nearly all would require additional actions. Stream roughening measures such as large wood placement would be necessary where reduction of gravel disturbances is required to meet the target range. The amount of added large wood required to sufficiently roughen the channels ranged among the mitigation scenarios from 3 to 9 pieces per 1000 ft of stream. Channel modifications to increase the number of gravel disturbances may be beneficial where results indicate disturbances occur less than once per year.

The ECY08 and LID80 scenarios were the best performers for water quality metrics, although neither could completely match results from the forested condition benchmark. ECY08 resulted in water quality exceedances equal to or below those for the modeled forest condition benchmark for dissolved copper and water temperature, while failing to do so for dissolved oxygen and fecal coliform bacteria. LID80 simulated exceedances were equal to or below forested benchmark levels for water temperature and dissolved oxygen, but failed for dissolved copper and fecal coliform bacteria. When using the 1977 benchmark scenario as a comparison, both scenarios were successful in producing results at or below the target exceedances for all four pollutants. Simulated annual loadings results for ECY08 and LID80 told a similar story: forested benchmark loadings could not be met, but significant reductions compared to the 1977 benchmark were predicted for all eight measured pollutants—with the exception of ammonia for the ECY08 scenario.

The remainder of the mitigation scenarios was less effective in improving water quality—none achieved exceedance targets for all four pollutants when compared to the 1977 benchmark scenario. Only two scenarios (CISTERNS and LID40+) achieved targets relative to the forested condition benchmark and for only one parameter—temperature. Simulated annual loadings results, when using the 1977 benchmark scenario for comparison, indicate that the LID40 and LID40+ scenarios were moderately effective meeting targets for 5 of 8 measured pollutants while LVL2WET and CISTERNS did so for only 3 pollutants.

Cost Estimates

Estimated costs in present (2011) dollars to achieve the mitigation scenarios ranged from $200 million to $1.4 billion dollars, equating to a range of $30 to $200 million per square mile. The most effective scenario, ECY08, was also the most expensive at $1.4 billion. LID80 was the second most effective scenario and second in projected cost at $ 1.2 billion. The remaining mitigation scenarios had lower costs along with generally low overall levels of effectiveness. LID40 and LID40+ had nearly identical costs of approximately $600 million; the difference being three added stormwater ponds in LID40+. The cistern scenario (CISTERNS) that captures rooftop runoff had a cost of $257 million, while the "gray" infrastructure scenarios LEVEL2 and LVL2WET were priced at $210 and $215 million, respectively. Cost estimates included all significant costs required to implement each mitigation scenario, including land purchases using prices specific to the Juanita Creek

basin, construction costs, and operation and maintenance costs accrued over an assumed 40 year life cycle.

Caveats

Several caveats must accompany any interpretation or use of this study's findings. Predicted BIBI scores in the study are based on statistical correlations derived from limited paired hydrology and BIBI data observed over a range of development in stream basins in King County. This study makes the untested assumption that absent other limiting factors prohibiting restoration of healthy stream conditions, improvement in the selected hydrologic metrics will result in improvements in BIBI scores of the order estimated from the derived statistical correlations. In addition, the correlations to hydrologic metrics used to predict BIBI scores portray an *average* response and therefore, for this study, simulated BIBI scores assume that Juanita Creek will similarly respond like an *average* basin.

Conclusions and Next Steps

Of the seven mitigation scenarios evaluated, only the ECY08 scenario successfully met the goal of restoring flow and water quality conditions supportive of aquatic beneficial uses.

Infiltration of stormwater runoff was instrumental in meeting the ECY08 scenario performance standard. In combination with conventional pond facilities, the scenario employed low impact development BMPs (i.e., rain gardens) to treat and infiltrate stormwater runoff from the majority (80%) of the impervious surfaces in the Juanita Creek basin where the mapped distribution of soil types is split nearly equally between those derived from glacial outwash (high permeability) and till (low permeability). The high rates of infiltration assumed for modeling rain gardens in outwash areas was critical in meeting the desired performance. To achieve the same performance in basins dominated by low-permeability soils, extensive use of soil amendments or greater reliance on conventional facilities (ponds) would be necessary.

In addition to being the best performing scenario, ECY08 was easily the most expensive at an estimated $1.4 billion—largely driven by the expenses associated with constructing and maintaining many small, individual rain gardens spread across the landscape. Operations and maintenance costs of the rain gardens account for nearly 60% of the total scenario costs and are expected to mainly be borne by private property owners. However, reliance on the performance of rain gardens (and other small scale BMPs such as soil amendments, etc.) located on private properties and maintained by homeowners may be a concern given that the long-term functionality, inspection, and maintenance requirements of these facility types are not yet well understood. Implementation strategies for the ECY08 scenario should evaluate whether regional infiltration facilities are practicable as these could provide the required infiltration performance at reduced costs, due to a presumptive economy of scale realized in construction and maintenance of fewer, larger facilities. This suggestion, however, includes the caveat that potential cost savings resulting from a regional facility approach will come at the price of allowing impacts from untreated stormwater to occur at all locations upstream of the regional facilities.

Results from the study indicate that the Juanita Creek basin, even if fully restored to the desired hydrologic conditions with amended gravel supportive of coho salmon, will likely require channel modifications to achieve target gravel disturbances of on average 1 to 3

per year. The reduced stream energy from hydrologic mitigation hinders mobilization of median gravel sizes (i.e., they would typically be immobile in most years) because the existing channels have widened due to stream bank erosion over time. This issue may be further compounded by the likely necessary addition of large wood (LWD) to the system to improve overall habitat diversity. In general, LWD will increase hydraulic resistance, lower reach average velocity and grain shear, which would further reduce the likelihood of gravel movement. However, judicious placement of LWD can actually accelerate flow in the middle of the channel and promote channel narrowing over time as sediment builds up near the banks. Thus, it is possible that the stream channels would return to a more natural balance between restored hydrologic regime, channel geometry, and gravel disturbance frequency over time. Further modeling of alternative permutations of LID BMP's (e.g., rain gardens) and detention that result in a hydrologic response comparable to the ECY08 scenario, which matches post-developed flows to forest condition flows for runoff events ranging from 8% of the 2 year storm to the 50 year runoff event, is recommended. Some such alternatives may be even more cost-effective and/or practical to achieve the predicted responses in some settings, depending upon factors such as land cost, public acceptance of LID, etc. Based on our results, however, it is virtually certain that any pond-only mitigation scenario will not effectively meet any credible performance goals.

For the Juanita Creek basin, achievement of the goal to improve flow and water quality conditions to levels identified as critical for supporting aquatic beneficial uses (e.g. fish use) will require stormwater performance that at minimum matches that of the ECY08 mitigation scenario evaluated in this report. Achieving this performance level may be accomplished through stormwater regulations applied to new and redevelopment projects as well as retrofitting existing development. Without an aggressive capital retrofit program, however, it could take many decades for the full benefits to be realized given the slow rates of new and redevelopment occurring in the basin.

Next steps towards implementing the study recommendation for the Juanita Creek basin are detailed in section 5 of the report. These include preparing an implementation plan, selecting a pilot sub-basin to construct and measure the results of the mitigation strategy, and determining required large wood densities to achieve target gravel disturbance metrics in the remaining stream reaches.

Although the study was limited to the goal of evaluating stormwater mitigations to improve flow and water quality conditions for supporting aquatic beneficial uses, a comprehensive approach to stream restoration would include other elements—likely a combination of riparian buffer restoration to provide improvement in stream shading as a strategy to address water quality (temperature) issues in the stream system, bank stabilization projects to improve water quality and reduce fine sediments that impact salmon spawning habitat, and addition of LWD to achieve densities within the range typical for a natural stream system to improve rearing habitat for juvenile salmonids.

1.0. INTRODUCTION

The Juanita Creek basin has been heavily impacted by changes in stormwater, a result of development that occurred over many decades with ineffective or no stormwater controls. Consequently, the Juanita Creek stream system, whose designated beneficial use for aquatic life is core summer spawning habitat for coho salmon, has been severely degraded.

To address the creek system's problems, King County (KC) partnered with the Washington State Department of Ecology (Ecology), the City of Kirkland (COK) and the Washington State Department of Transportation (WSDOT) to recommend a plan of action for making significant systematic and cost-effective improvements in water quality and hydrologic conditions that would be supportive of aquatic life. The project included extensive mapping, gauging, and water-quality testing, as well as the development of hydrologic and water-quality models characterizing the hydrologic regime of the basin. These models were then used to evaluate a range of stormwater mitigation scenarios that included varying combinations of green (i.e., low-impact development) and gray (i.e., detention ponds and wet ponds) infrastructure.

Funding for the project was provided by Ecology Grant #G0800618 with matching funds and labor provided by KC Department of Natural Resources and Parks, COK, and WSDOT. The Ecology funding source was the Stormwater Management Implementation Grant Program, FY2008, 2007-09 Biennial Capital Budget. Total estimated project cost was $922,389; total eligible cost was $805,479; and the Ecology share was $604,109. The grant agreement had an effective date of January 31, 2008 with an expiration date of April 15, 2012.

1.1 Project Area

Juanita Creek is located on the northeastern shore of Lake Washington (Figure 1). Land use in the basin is composed of mostly residential and commercial developments (including a major interstate freeway corridor, I-405), two large forested wetlands, and one small lake that receives stormwater runoff from a large portion of the commercial development in the eastern headwaters of the basin.

The drainage area is estimated to be 6.8 square miles, with approximately 15.7 miles of stream channel. Based on 2002 satellite data, approximately 65 percent of the basin is developed, mostly in residential land uses with smaller amounts of commercial use (Table 1). Two dominant soil types are found in the basin, one derived from low permeability glacial till (till soils) and the other derived from high permeability glacial outwash (outwash soils). Outwash soils generally occur at low gradient within stream valleys while till soils are found on steeper hill sides and ridges along the basin-perimeter. For the entire study area, till soils are more common making up 55 percent of the basin compared to 45 percent for outwash soils.

Table 1. Estimated 2002 land use for study area (Alberti, et al., 2004).

	2002 Land Use	
Land Use	Acres	Percent
High-Intensity Development	801	18.5%
Medium and Low-Intensity Development	1,996	46.0%
Bare Ground	19	0.4%
Dry Ground	22	0.5%
Grass/shrub	139	3.2%
Forest	1,313	30.2%
Wetlands	46	1.1%
Open Water	6	0.1%
Total	4,342	acres

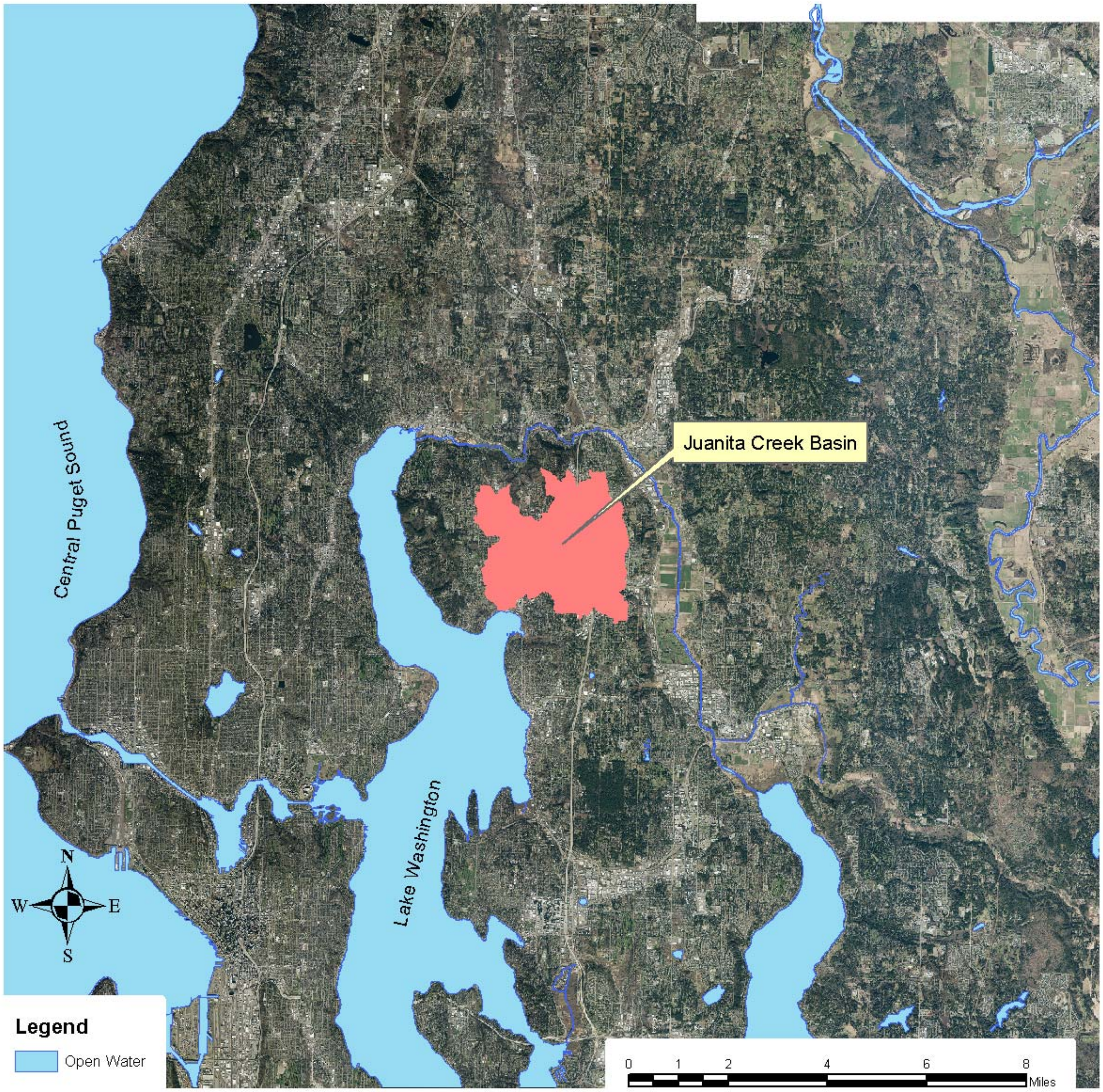


Figure 1 Location of Juanita Creek drainage basin.

1.2 Background

The Juanita Creek basin is a prime example of an area that was developed during the mid-20th century with few or no stormwater controls. Impervious area covers approximately 68 percent of the basin, and the equivalent volume of current stormwater storage is negligible. The vast majority of the stormwater detention facilities in the basin were designed to standards that are now understood to be ineffective with regard to protecting streams from erosive, channel-deforming flows and are only a fraction of the size that would be required under current stormwater standards (i.e., the 2009 King County Surface

Water Design Manual (KCSWDM) and Ecology's 2005 Stormwater Management Manual for Western Washington (SMMWW)).

Juanita Creek is on Ecology's 303(d) list of impaired water bodies for the following water quality parameters: fecal coliform bacteria, dissolved oxygen and temperature (Appendix A). Much of the basin was developed before requirements for water quality treatment of stormwater were implemented by King County via the 1990 KCSWDM. King County's Habitat Inventory and Assessment of Juanita Creek in 2000 (Rush et al., 2002), Otak's Juanita Creek Basin Stabilization Study (2000), and Northwest Hydraulic Consultants' Juanita Creek Basin Geomorphic Analysis (Appendix B) paint a picture of the Juanita Creek basin stream system where bank stability is a pervasive problem, habitat complexity is poor, excessive fine sediments limit salmon spawning areas, and the frequency of large wood in the channel is critically low. Recent BIBI analysis within the Juanita Creek basin by King County have found that biological conditions are generally poor (Appendix C).

The over-arching goal of the project was to inform the restoration and protection of the aquatic life beneficial uses of the Juanita Creek basin stream system that are affected by stormwater. The specific objectives proposed and accomplished by this project in support of this goal included: identifying a stormwater management mitigation strategy that, if implemented, would be supportive of restoring the beneficial use of the Juanita Creek basin stream system and developing cost effectiveness curves for various mitigation scenarios that can be applied to Juanita Creek basin. Additional objectives that apply more broadly to other urbanized areas of the Puget Sound included: developing analytic methods to support future basin retrofitting projects, and developing a template "cost and effectiveness" method for identifying retrofit projects with improved chances of restoring salmon habitat.

1.3 Project Management

Active management of the project was provided by King County. Quarterly reports were provided to Ecology detailing financial expenditures and project progress.

The project was implemented by two main teams:

King County Internal Design Team: Curt Crawford, Jeff Burkey, Dale Nelson, David Batts, Hans Berge, Mark Wilgus, and Steve Foley, supported by Northwest Hydraulic Consultants (NHC) and Stillwater Sciences.

Project Management Team: Ed O'Brien and Anne Dettelbach from Ecology; Curt Crawford, Mark Wilgus, Steve Foley, and Jeff Burkey from King County; Jenny Gaus from COK; and Bill Jordan from WSDOT.

2.0. METHODS

Multiple types of data were collected to support the goals and objectives of this study, either for model development and/or characterization of existing conditions. Much of the data-collection effort has been previously documented in reports and technical memorandums, which are included in the appendices for reference. The sections below are brief summaries of those documents where they exist.

2.1 Water Quality and Quantity Monitoring

Water quality monitoring stations were set at eight locations (Appendix A—Figure 2) within Juanita Creek: one near the mouth on the mainstem, one at each of the four major tributaries, one upstream and one downstream of a large regional wetland, and one at the mouth of Billy Creek. Water quality samples were collected during three base flow and seven storm flow conditions, over a period of 14 months (October 2008 through November 2009). Base flows were sampled twice per day (one AM and one PM) to better understand diurnal conditions at the sampling stations. Storm sampling included taking three samples 8 hours apart at each monitoring station. Laboratory analyses of the samples included: total suspended solids, copper and zinc (dissolved and total), nutrients, and biochemical oxygen demand. Parameters measured in the field included water temperature, dissolved oxygen, pH, and conductivity.

Continuous stream flows were measured at six of the eight locations (excluding inflows and outflows to the regional wetland). For further detail, see Appendix A.

2.2 Geomorphology Assessment

Northwest Hydraulic Consultants, Inc. (NHC) performed hydraulic and geomorphic analysis on Juanita Creek and several of its major tributaries. Stillwater Sciences (Stillwater) served as a subconsultant to NHC for geomorphology and biological issues. NHC analyzed geomorphic parameters for 39 assessment reaches on the Juanita Creek mainstem and nine tributaries. The parameters included: bank stability, substrate size and distribution, local slope, bankfull channel dimensions on priority reaches, and large woody debris (LWD) and large pool frequency. Nearly 3,800 feet of channel was included in the assessment of the seven parameters above, and an additional 4,800 feet of channel length was surveyed for LWD and pool frequency. Substrate fines were assessed over a total of 14,600 feet of channel length (30% of the open channel network in the entire basin and near 50% of the Juanita Creek mainstem). <u>Juanita Creek Basin Geomorphic Analysis</u> (2010) by NHC is included as Appendix B. The NHC assessment reviews complement and expand upon two previous studies, King County's <u>Habitat Inventory and Assessment of Juanita Creek in 2000</u> (Rush et al., 2002) and Otak's <u>Juanita Creek Basin Stabilization Study</u> (Otak 2000).

2.3 Stream Reconnaissance

A rapid assessment of basin streams was conducted using readily available points of access (road crossings and publicly held lands) to investigate relative habitat quality throughout

the watershed and any opportunities for habitat restoration. Observed channel characteristics were: cross-section geometry, wetted width and depth, riparian vegetation, and presence or absence of woody debris. Further detail can be found in Appendix C.

In September 2008, macroinvertebrate samples were collected from seven sites (Appendix C, Figure 1) in Juanita Creek following the protocols of Karr and Chu (1999). The purpose of macroinvertebrate sampling was to assess the benthic index of biotic integrity (BIBI) of Juanita Creek for comparison with other Puget lowland streams. We compared these data to those collected by the City of Kirkland using the same methods (2001 through 2008) at three sites in the same vicinity as our collections in 2008, which were subsequently used as part of the evaluation of a watershed model's ability to predict BIBI scores (Section 3.4).

2.4 Stormwater Mapping

Detailed mapping of the stormwater infrastructure was conducted by City of Kirkland. The mapping identified alignments and directions of flow for the network of pipes, vaults, ponds, etc. (Figure 2). This level of detail was used to refine catchment delineations that were topographically driven with storm pipe interceptions. Additionally, a rough estimate of transient storage in the pipe network was integrated into the watershed hydraulic routing schema. These data can be obtained from the City of Kirkland Public Works Department.

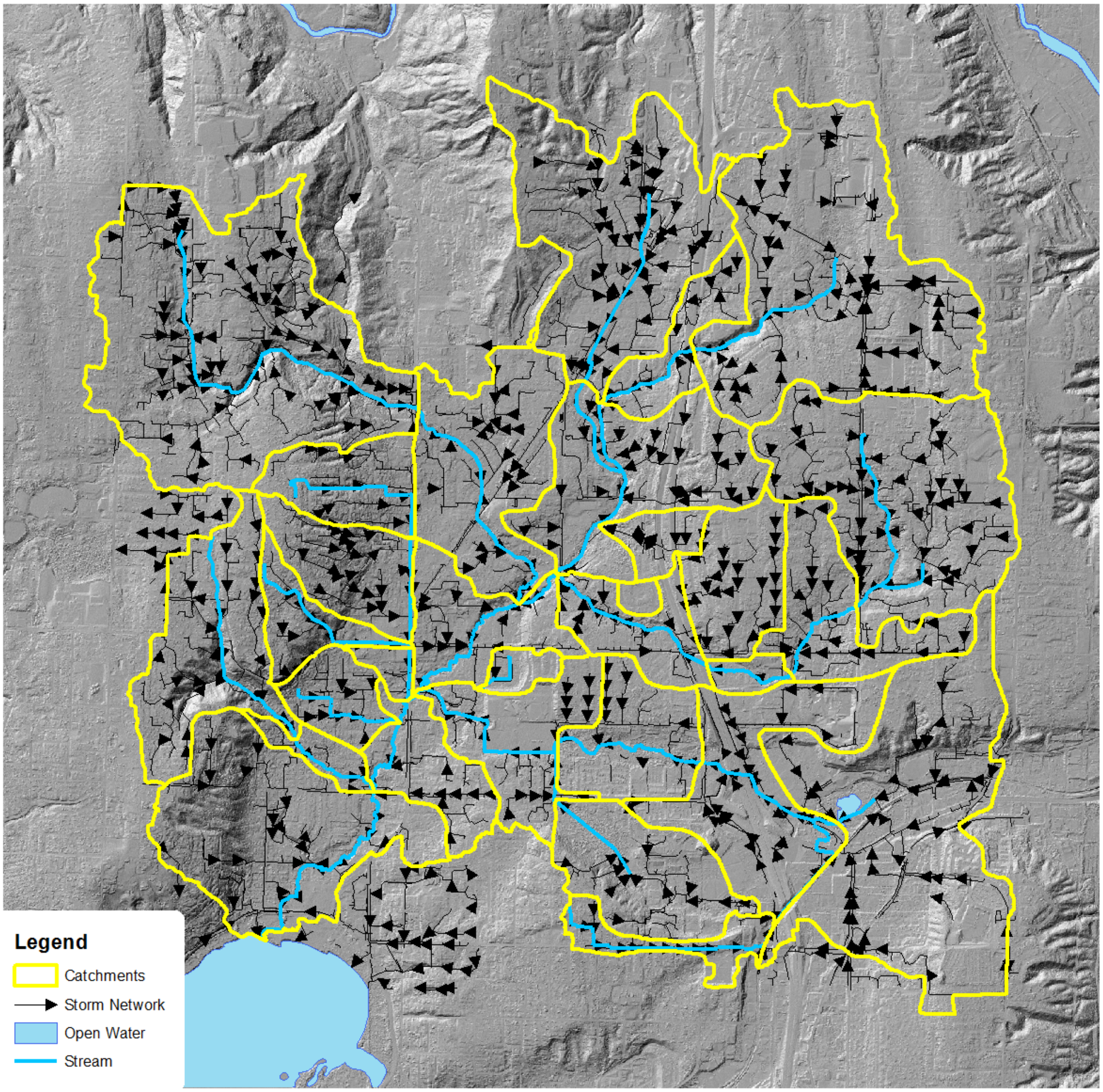


Figure 2 Map of stormwater infrastructure (City of Kirkland, 2010)

2.5 Watershed Modeling

In this study, EPA's watershed model, Hydrologic Simulation Program FORTRAN (HSPF v12), was used to simulate continuous stream flow, pollutant concentrations, and loads resulting from various mitigation scenarios. These outputs were post-processed to generate the metrics which characterize the relative effectiveness of each of the strategies.

The model was segmented into 30 catchments (Figure 2) averaging 145 acres and ranging in size from 9 to 484 acres. Internal model time steps were in 15-minute increments. Model outputs at either hourly or daily intervals were used in post processing depending

on the metric of interest. Calibration of the model was based on data collected from October 2008 to December 2009 at the eight monitoring locations detailed in Section 2.1, as well as data from an earlier period (1995 – 2010) from a site near the mouth of the creek where King County has been monitoring for other purposes.

More detailed discussions of model development, calibration, and model limitations, are provided in sections 3.4 and 3.5, and Appendix E.

2.5.1 Model Accuracy

Model accuracy was evaluated by comparing simulated to observed values for three elements: 1) stream flow rates, 2) water quality concentrations, and 3) hydrologic metrics used for predicting BIBI scores.

Flow rates, the basis of all of this study's model predictions, were scrutinized the most by applying a comprehensive suite of statistics quantifying (thus identifying) the model's strengths and weaknesses. The model accuracy was evaluated considering durations, cancelation of errors, compounding of errors, and the model's ability to predict given the observed variability in the data. Dependent on this accuracy of predicted stream flows is the model's ability to simulate instantaneous water quality concentrations used for evaluating impacts sensitive to short and long duration exposures. The amount of data available for water quality is substantially less than continuously monitored stream flows. Consequently, a subset of the stream flow statistics is applied to water quality concentrations focusing on model correlations and predictiveness.

With much of this study's emphasis on biotic measures in the stream system (section 2.7.1), a non-parametric equivalency test (Mann-Whitney *U*-test) comparing hydrologic flashiness between simulated and observed conditions (section 2.7.2) was performed providing additional validation of the model's predictiveness. A list of statistics used is summarized in Table 2 below, and a complete list of model accuracy is in the watershed calibration report (Appendix E).

Table 2 List of statistics used for assessing model accuracy based on simulated hourly timesteps.

<table>
<tr><th>Statistic</th><th>Description</th><th>Applied to:</th></tr>
<tr><td colspan="2">Model Biases</td><td rowspan="9">Flow Rates</td></tr>
<tr><td>Mean Error</td><td>Overall difference allowing for cancelation of errors.</td></tr>
<tr><td>Mean Absolute Error</td><td>Overall difference not allowing for cancelation of errors</td></tr>
<tr><td>RMSE</td><td>Square root of the square of the mean error</td></tr>
<tr><td>Mean (RPD)</td><td>Relative difference in mean error between simulated and observed</td></tr>
<tr><td colspan="2">Model Predictions</td></tr>
<tr><td>Pearson Coefficient</td><td>Measure of correlation</td></tr>
<tr><td>r-square</td><td>Measure of variance explained</td></tr>
</table>

Statistic	Description	Applied to:
Nash-Sutcliffe	Measure of correlation taking into account data variability	
Skill Score	Similar to Nash-Sutcliffe, measuring the models predictive capability	
Distribution Shifts (RPD)		
90-Percentile 75-Percentile 50-Percentile 25-Percentile 10-Percentile	Relative difference of simulated minus observed at each of the quantiles ranges.	
Other statistics		
Annual 7-Day Low Flow	(mean difference)	
	(RPD)	
Instantaneous Daily Maximums	(mean difference)	
	(RPD)	
Instantaneous Annual Maximums	(mean difference)	
	(RPD)	
RMSE	see above	Water Quality
Mean Error		
Relative Percent Difference (RPD)		
r-square		
Mann-Whitney U-Test	A non-parametric test comparing distributions	
Mann-Whitney U-Test	see above	Hydrologic Metrics

Equation 1 Mean error (cfs)

$$\sum(Q_{sim} - Q_{obs})$$

Equation 2 Mean absolute error

$$\sum|Q_{sim} - Q_{obs}|$$

Equation 3 Relative percent difference (RPD)

$$\left(\sum\frac{(Q_{sim} - Q_{obs})}{Q_{obs}}\right) \times 100$$

Equation 4 Root mean square error (RMSE)

$$\sqrt{\frac{\sum(Q_{sim} - Q_{obs})^2}{n}}$$

Equation 5 Skill Score

$$1 - \frac{RMSE}{\sqrt{\frac{\sum\left(Q_{obs} - \overline{Q}_{obs}\right)^2}{n}}}$$

2.6 Defined Removal Efficiencies for Facilities

The HSPF model developed for this study requires (among other inputs) pollutant-removal efficiency values for several pollutants for two types of water-quality treatment facilities—rain gardens and regional wet ponds. Removal efficiency estimates for pollutants considered in this study were based on a literature review with the exception of total suspended solids which used an assumed 80% removal efficiency for both types of treatment as specified in the 2009 King County Surface Water Design Manual (King County, 2009). During the course of this study, removal efficiencies continued to be under study and final values (Appendix D) are slightly different from those used in modeling. See Table 3 for the complete list of assumed removal efficiencies.

Table 3 Assumed percent removal efficiencies for rain gardens and regional wet ponds.

Parameter	Maximum Percent Removal Efficiencies	
	Rain Gardens	Regional Ponds
Total suspended solids (TSS)	80	80
Copper – particulate (Cu)	80	80
Copper – dissolved (DCu)	20	20
Phosphorus – total (TP)	50	40
Phosphorus – SRP (PO_4)	0	0
Nitrate (NO_3)	30	30
Ammonia/ammonium (NH_3)	0	0
Total Nitrogen (TKN)	10	10
Fecal coliform bacteria (FC)	0	0

Facility pollutant-removal efficiencies are inherently inexact because they depend in part on flow rates and influent pollutant concentrations, which may be highly variable. There is also variability in facility design which stems from a lack of accepted standards for design, configuration, implementation, and performance monitoring.

To characterize the variable removal efficiencies of the mitigation facilities, an empirical relationship was developed. As a result of longer residence times, ponds with larger volumes in relation to the volume of storm runoff directed to them are more effective at removing pollutants (Figure 3). A maximum limiting rate is assumed to occur when pond volumes (VB) exceeded three times the volume of the mean annual storm (VR)--defined as the annual rainfall divided by the number of storms per year. To illustrate how different

maximum rates were assigned in the model for different water-quality parameters, four alternative y-axes are included to represent effectiveness by parameter.

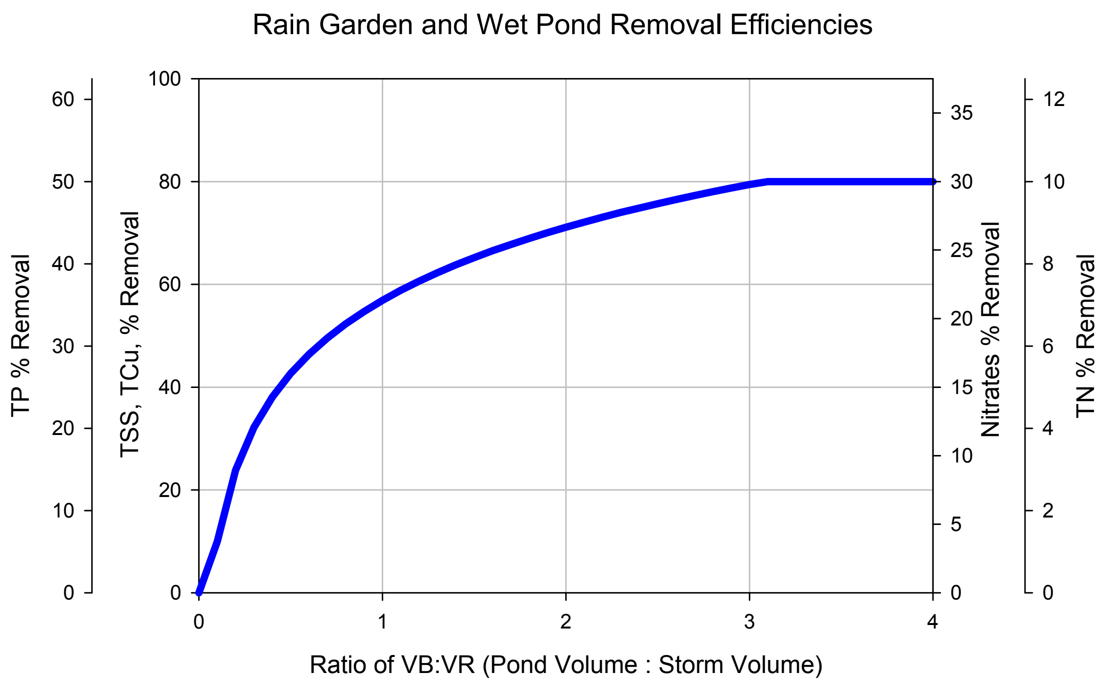


Figure 3 Removal efficiency curve for rain gardens and regional wet ponds. Further detail can found in Appendix D—Section 5.

2.7 Basin Metrics

Multiple metrics were utilized to judge the effectiveness of proposed mitigation strategies to create conditions supportive of the aquatic life beneficial use in Juanita Creek. These metrics were selected to reflect hydrologic, geomorphologic, biologic, and water quality conditions. The reasoning behind using multiple metrics is that for any given predicted outcome, there may be a limiting factor not accounted for in any individual metric (Konrad et al. 2008). Thus, this multiple metric approach was adopted to increase the robustness of mitigation strategies suggested by study results and reduce the possibility of overestimating beneficial outcomes. Furthermore, it was considered advisable that mitigation strategies be required to show improvement across all metrics to be judged adequate in meeting basin management goals. Mitigation strategies exhibiting weaker improvements for some metrics suggest that additional work would be required before adoption, either by bolstering its prescriptions if its deficiencies are sufficiently understood, or possibly by further study to determine how it could be strengthened.

These four categories of metrics (hydrologic, geomorphologic, biologic, and water quality) are not meant to be comprehensive in predicting success. Indeed, there are many other facets to stream health not accounted for in this study that can be equally limiting to the

achievement of a sustainable beneficial use. As an example, a predicted hydrologic regime may be similar to the goal of a forested condition, but without riparian habitat and adequate water quality, the predicted outcome likely would not be achieved as a consequence of other limiting conditions. However, these four metric categories have been used in previous research describing stream health in quantitative measures and are considered strong indicators of effectiveness for this study (e.g. King County 1993).

2.7.1 Biotic Metrics

In lotic systems, benthic macroinvertebrates constitute the biomass of primary consumers, process allocthonous energy sources, serve as detritivores, and filter organic matter carried in the water column (Hynes 1970). Invertebrate assemblages are sensitive to perturbations in water quality (Rosenberg and Resh 1993; Nelson and Roline 1996), hydrology (Booth et al. 2004; DeGasperi et al. 2009), and channel hydraulics (Minshall 1984; Hilderbrand et al. 1997; Pastuchova et al. 2008). Because of their sensitivity to multiple stressors in streams, macroinvertebrates have been shown to be effective ecological indicators of overall water quality and are particularly sensitive to activities such as logging, agricultural runoff, urban runoff, road crossings, herbicide application, and mining (Rosenberg and Resh 1993; Holt and Miller 2011).

The Benthic Index of Biotic Integrity (BIBI) was developed as a mulitmetric index to quantify the ecological condition of streams in the Pacific Northwest (Kleindl 1995, Karr and Chu 1999). It is based on ten metrics that represent the presence of important taxa at the sampling location. Metrics include total taxa richness, ephemeroptera richness, plecoptera richness, trichoptera richness, intolerant taxa richness, clinger taxa richness, long-lived taxa richness, percent tolerant, percent predator, and percent dominance. Each metric is assigned a score between 1 and 5, and the individual metric scores are summed to calculate a score for a given site, between 10 and 50. Total scores are binned and assign qualitative descriptions of condition (Table 4).

Table 4 Five qualitative categories of biological condition from Karr et al. (1986) as included in Morley (2000).

Condition	General Description	BIBI Range
Excellent	Comparable to least disturbed reference condition; overall high taxa diversity, particularly of mayflies, stoneflies, caddis flies, long-lived, clinger, and intolerant taxa. Relative abundance of predators high.	46-50
Good	Slightly divergent from least disturbed condition; absence of some long-lived and intolerant taxa; slight decline in richness of mayflies, stoneflies, and caddis flies; proportion of tolerant taxa increases.	38-45
Fair	Total taxa richness reduced – particularly intolerant, long-lived, stonefly, and clinger taxa; relative abundance of predators declines; proportion of tolerant taxa continues to increase.	28-37
Poor	Overall taxa diversity depressed; proportion of predators greatly reduced as is long-lived taxa richness; few stoneflies or intolerant taxa	18-27

Condition	General Description	BIBI Range
	present; dominance by three most abundant taxa often very high.	
Very Poor	Overall taxa diversity very low and dominated by a few highly tolerant taxa; mayfly, stonefly, caddis fly, clinger, long-lived, and intolerant taxa largely absent; relative abundance of predators very low.	10-17

In the Puget Sound region, the BIBI has been used extensively as an indicator of stream health by federal, state, and local agencies (Morley and Karr 2002). The BIBI has proved useful in understanding thresholds at which flow, habitat degradation, and water quality degrade stream health.

2.7.2 Hydrologic Metrics and BIBI

Alterations in hydrology have been measured using dozens of flow-rate metrics that numerically characterize the hydrologic regime into five broad categories: magnitude, timing, duration, frequency and rate of change (e.g. Richter et al 1996, Poff et al 1997). These categories are qualitatively related to channel habitat on larger stream and river ecosystems. More recent studies (Cooper 1996, Booth et al. 2004, Cassin et al. 2005, and DeGasperi et al. 2009) have focused on smaller stream systems local to the Puget Sound and begin to quantify the quality of channel habitat using biological indicators (see section 2.7.1 above), yielding a more direct linkage to measured stream health and hydrology.

Fundamentally, the metrics with the best identified correlations to BIBI measure flashiness in the system. These metrics are associated with a range of conditions beyond those normally included in routine flow assessments which normally focus on peak annual frequencies (e.g., half the 2-year through 50-year flood frequencies). In addition to eight metrics identified in DeGasperi et al. (2009), a ninth metric was included in this study based on the work of Cooper (1996). These nine hydrologic metrics, summarized in Table 5, were used to estimate responses in BIBI scores using predicted changes in hydrology under various benchmark and mitigation scenarios. For this study, catchment level BIBI scores are calculated by averaging the responses derived from each of the nine metrics. The catchment level scores are then averaged over all catchments to produce a basin level response—further referred to as the basin-wide average.

Table 5 Summary of hydrologic metrics used in this study

Metric	Name	Description
LPC	Low Pulse Count	Number of times each calendar year that discrete low flow pulses occurred
LPD	Low Pulse Duration	Annual average duration of low flow pulses during a calendar year
HPC	High Pulse Count	Number of days each water year that discrete high flow pulses occur
HPD	High Pulse Duration	Annual average duration of high flow pulses during a water year
HPR	High Pulse Range	Range in days between the start of the first high flow pulse and the end of the last high flow pulse during a water year
FR	Flow Reversals	The number of times that the flow rate changed from an increase to a decrease or vice versa during a water year. Flow changes of less than 2% are not considered
TQmean	TQ_{mean}	The fraction of time during a water year that the daily average flow rate is greater than the annual average flow rate of that year
RBI	R-B Index	Richards-Baker Index – A dimensionless index of flow oscillations relative to total flow, based on daily average discharge measured during a water year
PK2YR[1]	Peak 2-yr:Winter Baseflow	Ratio of the estimated 2-year peak flow to winter baseflow (i.e., mean flow for October through April)

[1]Relationship between metric and BIBI still in development as part of EPA WRIA 9 grant.

The responses in BIBI are derived from observed relationships between the metrics listed above and BIBI scores. Using the BIBI data from the DeGasperi (et al.) and Cooper studies, these relationships were quantified as regressions defined either as log-linear (Equation 6), linear (Equation 7), or as exponential (Equation 8) to achieve a *best fit* among the data. In addition to the listing of the coefficients for the regressions in Table 6 below, the r-square for each is also given providing context to the accuracy of these regressions predicting BIBI scores. As shown in the table, the high pulse metrics have the best correlations to the regressions with an R^2 greater than 0.63. The ratio of 2-year peak flow to winter base flow (PK2YR) had the least predictive skill with an R^2 equal to 0.22, however this metric's regression is being refined in another study and will likely change. The PK2YR regression is based on 56 data points, while the other eight metric regressions used 16.

Equation 6 Regression used for LPC, LPD, HPC, and HPD.

$$y = a + b \log_{10} x$$

Equation 7 Regression used for HPR, QR, TQmean, and RBI

$$y = a + bx$$

Equation 8 Regression used for PK2YR.

$$y = ax^b$$

Table 6 Regression coefficients (a,b) for predicting BIBI (y) from hydrologic flashiness metrics (x).

Metric (x)	Equation Used	a	b	R^2
LPC	1	45.331	-22.466	0.44
LPD	1	-5.1273	23.214	0.59
HPC	1	53.05	-30.106	0.71
HPD	1	8.9753	23.498	0.64
HPR	2	44.167	-0.1148	0.73
FR	2	66.994	-0.7664	0.42
TQ_{mean}	2	-21.493	147.3	0.47
RBI	2	38.616	-51.851	0.49
P2YR	3	57.277	-0.311	0.22

When applying this method, model results should be interpreted with an awareness of a several key assumptions and caveats. The regressions predict an *average* response—the actual responses in BIBI scores from the simulated scenarios could be lower or higher for any given basin. Further illustrating this are scatter plots of the observed data used for developing the regressions showing that drainage basins with similar hydrologic metric values might range several points in BIBI score, possibly even spanning qualitative categories as described in Table 4. <u>For this study, we are assuming Juanita Creek will respond like an average basin vis-à-vis the regressions; and if this method is applied over a broad area, the predicted responses from mitigation strategies on average should be achieved</u>.

The responses in stream health predicted by the scenarios also assume the absence of other limiting environmental conditions (e.g., lack of riparian vegetation, poor water quality, scarcity of large wood, bank instability, etc.) that may suppress BIBI scores. It is likely, however, that such limiting conditions were present within even the least disturbed watersheds used for developing the relationships between BIBI and hydrology. This possibility is reinforced by the fact that only one of all observed basins met the criteria describing a stream system in (near) *excellent* condition (Figure 4 and Figure 5). This suggests that predicted BIBI responses from the mitigation strategies may potentially be conservative (i.e., under-predicting scores) when considering strategies that address limiting factors either explicitly (within the ascribed mitigation itself) or when used in concert with other restoration efforts.

<u>Moreover, it is worth noting that there are no known studies testing the hypothesis that a watershed can be fully restored (as measured using BIBI) to a healthy stream system supportive of sustainable beneficial uses.</u> This uncertainty in competency restoring a stream system is ameliorated in this study with a multiple lines of evidence approach that

uses various indicators (hydrology, biology, water quality, and geomorphology) to evaluate the effectiveness of the mitigation scenarios.

Lastly, it is not known if any one or more of the nine hydrologic metrics are better indicators for predicting BIBI. It is possible that a low predicted BIBI for a given individual metric may indicate an actual condition limiting aquatic health which might not be recognized if only the ensemble average is evaluated.

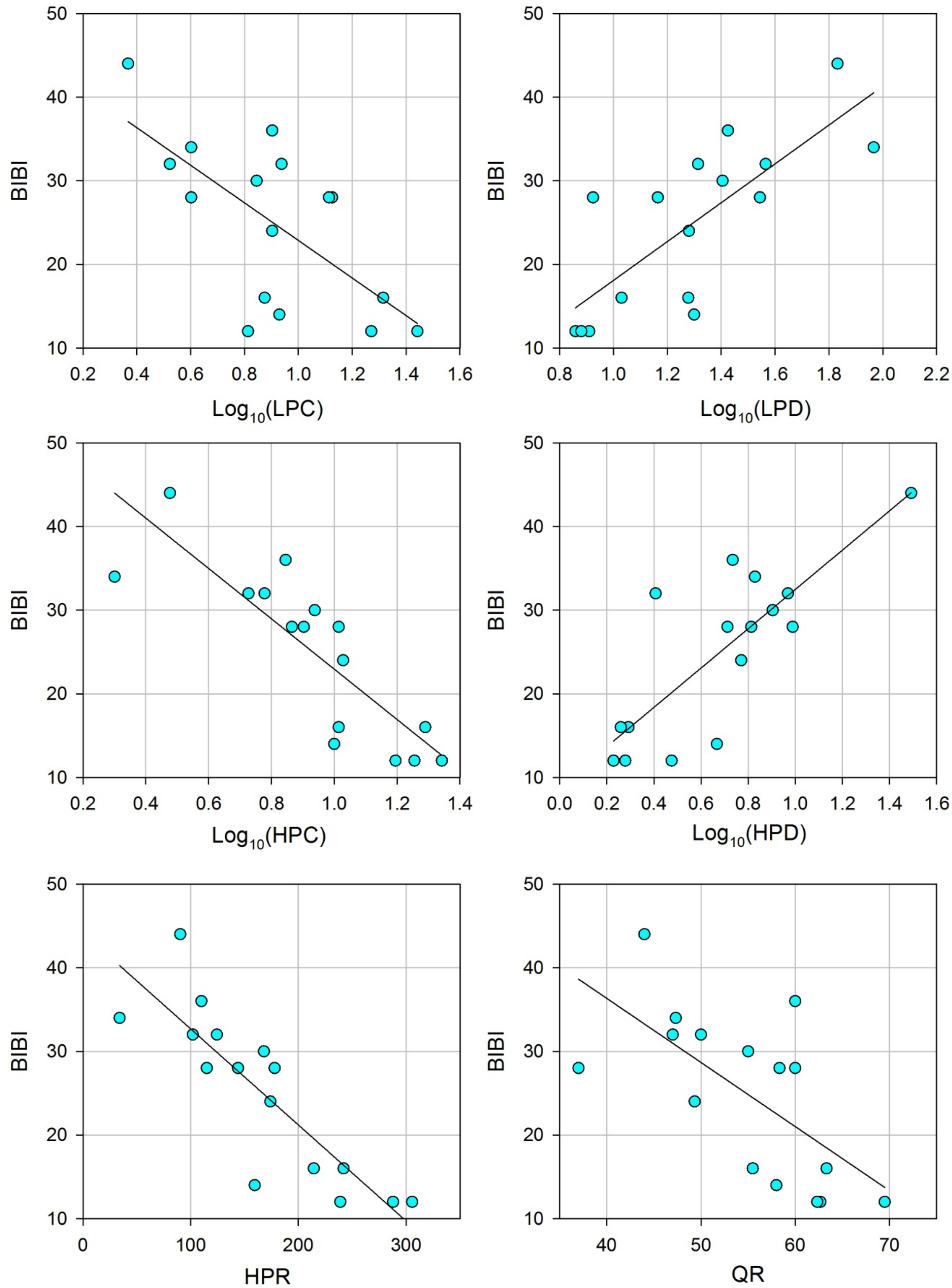


Figure 4 Scatter plots with regressions lines for LPC, LPD, HPC, HPD, HPR, and QR.

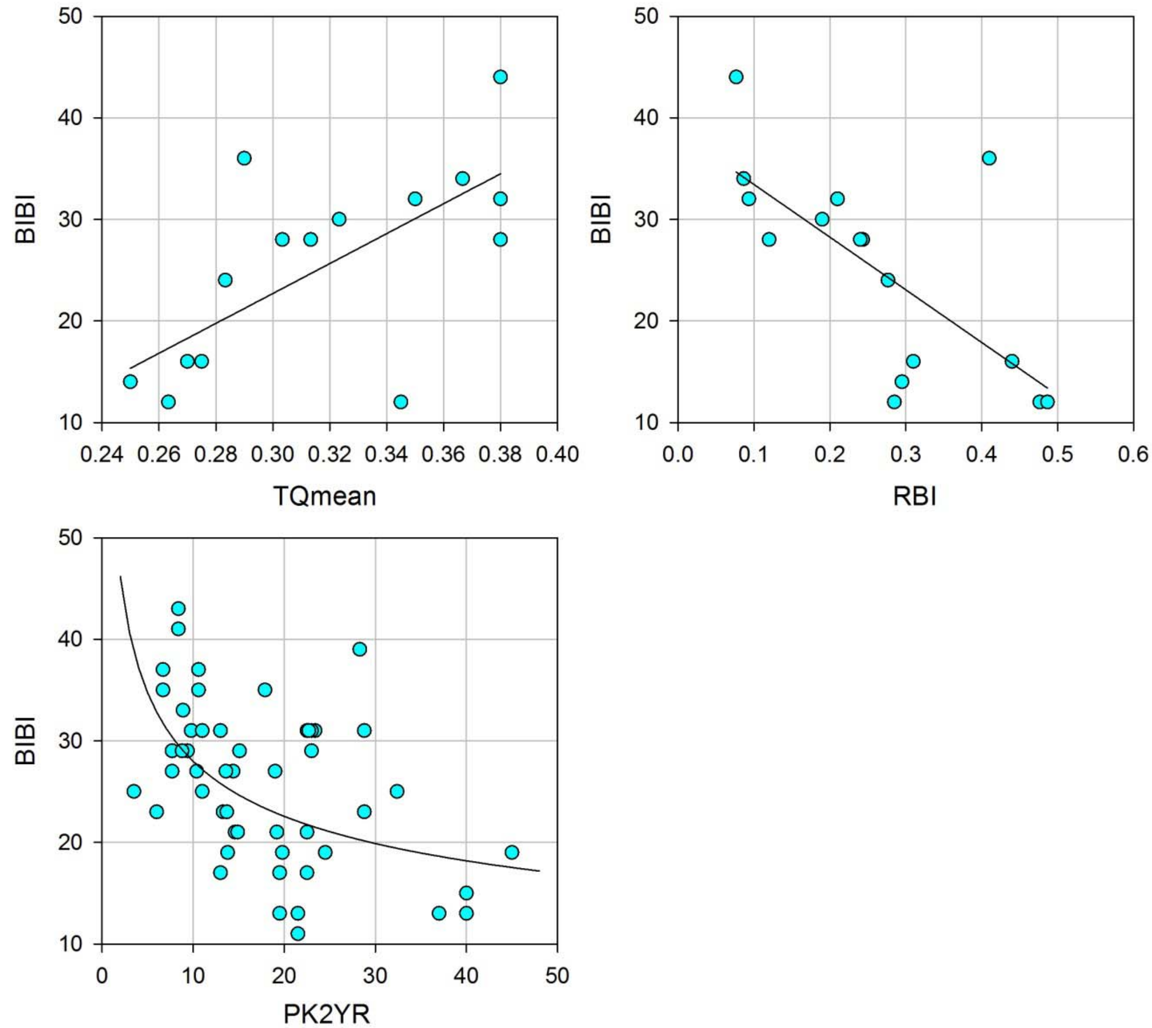


Figure 5 Scatter plots with regression lines for TQmean, RBI, and PK2YR

2.7.3 Water Quality Metrics

Water quality metrics are grouped into two types, concentrations and mass loadings. For water quality constituents with established concentration standards, the total number of simulated exceedances of water quality standards (173-201A WAC) by the HSPF model were divided by the by the number of years of simulation to compute the average number of violations per year. Where criteria (standards) do not exist, average annual loadings rates were computed (Table 7).

Parameters evaluated as concentrations include dissolved copper (**DCu**), fecal coliforms (**FC**), water temperature (**Temp**), and dissolved oxygen (**DO**). Each of the concentrations is aggregated differently; depending on the method prescribed in the state standards, and are listed below.

- Dissolved copper concentrations are calculated at hourly time increments and adjusted for hardness. In addition to hourly concentrations, a moving 4-day average is also computed for evaluating chronic conditions.
- Fecal coliforms concentrations are calculated as moving 30-day geometric means.
- Water temperatures are calculated as a moving 7-day average.
- Dissolved oxygen is evaluated using daily minimum concentrations.

Annual loading rates were computed for: total suspended solids, total copper (**TCu**), soluble reactive phosphorus modeled as ortho-phosphate (**PO_4**), total phosphorus (**TP**), nitrates plus nitrite (**NO_3**), ammonia-N (**NH_3**), and total nitrogen (**TN**).

Table 7 List of water quality metrics

Parameter	Exceedances	Loading Rates
Total Suspended Solids		mt[1]/year
Dissolved Copper	# exceedances / year (µg/L)	
Total Copper		kg/year
Orho-phosphate		kg/year
Total Phosphorus		kg/year
Nitrate		kg/year
Ammonia		kg/year
Total Nitrogen		kg/year
Fecal Coliform	# exceedances / year (50/10%/200 cfu/100ml)	
Water Temperature	# exceedances / year 55.4°F, 60.8°F	
Dissolved Oxygen	# exceedances / year (8.0, 9.5, -0.2 mg/L)[2]	

[1]Metric ton (mt) equivalent to 1000 kilograms

[2]Dissovled oxygen concentrations are seasonally specified: none summer, summer, and amount concentrations can drop below background conditions.

2.7.4 Geomorphologic Metrics

A healthy stream that is supportive of aquatic life requires gravel mobilizing events (disturbances) to maintain clean gravels, recruit large woody debris, and engage other natural channel-forming processes. In channels where disturbances occur too frequently, however, it is difficult for long- lived taxa to persist and for those biologic communities to

reach any type of equilibrium. Likewise, channels where disturbances occur too infrequently can become hardened and relatively devoid of complexity and the nutrients necessary for supporting beneficial uses (Doyle et al. 2000).

Average annual frequency of gravel disturbances was selected as a geomorphologic metric to define a target that represents a condition of sustainable beneficial uses. Gravel disturbances are defined as discrete events (of variable duration) where flow rates reach levels capable of mobilizing the median gravel size (D50) in the channel and then recede back below the same flow threshold. Magnitude and duration of the events are not considered as part of this metric—a catastrophic single event that may destabilize a system is given the same weight as a more typical event that would not do so.

The threshold of gravel mobilization is a parameter that is likely unique to every reach in the system and depends on gravel size, gravel density, channel roughness, and flow velocity.

Shear stress values at which target gravels are mobilized were calculated using identified methods of incipient motion detailed in Appendix B. A HEC- RAS model (HEC 2010) was then used to identify flow rates associated with those shear stresses at three stream reaches (Sites 203, 105, and 107) shown on Figure 6. Flow rate thresholds were calculated at each location for existing gravel sizes and a gravel size (D50 = 16.5mm) considered supportive of coho spawning habitat (Kondolf and Wolman 1993)—the target salmonid species for restoring beneficial uses in Juanita Creek.

HSPF was used to calculate the average annual number of gravel disturbances using 60 years of simulation for each benchmark (assuming existing gravel sizes) and mitigation scenario (assuming D50 = 16.5mm).

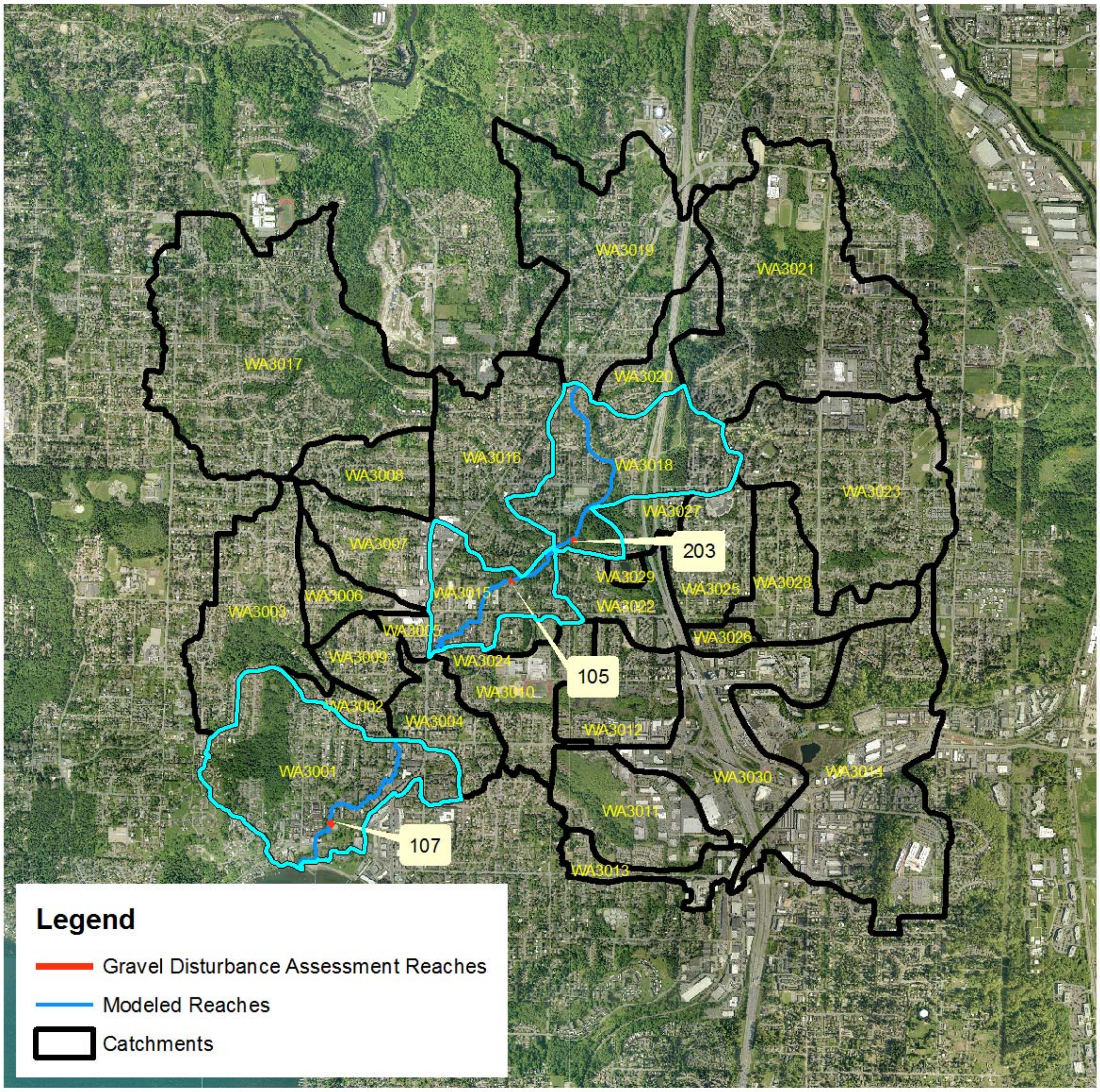


Figure 6 Location of catchments, modeled stream reaches, and specific assessment stream segments for assessment of gravel disturbances.

2.8 Mitigation Unit Costs

To create the cost mitigation curves (section 3.7.5), which plot cost (as net present value) versus performance metrics (BIBI, water quality excursions, and gravel disturbances), unit costs were developed for the construction, maintenance, and land-acquisition components required to implement each scenario. Net present value over a 40-year life cycle with an assumed four- percent inflation rate was calculated for each mitigation scenario (section 3.7.4). Table 8 is organized by facility type and includes the unit costs used for each scenario as well as the source of the unit cost data. Spreadsheets included in Appendix F of

this report contain the detailed calculations of net present value and supporting information.

2.8.1 Dry and Wet/Dry Ponds

Scenarios with wet ponds and wet/dry ponds assume that these will be public facilities that require purchase of land and ongoing public operations and maintenance. Costs for manholes, flow restrictors, and stormwater conveyance were not included in the unit costs and are likely small relative to the overall costs of regional facilities. The land costs were determined by choosing a random sample of ten residential properties from each catchment. The value of the properties was determined by researching the value of the land and the improvements from the tax assessor's records in 2011 dollars. The average cost of the land and improvements from the 30 catchments was averaged to obtain a cost per acre for the land required to site the ponds in the basin.

Soft costs include design, permitting, administrative, real-estate transaction fees, etc. and are estimated as 35% of construction costs for wet ponds and dry ponds.

The unit cost for construction is based on WSDOT bid tabulation data with the caveat that costs for any given site will be highly variable with facility size and site parameters.

The use of underground vaults for detention and water-quality treatment could be part of an implemented urban basin-wide retrofit but was not factored into any of the scenarios or associated costs. Underground vaults offer the benefits of being built within the right-of-way with reduced or no acquisition costs, reduced water temperatures as compared to large open-water facilities, and come without the limitations that may hamper large land acquisitions necessary for regional pond construction, such as waiting for properties to be available for purchase or initiating condemnation procedures. However, based upon personal discussion with sales staff from the Utility Vault Company, a typical planning estimate delivered unit cost ($10 per cubic foot displacement/storage) for a prefabricated vault (not including construction) is nearly triple the estimated combined unit cost ($3.4 per cubic foot) of the construction and land acquisition components for a regional open air pond on average.

2.8.2 Low Impact Development (Rain Gardens and Cisterns)

Scenarios that include low impact development (LID) assumed that rain gardens and cisterns (for the stand-alone cistern scenario) will be located on private properties and include a public inspection cost. Maintenance costs are considered to be private but are included in the total cost of the associated scenario. Note that standards for maintenance and operation of disparate small rain garden and cistern facilities located on private properties, and the effectiveness of such maintenance, is not yet well understood. Soft costs for rain gardens and cisterns are included in the unit cost for construction—sourced from and based on discussion with City of Seattle Rainwise Program staff.

Table 8 Elements used for defining costs for the mitigation strategies.

Facility Type and Cost Source	Land Acquisition	Construction	Soft Costs	Maintenance and Operations (Public)	Maintenance and Operations (Private)
Wet/Dry Ponds	$1.3 million/acre	$6/cubic yard	35% of construction (includes design, permitting, and administrative)	$0.004/cubic foot-year	N/A
Cost Source	Average KC Tax assessments	WSDOT Bid Tabs for excavation and haul	King County standard multiplier	King County records averaged	
Rain gardens	N/A-placed on private property	$20/square foot of rain garden footprint	Included in construction	$0.36/cubic foot-year	$1.86/cubic foot-year
Cost Source		Seattle Rainwise Program reimbursement plus 25% for program admin/soft costs		King County estimate (See Appendix F)	King County estimate (See Appendix F)
Cistern	N/A-placed on private property	$4/square foot of tributary area	Included in construction	$90.36/each cistern-year	N/A
Cost Source		Seattle Rainwise Program reimbursement		King County estimate (See Appendix F)	

2.9 Target Definitions

Using the metrics previously described, a set of targets were defined either with values identified independent of the modeling or derived from model simulated outputs characterizing a set of benchmark conditions.

Targets with values identified independent of the modeling include a BIBI level cited as supportive of sustainable healthy stream systems, water quality parameters sourced from the Washington State Administrative Code (WAC), and a range of annual gravel disturbance events associated with stable, habitat- diverse streams.

Simulated outputs for BIBI, water quality, and gravel disturbance metrics used for targets were produced for three benchmark conditions that represent various levels of disturbance in the basin.

The forested condition benchmark (Forested) represents undisturbed, pristine conditions and is used as the goal for fully restoring streams to their beneficial uses. For any given location, the metric values produced by the model for the forested benchmark are assumed to be the best that can be achieved at that location. Logic suggests that the targets identified independent of the model for BIBI, water quality, and gravel disturbances would be met in Juanita Creek and most other basins under the forested benchmark.

The limited development (65/10) benchmark is based on and referenced in early King County basin planning work which identified a disturbance with a minimum of 65 percent forest retention and a maximum of 10 percent effective impervious area (EIA) as a functional threshold marking the transition to unstable channels and resultant degraded stream conditions (Booth and Jackson, 1997). As a means of defining a 65/10 benchmark that shifts a basin and its stream channel away from the "threshold" toward stable, un-degraded conditions, the 10 percent impervious was assumed to be total impervious area (TIA) in this study, resulting in EIA of 5% being routed to the streams.

The third benchmark (LU1977) represents a level of disturbance that was in existence in 1977, used in this study as a proxy for conditions existing in 1975 when the Clean Water Act was implemented for maintaining the chemical, physical and biological integrity of the Nation's waters.

The effectiveness of the mitigation scenarios was evaluated on whether model-independent targets identified for BIBI, water quality, and gravel disturbances were achieved and by comparing mitigation scenario results to those simulated for the benchmarks. No definitive condition (i.e., Forested, 65/10, or LU1977) for establishing targets for Juanita Creek has been identified at this time.

2.9.1 Beneficial Uses

A primary goal of the project is to identify mitigation strategies that will result in stream flow and water quality conditions necessary for restoring beneficial uses and complying with state water quality standards in Juanita Creek. 'Beneficial uses' are typically referred to as 'designated uses' in the WAC (Washington Administrative Code) and include public water supply, protection for fish, shellfish, and wildlife, as well as recreational, agricultural,

industrial, navigational and aesthetic purposes. Water quality criteria are designed to protect the designated uses and are used to assess the general health of Washington surface waters and set permit limits. Aquatic life uses and the accompanying criteria are those that are considered in this project since they are the most stringent and directly related to the mitigation strategies being considered. Per WAC 173-201A-600, Juanita Creek and its tributaries have a designated aquatic life use of "Core Summer Habitat" for salmonids. The details of the Washington State Water Quality Standards are included in Section 2.9.3 of this report.

2.9.2 Biological Targets

BIBI targets used for evaluating the effectiveness of the mitigation scenarios include a cited value defined independently of benchmark conditions and the predicted BIBI scores of the benchmarks (forested, limited development (65/10), and 1977 land use). A BIBI score of 35 was defined as a target based on *EPA's Review of Washington's Water Quality Criteria: An Evaluation of Whether Washington's Criteria Proposal Protects Stream Health and Designated Uses* (Karr et al., 2003), which found that scores below this value and salmonid viability typically drops precipitously. Although Karr et al. (2003) suggests that a BIBI score of 35 sets a reasonable lower limit, further study of the relationship between salmon population viability and BIBI levels seem warranted.

Of the three benchmark conditions used for evaluating mitigation scenario results, the forested condition is given special consideration. Since it represents pristine, undisturbed conditions which are assumed to result in predicted BIBI scores that are 'the best that you can get' at a given location, it is instructive to measure the effectiveness of each scenario by normalizing their results against the maximum result produced by the forested condition. This is accomplished by dividing the basin-wide average BIBI scores predicted for each scenario by the basin-wide average predicted for the forested condition and reporting as *percent of potential maximum*.

2.9.3 Washington State Water Quality Standards (WAC)

Modeled water quality parameter targets are based on the Washington state surface water quality criteria given in WAC 173-201a, except for pollutants for which there are no surface water quality standards, in which case pollutant load reduction is the metric. Target load reductions are modeled from literature percent removal values which are almost all based on difference between treatment influent and effluent concentrations, with no flow measure given. The following table gives modeled parameters and targets.

Table 9 Modeled pollutants and target metrics

Pollutant	Comparison Metric Surface WQS WAC 173-201a	 Load Reduction	 Other Considerations
Total suspended solids (TSS)	---	X	
Copper – dissolved (DCu)	Two WQS criteria: acute and chronic Basis is hardness, which is variable	---	e.g. hardness mg/L; Cu-diss criteria ug/L chronic, acute 25: 3.5, 4.6 37.5: 4.9, 6.8 50: 6.3, 8.9
Copper – solid (TCu)	---	X	.
Phosphorus - orthophosphate (PO_4))	---	X	
Phosphorus – total (TP)	See *Other Considerations*	X	There are total phosphorus criteria for lakes, but not for surface water streams or groundwater.
Nitrate plus Nitrite (NO_3) as N	---	X	
Ammonia (NH_3)	WQS is for ammonia, and is pH dependent	X	The ratio of NH_3/NH_4 at pH 7 is 0.0057. At pH 6 the ratio is $5.7*10^{-4}$.
Total Nitrogen (TN)	---	X	
Fecal coliform bacteria (FC)[1]	GMV[2] ≤ 50cfu/100 mL ≤ 10% of all samples (or any single sample when < 10 samples exist) obtained for calculating GMV exceeding 100 cfu/100 mL.	---	SWQS is a two-part standard. If either criterion is exceeded, the WQS is exceeded.
Total coliform bacteria	---		*Not being modeled, but should mindful with regard to potential for bacteria to migrate through soil to groundwater, as this is a hydraulic pathway in the model.*

[1] WAC 173-201a-600(a)(ii).
[2] geometric mean value

2.9.4 Gravel Disturbance

A target range of 1 to 3 gravel disturbances per year was selected to represent a condition of sustainable beneficial uses based upon Doyle et al. (2000) which found this to be a range typical of stable, perennial streams in humid-regions.

2.9.4.1 Large Woody Debris

Further adjustments in estimated disturbances will result from an adjustment in assumed channel roughness. As part of this study, rating curves were developed to characterize the effective roughness in a channel with the presumed placement of large woody debris in the stream channel. Roughness increases as more LWD installations are placed per unit length (Appendix B). Additionally, orientation of LWD placement was also considered in quantifying roughness estimates. For a given retrofit strategy that did not meet the target of 1 to 3 disturbances per year, LWD densities were adjusted until targeted conditions were met.

2.10 Scenario Descriptions

Modeled scenarios for this study characterize three types of conditions: 1) benchmark, 2) existing, and 3) future. Benchmark scenarios include: 1) undisturbed fully forested, 2) assumed limited disturbance with 65 percent forest retention and 10 percent impervious surfaces, and 3) moderately developed based on 1977 land use conditions.

Existing conditions were defined based on land use interpretations from 2002 satellite imagery, with the inclusion of three existing regional stormwater facilities.

Future conditions assume full potential build-out to the most intensive land use allowed by current (circa 2010) zoning. Seven stormwater mitigation scenarios were developed that include gray and green infrastructures represented using ponds, cisterns, and rain gardens with various levels of retention and detention. One future scenario was developed with no mitigating strategies to evaluate maximum potential impacts forecasted with the models. In total, twelve scenarios were evaluated and are listed in Table 10 below.

Table 10 Summary of Scenarios

<table>
<tr><th>Scenario</th><th>Category</th><th>Description</th></tr>
<tr><td>Forested</td><td rowspan="3">Benchmark</td><td>Basin is assumed fully forested, pristine conditions</td></tr>
<tr><td>LU1977</td><td>Land use derived from 1977 aerial imagery</td></tr>
<tr><td>65/10</td><td>Basin is assumed to have evolved under early watershed planning threshold of stream stability with forest retention and limited impervious surfaces</td></tr>
<tr><td>LU2002</td><td>Existing</td><td>Existing conditions as defined with 2002 satellite imagery.</td></tr>
</table>

Scenario	Category	Description
FUTURE	Future	Full build-out of potential land use based on current zoning, no mitigation.
LEVEL2		Future land use with King County Level 2 stormwater ponds applied basin-wide.
LID40		Future land use with 40% TIA captured by rain gardens
LID80		Future land use with 80% TIA captured by rain gardens
ECY08		Ecology-proposed matching durations to 8% of the 2-year forested to the 50-year forested, using a combination LID80 and stormwater detention ponds stacked on basic wetponds applied basin-wide.
LID40+		Combination of LID40 throughout the basin and King County Level 2 stormwater detention ponds stacked on basic wetponds in three catchments
LVL2WET		Future land use with King County Level 2 stormwater detention ponds stacked on basic wetponds applied basin-wide.
CISTERNS		Future land use where roof area runoff from a mild wet season of rainfall is captured then released July-Sept each calendar year at a constant rate.

2.10.1 Scenario—Benchmark Benchmark Forested Basin (FORESTED)

Characteristics of a theoretical fully forested landscape are assumed to be representative of pristine conditions prior to any anthropogenic disturbances. For any given location, the hydrologic, biologic, geomorphologic, and water quality conditions produced by the fully forested benchmark are assumed to be the best achievable.

2.10.2 Scenario—Alternative Baseline 65/10 (65/10)

This scenario assumes development would occur while maintaining 65% forest retention, no more than 10% total impervious surfaces, and with the remaining landscape in grass. Grass and forest land cover were segregated into high (outwash) and low (till) permeability soils based on the existing underlying surficial geology (Booth and Sackett 2006) with existing wetland areas modeled as saturated soils in this and all other scenarios. There is no assumed spatial distribution (i.e., no clustering) of where and how development might occur. The 65/10 land use threshold is referenced in early King County basin planning work as marking the transition to severely degraded stream conditions in

regards to channel stability (Booth et al., 2002), albeit with the use of effective impervious area as the metric.

2.10.3 Scenario—1977 Land Use (LU1977)

Congruent with the implementation of the Clear Water Act in 1975, this benchmark is based on land use composition near that same period. Ortho-rectified photography was available from 1977 and was assumed to be sufficiently representative of 1975 conditions. The level of existing development is more similar to existing conditions (2002) than the restricted development benchmark (65/10), and significantly more disturbed than pristine forested conditions.

2.10.4 Scenario—Existing Conditions (LU2002)

This scenario modeled approximate existing land use based on 2002 land cover developed by University of Washington UERL (Alberti et al., 2004).

2.10.5 Scenario—Future Land Use Unmitigated (FUTURE)

This scenario modeled land use based upon predicted/simulated future development in absence of stormwater controls applied to the new development. It is included as a useful comparison tool (or bookend) for a worst case condition. Future conditions are assumed to be fully built-out to maximum potential development based on current zonings.

2.10.6 Scenario—Level 2 Stormwater Dry Ponds (LEVEL2)

This scenario modeled future land use with a basin-wide retrofit utilizing detention ponds sized to the 2009 King County Surface Water Design Manual (KCSWDM) Level 2 standard applied to all drainage areas. The Level 2 standard requires developed flows to be released at predeveloped rates and durations for the range of flows from 50% of the 2 year storm to the 50 year storm where pre-developed conditions are modeled as forested.

2.10.7 Scenario—LID 40% Mitigation (LID40)

This scenario modeled future land use with a basin-wide retrofit utilizing rain gardens sized per 2009 KCSWDM methodology applied to 40% of the total impervious area of the basin. Three inches of storage is provided for treated tributary impervious area and rain garden facility footprints are consistent with 1 foot of dead storage. Rain gardens were modeled with infiltration rates of 0.2 inches/hour in areas mapped as till soil and 3 inches/hour in areas mapped as outwash soil. The outwash soil rain garden infiltration rate is a long-term estimate based upon providing a compost/soil blend treatment liner between these facilities and the native outwash soils.

2.10.8 Scenario—LID 80% Mitigation (LID80)

This scenario used the same mitigation design as LID40, but applied to 80% of the total impervious area of the basin.

2.10.9 Scenario—Ecology 8% Proposed Standard (ECY08)

This scenario modeled future land use with a basin-wide retrofit utilizing a combination of rain gardens sized per 2009 KCSWDM methodology applied to 80% of the total impervious area of the basin and combined detention/wet ponds applied to all modeled overflows from the rain gardens as well as all other unmitigated surfaces in the basin. The rain garden and detention pond combination were used to achieve the Washington State Department of Ecology (Ecology) proposed 8% standard which matches post-developed flows to pre-developed flows for runoff events ranging from 8% of the 2 year storm to the 50 year runoff event assuming the predeveloped condition as forested. The wet pond volume component of the combination pond was sized per 2009 KCSWDM methodology for "Basic Wet pond" with a treatment goal of removing 80% of total suspended solids (TSS).

2.10.10 Scenario—LID 40% w/three regional ponds (LID40+)

This scenario modeled future land use with a basin-wide retrofit utilizing a combination of rain gardens sized per 2009 KCSWDM methodology applied to 40% of the total impervious area of the basin and three Level 2 combination detention/wet ponds each located in a separate catchment. The three sites were chosen based on availability of a sufficiently large tract of currently undeveloped land at a location low enough in the given catchments to treat a meaningful percentage of drain area. The identified parcels are classified currently as open space. The wet pond volume component of the combination pond was sized per 2009 KCSWDM methodology for "Basic Wet pond" with a treatment goal of removing 80% of total suspended solids (TSS).

2.10.11 Scenario—Level 2 with Wet Ponds (LVL2WET)

This scenario modeled future land use with a basin-wide retrofit utilizing combination detention/wet ponds sized to the 2009 King County Surface Water Design Manual (KCSWDM) Level 2 flow control standard applied to all drainage areas. The Level 2 flow control standard requires maintaining the durations of flows at their predevelopment levels for all flows greater than 50% of the 2- year peak flow up to the 50-year peak flow. The predeveloped condition is modeled as historic (forested) condition. The wet pond volume component of the combination pond was sized per 2009 KCSWDM methodology for "Basic Wet pond" with a treatment goal of removing 80% of total suspended solids (TSS).

2.10.12 Scenario—Residential Cisterns (CISTERNS)

This scenario modeled future land use with a basin-wide retrofit utilizing cisterns designed per the 2009 KCSWDM Appendix C methodology applied to an estimate of residential and commercial roof areas in the basin. Cisterns were sized to provide 2.5 ft. depth of storage over the treated tributary impervious area. The cisterns are modeled to completely drain the accumulated rainfall at a constant release rate during the months of July through September each calendar year. Winter rainfall that exceeds 2.5 feet (typical annual rainfall equals 3.25 ft.) overtops the available cistern storage and is routed in the model to the stream system.

3.0. RESULTS SUMMARY

This section represents a summary of field data collection and watershed model simulation results. Details of study components are presented as separate reports in the appendices. The presentation of water quality modeling results focused on the mainstem stream reaches, where the calibrated model exhibited the best fit to the available data. Simulated BIBI results are presented both as a basin-wide average response and at the tributary catchment scale (with model limitations noted where appropriate) to allow a broader spatial comparison among of the scenarios. Gravel disturbances results are focused on three specific stream reaches.

3.1 Water Quality and Quantity Monitoring

Water quality and quantity monitoring was performed as described in section 2.1 and includes data collected for stream flows, dissolved oxygen, water temperature, pH, fecal coliforms, dissolved copper, and dissolved zinc. Results[1] are summarized in the following sections and detailed in the monitoring report (appendix A).

3.1.1 Precipitation and Stream Flows

Rainfall measured at National Weather Service Sea-Tac monitoring station during water year 2009 was slightly less (34.0 inches) than average (38.2 inches) when comparing to the historical period of record. Annual volume measured at Juanita Creek (KC 27u) was similar to Sea-Tac totaling 34.4 inches. The number and volume of storms were also similar to historical averages with Juanita creek experiencing 63 storm events[2] that average 0.47 inches in volume, compared to Sea-Tac's 52 storms for the same year with an average volume of 0.56 inches. Sea-Tac's historical average is 61 events with an average volume of 0.55 inches.

Stream flows in Juanita Creek respond quickly to rainfall with rising hydrographs observed near the mouth (KC 27a) lagging an hour or less and even shorter durations occurring in the tributaries. Not accounting for attenuation of storm peaks being routed downstream, the five tributaries monitored produce approximately 60 to 70 percent of the storm peaks observed near the mouth during the wet season and more than 90 percent during the dry season. The peak runoff event during WY 2009 was approximately 190 cfs near the mouth, and a maximum 350 cfs was recorded in October 2010. Except for Billy Creek (KC 27i), the lateral tributaries respond with similar magnitudes (including Totem Lake) ranging from about 10 to 20 percent of observed near the mouth. Billy Creek has a smaller drainage area accounting for less than 10 percent.

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[1] As part of a County wide monitoring program, two of the monitoring stations in that program are within Juanita Creek. As a result, more recent data are available then reported as part of this study (King County 2012).

[2] Event must be greater than 0.15-inches in volume and a minimum of 6-hours with no measurable rainfall prior to the event.

Each of the stream gauges and precipitation gauge are plotted as time series to facilitate how Juanita Creek and its tributaries behaved during the monitoring period of this study (Figure 7).

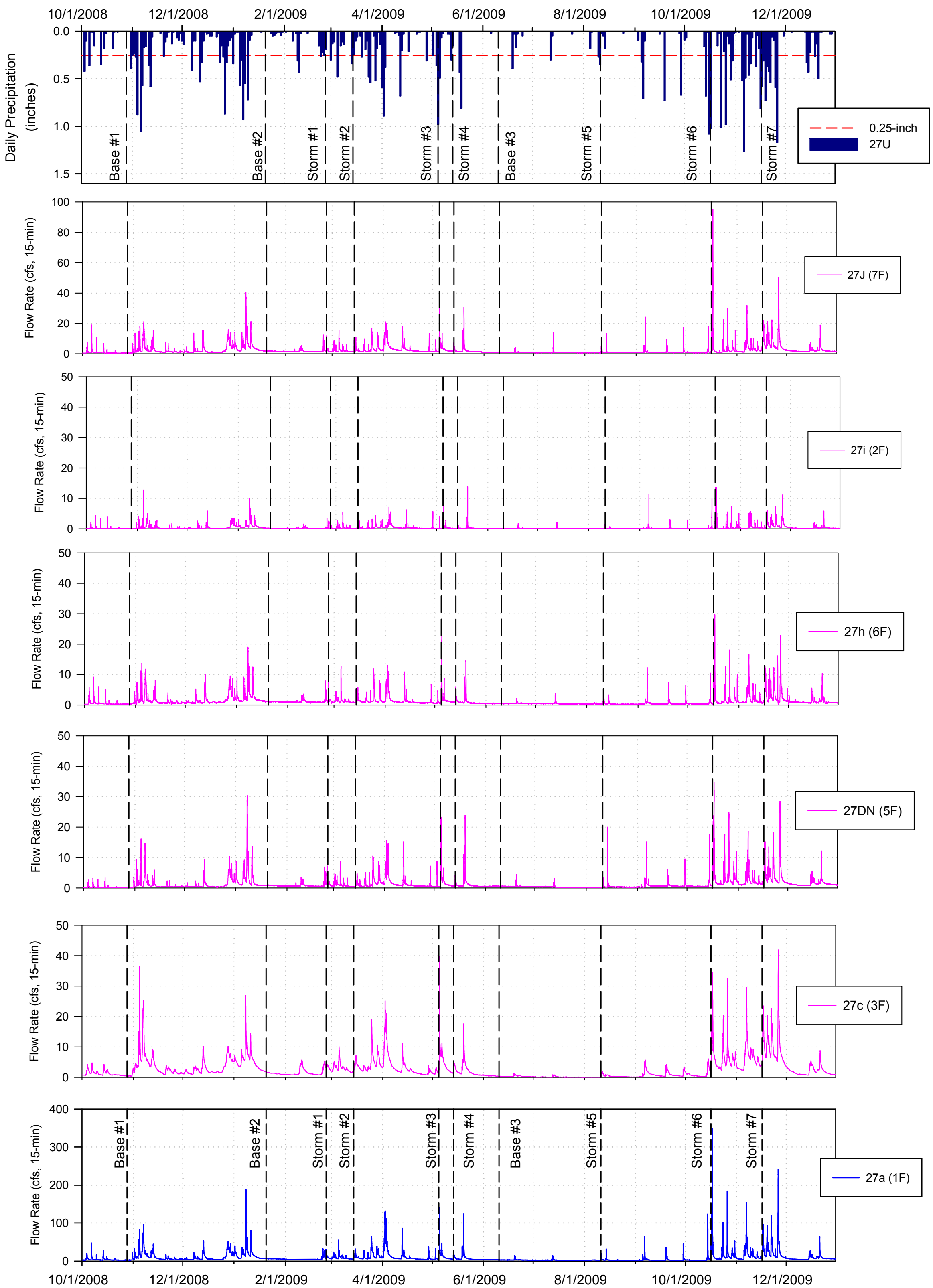

Figure 7 Summary of daily accumulated precipitation, 15-minute flow rates, and sample event dates.

3.1.2 Dissolved Oxygen

The dissolved oxygen (DO) concentrations were below the state criteria of 9.5 mg/L during both base and runoff events at all the locations, during at least one or more events. In particular, wetland inflows (4GI) had the lowest DO concentrations during baseflow events. One storm in particular (10/16-17/2009) had anoxic DO concentrations ranging from 0.7 to 3.8 mg/L. Samples taken at the inlet to the wetland (4GI) are taken in backwater conditions, which likely further suppress observations recorded.

3.1.3 Water Temperature

All continuous temperature sites had results exceeding the criterion (16°Celsius) intended to protect salmonid rearing and migration for durations ranging from 10 to 35 percent of the year, depending on the location (Table 11).

Table 11 Percent of Time stream temperatures were above state criterion.

Subbasin	Gauge Site	Percent of Time above Criterion
Mainstem	27a (1G)	24%
West Branch	27dn (5G)	10%
North Branch	27h (6G)	35%
East Branch	27j (7G)	11%

3.1.4 pH

With the exception of five samples, the pH data for all baseflow and runoff events were within the state criteria of greater than 6.5 and less than 8.5. Three samples were below pH 6.5 (~ 6.3) at the west branch (5GF) site and two samples were just under 6.5 (~ 6.47) at the inflow to the wetland (4GI) downstream of Totem Lake.

3.1.5 Fecal Coliforms

Fecal coliform were detected in all samples from baseflow and runoff events. The geometric mean of all baseflow event samples was 64 CFU/100 ml while the geometric mean of all runoff event samples was 765 CFU/100 ml, approximately an order of magnitude greater during runoff events. Both of these results are above the Extraordinary Primary Contact Recreation criterion of 50 CFU/100 ml. Storm 5 (08/10-11/2009) had the highest geometric mean of all runoff events with 3,823 CFU/100 ml in the east branch tributary (6G) with a geometric mean average for all sites of 2,479 CFU/100 ml. For the baseflow events, event #3 (06/20/2009) had the highest geometric mean value with 622 CFU/100 ml again in the east branch tributary site (6G) with a geometric mean of all sites equal to 204 CFU/100 ml. These elevated concentrations of bacteria were consistent with another study that conducted a comprehensive monitoring program tracing sources of contamination back to its sources (City of Kirkland 2009).

3.1.6 Dissolved Copper

For dissolved copper, base flow event samples did not exceed either the acute or chronic criteria. However, two of the 142 samples did exceed the acute criterion. These samples were from Storm 5 (08/20/2009) and occurred at inflows into the wetland downstream of Totem Lake (4GI) and flows from west branch tributary (5G). The chronic criterion was exceeded in 12 samples; with Storm 5 producing nine exceedances at a couple locations and site 4GI having six samples exceed criteria during a few storms. Storm 3 had two samples with values over the chronic criterion while Storm 7 had one. The other sites with sample values over the chronic criterion included east and west branch tributaries (5GF and 6G) with two samples each and near the mouth of the mainstem and Billy Creek (1G and 2G) with one sample each.

3.1.7 Dissolved Zinc

For dissolved zinc, six of the 142 storm samples exceeded the chronic and acute criterion. The location of these exceedances is different than those for dissolved copper. Storm 2 near the mouth of the mainstem (1G) had all three samples over the criteria. Storm 5 had two samples with values over the chronic criterion while Storm 7 only had one. The other sites with sample values over the chronic criterion included inflow and outflows of the wetland (4GI and 4GO) and the west branch tributary (5GF) with one sample each.

3.2 Stream Reconnaissance

The stream habitat assessment and BIBI scores observed in Juanita Creek (Appendix C) reinforce the work of Rush et al. (2002), which reported habitat conditions were degraded from "properly functioning conditions." BIBI scores in Juanita Creek in 2008 ranged from 19 to 35, or from "poor" to "fair" (Table 12), with higher scores in the headwaters and lower scores lower in the system (as shown in Figure 8 with square symbols). Observed BIBI scores used (King County 2009b) as part of the model accuracy assessment (plus four additional sites added in 2010/2011) reflect consistently poor conditions (i.e. *Very Poor* to *Poor*) with scores ranging 11 to 20. These "historical" sites are shown in Figure 8 as circular symbols.

Like other urban streams in the Puget Sound lowlands, Juanita Creek lacks suitable large wood densities (LWD). In Juanita Creek, LWD is found in much lower densities than even the low end ranges for natural conditions in the Pacific Northwest (Rush et al. 2002). Approximately 150 to 670 pieces of LWD per km are needed for properly functioning conditions, and none of the reaches sampled in Juanita Creek met that criteria (Rush et al. 2002).

Table 12 Averaged BIBI scores measured for 2008 monitoring.

Location	B-IBI Score (10 - 50)
Site 1	18. 7
Site 2	26.3
Site 3	21.3
Site 4	26
Site 5	25
Site 6	30
Site 7	35.3

Table 13 Measured BIBI scores from 2002 to 2011. Green shaded cells used for model verification.

Location	Description	2002	2003	2005	2006	2007	2008	2009	2010	2011	Avg.
1	BillyKirk1									14	14.0
2	E1186									14	14.0
3	JuanitaKirk1	12.7		12.7	15	15.3	16	17.3	19.3	16	15.5
4	JuanitaKirk2			12	13.3	14.7	16	14.7	15.3	20	15.1
5	JuanitaKirk3	14.7		10.7	12.7	12.7	14.7	19.3	17.3	18	15.0
6	JuanitaKirk4	16	14.7	13.3	15.3	14	17.3	16	18	18	15.8
7	KingsgateKirk1									16	16.0
8	WAM06600-083959								14	20	17.0
9	E1186				12	12	20	18	16		15.6

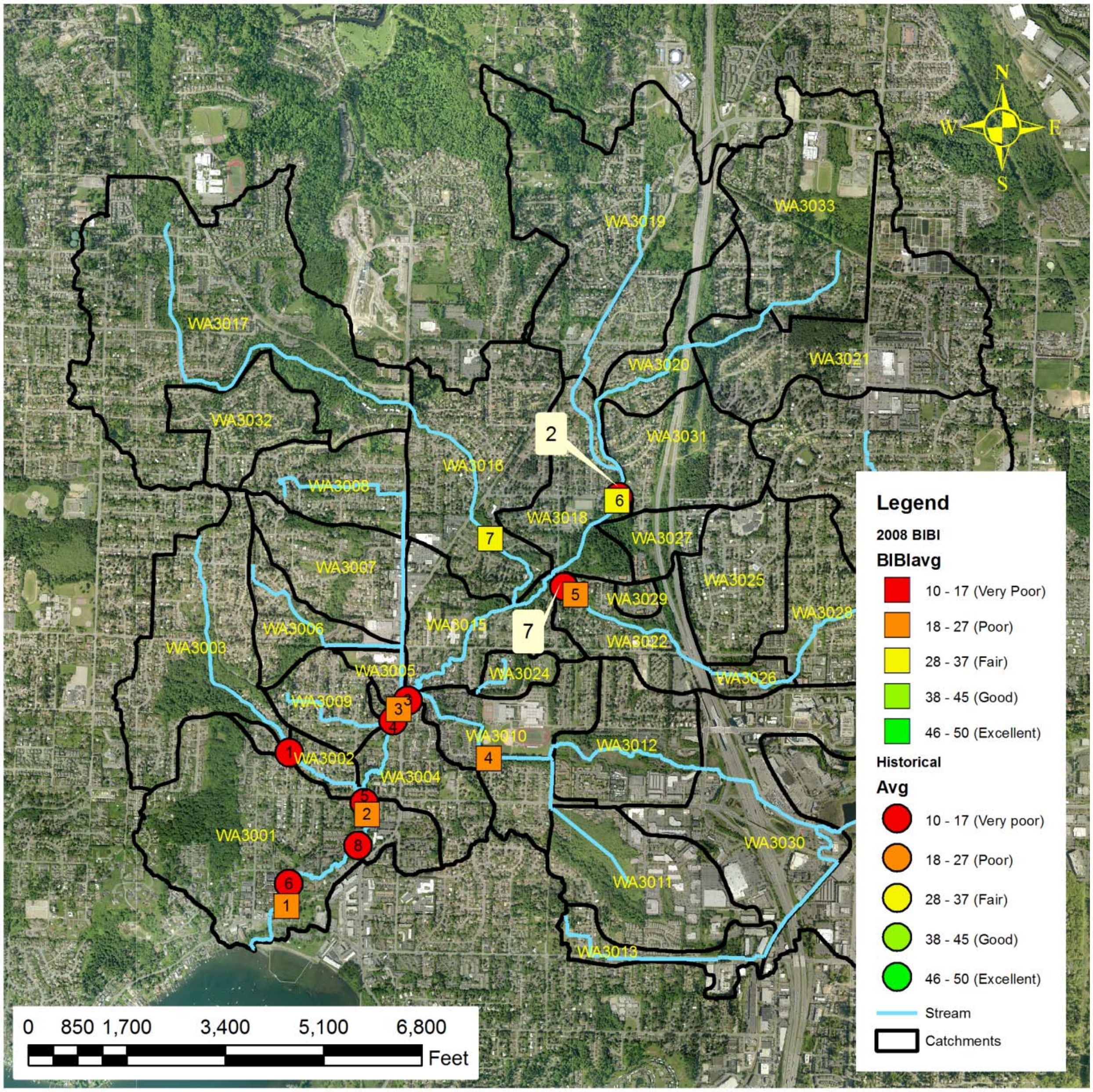

Figure 8 Measured BIBI scores in Juanita Creek for 2008 (Berge and Burkey 2009) and Historical (King County 2009) monitoring. Numbers in the symbols represent locator numbers listed in Table 12 and Table 13.

3.3 Geomorphology Assessment

As detailed in Appendix B, bank instability is pervasive throughout most of the Juanita Creek Basin watershed and is associated with areas of high discharge and, in many cases, more erodible substrates. Observed amounts of fine sediment in the system reflect substantial degradation of aquatic habitat. Bank erosion is occurring nearly everywhere in the basin with no obvious discrete point sources in terms of fine sediment source locations. It is postulated that reaches in erosive geologic substrate have the highest potential for continued erosion pointing to Billy Creek, the upper reaches of Cedar Creek, the upper

mainstem, and potentially the Juanita-Woodinville Way tributary as areas of particular concern.

Backwater effects from various culverts limit downstream sediment transport, especially gravels. Replacing culverts in this basin needs to be done with caution to avoid unintended consequences from increased downstream sediment loads, loss of gravels, and potentially bed incision in addition to already- occurring channel widening.

Visual estimates of the spatial extent of sand and fines that cover the stream bed strongly suggest that spawning habitat is limited and is a significant factor in limiting the productivity of native fish populations. When spawning gravels are embedded with fine sediment and sand, it is difficult for native fishes (especially salmonids) to construct redds. In addition, fine sediment reduces the success of incubating embryos by preventing adequate oxygen delivery and removing metabolic waste from egg pockets in redds (Everest et al. 1987; Reiser and White 1988).

Tables 7 and 8 of the Juanita Creek Basin Geomorphic Analysis in Appendix B identify specific critical reaches/locations that data suggests may be significant problem areas or those that may have the most potential for improvement. Problem types including hydrology, bank instability, fine sediment, sediment transport, and habitat are listed with corresponding locations.

3.4 Model Accuracy

As previously defined in section 2.5.1, statistics used for assessing model accuracy were applied to stream flows, water quality concentrations, and the hydrologic flashiness metrics. A comprehensive assessment of model accuracy can be found in the calibration report in appendix E.

3.4.1 Stream Flows

Model accuracy was variable depending on the simulated parameter and location, but generally calibrated well for stream flows at all six monitoring stations (Table 14 and Table 15) with mean errors ranging from -2% to +18% and root-mean-square-error (RMSE) ranging from 0.45 to 2.0 cfs for the tributaries and 8.1 cfs for the mainstem. Other than Totem Lake (27c) and East Branch (27h) tributaries being poorly calibrated for summer low flow conditions, statistics summarized in Table 14 and Table 15 and an example of scatter plots shown in Figure 9 reflect a well calibrated model and are similar in accuracy with other modeling efforts conducted in King County (e.g., King County 2003).

Table 14 Summary of simulated and observed flow rates.

Statistic	Data Source	Mainstem (27a)	Billy Creek (27i)	Totem Lake Trib. (27c)	West Branch (27dn)	East Branch (27h)	North Branch (27j)
Mean Annual Flow Rate	Obs	11.0	0.3	2.4	1.2	1.2	2.3
	Sim	10.8	0.4	2.7	1.3	1.3	2.5
Mean Peak Annual Flow Rate	Obs	184	6.8	32.5	19.7	16.4	42.0
	Sim	186	6.2	33.9	15.9	18.2	34.6
Mean 7-Day Average Daily Minimum Flow Rates	Obs	3.97	0.18	0.88	0.68	0.57	1.33
	Sim	3.91	0.22	1.67	0.74	0.88	1.56
Mean Daily Maximum Flow Rate	Obs	17.3	0.9	3.4	2.5	2.5	4.4
	Sim	19.2	0.9	4.2	2.6	2.8	5.0
Mean Annual	RPD	-2%	18%	13%	6%	8%	10%
Mean Peak Annual		1%	-8%	4%	-19%	11%	-18%
Mean 7DADM		-1%	26%	90%	9%	55%	17%
Mean Daily Maximum		11%	3%	24%	1%	12%	16%

Table 15 Summary statistics of model accuracy for stream flows based on hourly time increments

Statistic	Mainstem (27a)	Billy Creek (27i)	Totem Lake Trib. (27c)	West Branch (27dn)	East Branch (27h)	North Branch (27j)
Model Biases						
Mean Error (cfs)	-0.2	0.06	0.31	0.08	0.1	0.23
Mean Absolute Error (cfs)	4.19	0.2	1.05	0.64	0.34	0.69
RMSE (cfs)	8.1	0.45	1.99	1.35	0.74	1.49
Model Predictions						
Pearson Coefficient	0.86	0.76	0.88	0.80	0.91	0.91
r-square	0.74	0.58	0.78	0.64	0.83	0.82
Nash-Sutcliffe	0.69	0.44	0.61	0.52	0.81	0.75
Skill Score	0.45	0.25	0.37	0.31	0.56	0.50
Distribution shifts (RPD)						
90-Percentile	5%	22%	11%	27%	-2%	26%
75-Percentile	-2%	12%	-15%	-8%	3%	10%
50-Percentile	-19%	26%	-2%	-14%	20%	-5%
25-Percentile	-23%	-3%	38%	-35%	32%	-8%
10-Percentile	-13%	2%	187%	-13%	25%	-26%

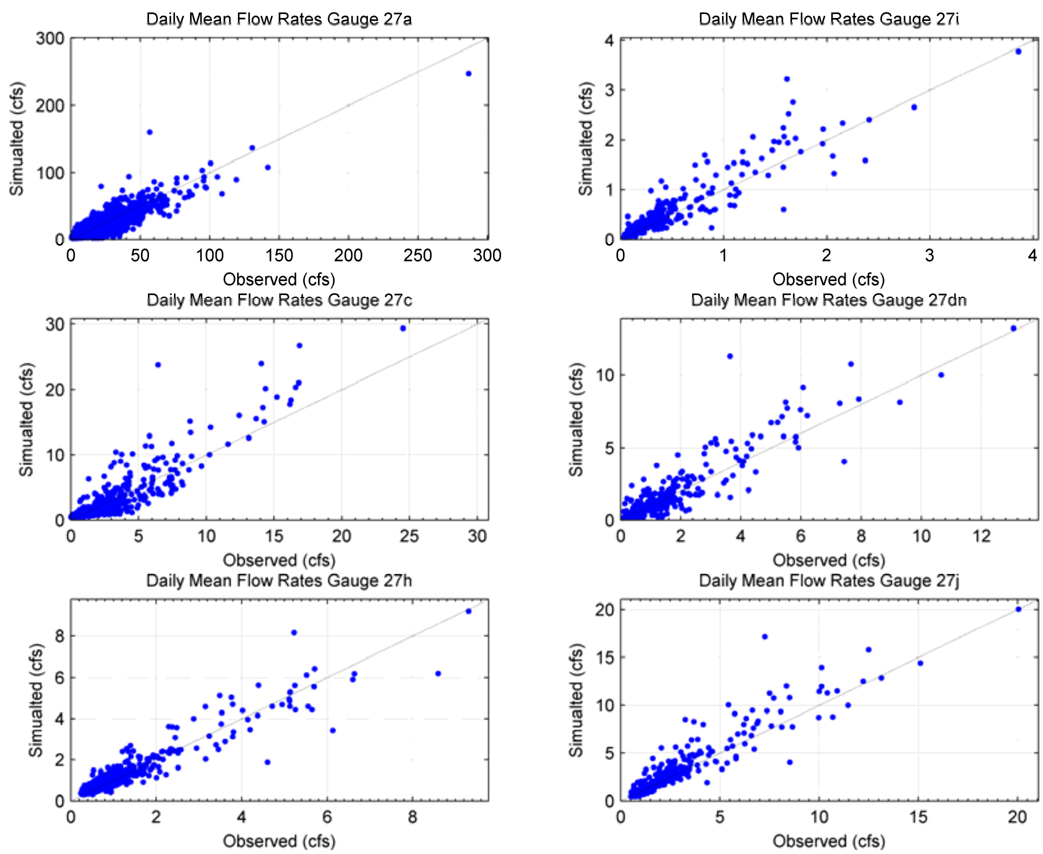

Figure 9 Scatter plots comparing observed to simulated for mean daily flow rates

3.4.2 Hydrologic Flashiness Metrics

With too few of data points on the tributaries (i.e. one data point), comparing simulated hydrologic flashiness metrics to observed was limited to the mainstem of Juanita Creek. As previously described in section 2.7.2, the Mann-Whitney *U*-test was used to compare distributions for equivalency between simulated and observed. Two of the metrics (HPC and RBI) fail the test individually with p-values less than 0.05; however, combining all of the distributions passes the test (Table 16). Accuracy of the P2YR requires a minimum of ten years of data to compute a peak 2-year return period. With insufficient data for the mainstem, accuracy is based on comparing components of the metric as a surrogate. Simulated annual peaks are approximately one percent larger than observed (Table 14) and seasonal winter flow rates were over simulated by 12 percent. These error rates seem reasonable to assume this metric "passes" the accuracy test.

Table 16 Summary of accuracy for simulated hydrologic metrics.

Metric	Description	27a *p*	Mann-Whitney
LPC	Low Pulse Count	0.08	Pass
LPD	Low Pulse Duration	0.70	Pass

Metric	Description	27a p	Mann-Whitney
HPC	High Pulse Count	0.03	Fail
HPD	High Pulse Duration	0.39	Pass
HPR	High Pulse Range	0.12	Pass
FR	Flow Reversals	0.09	Pass
TQ	TQmean	0.54	Pass
RBI	RBI Index	0.00	Fail
P2YR	2yr:winter	n/a	n/a
All above		0.70	Pass

3.4.3 Simulated BIBI

The relationships defined between the hydrologic metrics and BIBI scores (section 2.7.2) were further evaluated for Juanita Creek by comparing observed BIBI scores (section 3.2) measured at five locations to the predicted BIBI scores for the same approximate locations. Simulated scores (BIBI = 17) averaged three points higher than observed (BIBI ≈ 14) over a similar time period (Table 17). This level of accuracy is far greater than the identified accuracy among the defined relationships (section 2.7.2), thus predicting existing conditions to within a few BIBI points establishes a reasonable accuracy for further applications in this study.

Table 17 Simulated versus observed BIBI scores.

Site Code	Mean BIBI Observed	Mean BIBI Simulated
JuanitaKirk1	14	16
JuanitaKirk2	14	16
JuanitaKirk3	13	18
JuanitaKirk4	15	18
E1186	16	17

3.4.4 Water Quality

Accuracy in simulated water quality parameters resulted in limiting the focus of responses from the mitigation strategies to the mainstem when evaluating instantaneous concentrations. The r-square values for simulating instantaneous concentrations over a five-year period (2005 – 2009) were adequate, ranging from 0.45 to 0.95 for total suspended solids, total copper, dissolved oxygen, and water temperature, and with lower r-square values (< 0.2) for dissolved copper and fecal coliforms (Table 18).

The model “predictiveness” for dissolved copper and fecals would improve by removing one event (August 2009) from the observations, but for completeness there was no censorship and all data points were used in the accuracy assessment. This is evident by

plotting the simulated hourly time series and observed data collected during this study, which illustrates the closeness between simulated and observed throughout the year for base flows and runoff events (Figure 10 and Figure 11).

Table 18 Summary statistics of model accuracy for total suspended solids, total copper, dissolved copper, fecal coliforms, and water temperature for mainstem Juanita Creek.

Statistic	TSS (mg/L)	T.Cu (ug/L)	D.Cu (ug/L)	Fecals (cfu/100ml)	DO (mg/L)	Temperature (deg-F)
RMSE	67.95	10.18	2.03	1499	0.82	3.31
Mean Error	-24.87	0.00	0.81	-402	-0.45	-2.78
Mean Error (RPD)	-43%	0%	39%	-49%	-4%	-5%
r-square	0.66	0.45	0.15	0.18	0.69	0.95

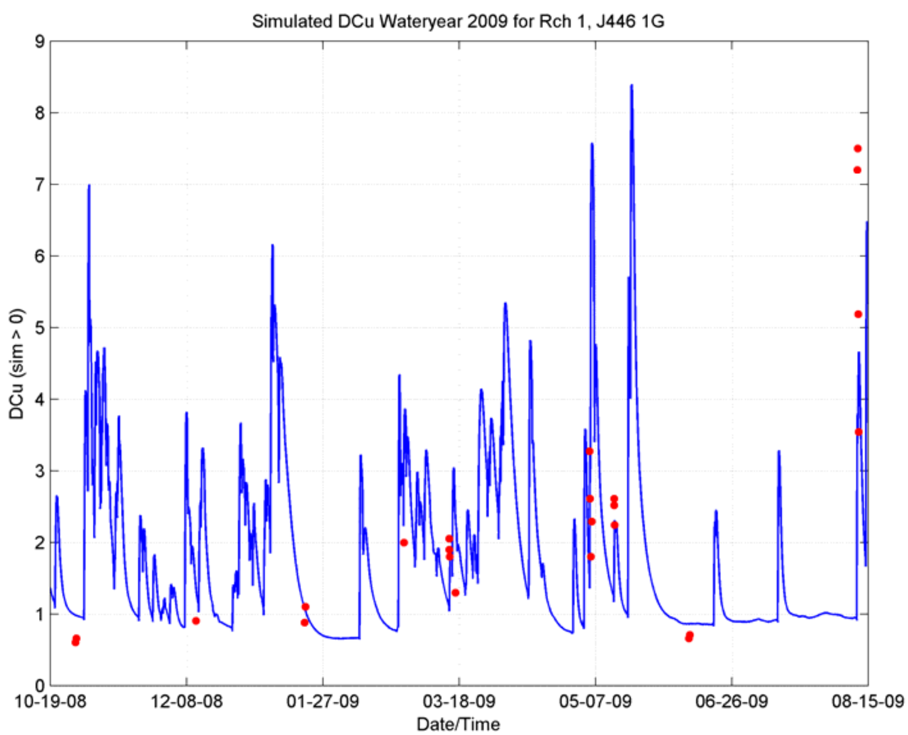

Figure 10 Mainstem simulated and observed dissolved copper during the monitoring period of this study (water year 2009).

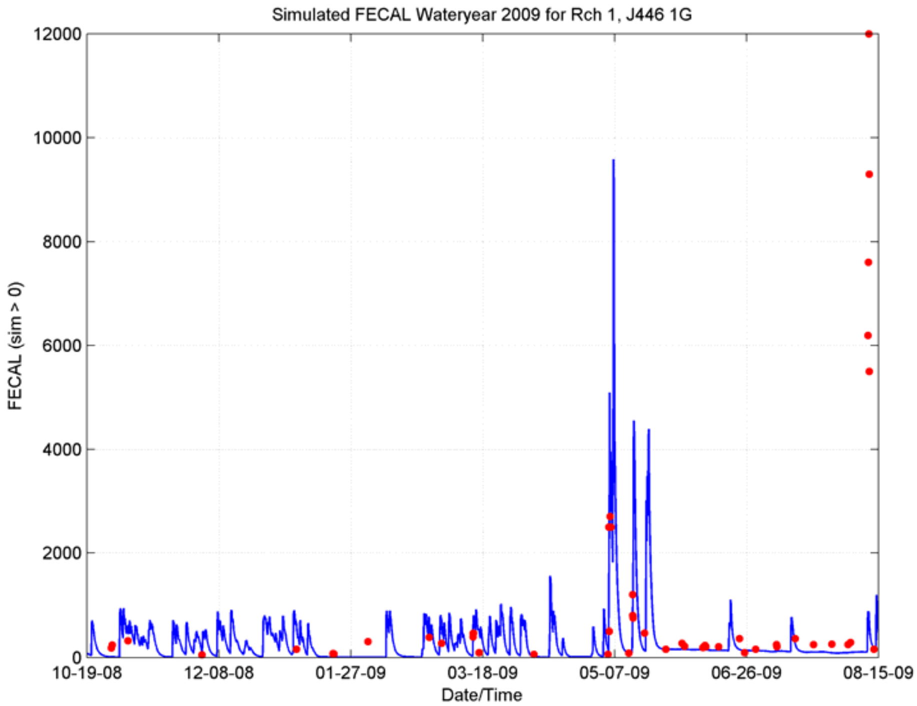

Figure 11 Mainstem simulated and observed fecal coliforms during the monitoring period of this study (water year 2009).

Mann-Whitney *U*-test on annual loading metrics (i.e. nutrients) result in three of the four parameters validating simulated distributions are equivalent to observed (Table 19). However, nitrates for this study are secondary in priority, and failing this test does not adversely affect possible outcomes from this study. Further detail on model accuracy can be found in Appendix E.

Table 19 Summary of Mann-Whitney U-test for N, NH_4, PO_4, and TP for the mainstem.

Mann-Whitney *U*-test		
Parameter	*p*	Test
Nitrates	0.01	Fail*
Ammonia-N	0.24	Pass
Orthophosphorus	0.41	Pass
Total Phosphorus	0.31	Pass

*Passes all headwater tributaries
(West = .75, North = .57, East = .08)

3.5 Model Limitations

Accuracy of the water quality simulation was inconsistent among the tributaries and the parameters. Thus, outcomes from simulated scenarios were limited to the mainstem of Juanita Creek.

Simulated flow rates at the six monitoring locations were good to exceptionally accurate (see section 3.4 and Appendix E) over the entire hydrologic regime, based on continuous hourly data over more than a year at the tributaries, and multiple years for the mainstem. What further substantiates the distributed accuracy beyond the points of measure is in the method of calibration performed for stream flows and water quality. With not enough identified differences in the landscape to warrant unique parameter adjustments (i.e. multiple calibrated sub-models) for each of the calibration points, adjustment made affected all catchments and subbasins.

Gravel disturbance analyses were limited to three locations within the study basin. True hydraulic characteristics will be unique for any given site in the basin and can be highly variable. Thus, these three locations are surrogates for what is assumed typical in the basin, but true basin wide responses to the various mitigation strategies are unknown and should be evaluated as such.

BIBI limitations in application for forecasted responses have been previously articulated and can be found in section 2.7.2.

3.6 Benchmark Scenario Simulation Results

Results for three benchmark scenarios representing different levels of disturbance in the basin (Table 20) were calculated for comparison with the proposed mitigation scenarios and discussed in the sub-sections that follow.

Table 20 Summary of benchmark simulation results per metric

Metric	Units / Description	Benchmark Results		
		FORESTED	65/10	LU1977
Total Suspended Solids	mt/year	0.1	0.6	4.4
Dissolved Copper	# exceedances / year	0.0	1.0	13.4
Total Copper	kg/year	0.1	0.4	1.3
Soluble Reactive Phosphorus	kg/year	1.9	3.6	8.4
Total Phosphorus	kg/year	7.7	11.2	17.8
Nitrates	kg/year	35.4	45.3	281
Ammonia	kg/year	5.0	4.8	7.5
Total Nitrogen	kg/year	101	129	386
Fecal Coliforms	# exceedances / year 50/10/200 cfu/100ml	101	163	332
Water Temperature	# exceedances / year 55.4°F, 60.8°F	46	50	60
Dissolved Oxygen	# exceedances / year 8.0, 9.5, -0.2 mg/L	0	60	68
Gravel Disturbance (D_{50} = 16.5 mm)	(# / year) at sites 107, 105, and 203 respectively	0.5, 0, 0	5, 0, 0	30, 8, 3
BIBI	Numerical Score	38	29	19
	Stream Health Description	Good	Fair	Poor

3.6.1 Forested Condition Benchmark (FORESTED)

Metric values calculated for the forest condition benchmark are generally consistent with the paradigm of pristine conditions. This scenario resulted in low pollutant loadings and water quality exceedances for two of the four measured parameters—fecal coliform bacteria and water temperature. A modest average riparian shading of 60% was used in the calibrated model and may partially explain the water temperature exceedances since a fully forested basin would likely have shading of 90% (King County 2005, Ecology 2011). Simulated BIBI scores at the sub-catchment level ranged from 33 to 43 with a resultant basin average of 38--equating to a category of "Good" ("Excellent" ranges from 46– 50) and exceeding the model-independent target score of 35. Gravel disturbances under the forested condition were less frequent than the target of 1-3 disturbances per year. This is most likely a result of using the modern-day channel geometry instead of the likely narrower channel that would have been present in a fully forested watershed. This emphasizes the likelihood that restoring hydrology to forested conditions would also require modifying channel geometry or allowing for an extended period of natural re-equilibration.

3.6.2 Limited Development Benchmark (65/10)

The 65/10 benchmark results failed to meet State water quality standards for all four metrics with defined values: dissolved copper, fecal coliform bacteria, dissolved oxygen, and water temperature. Pollutant loading rates are significantly elevated as compared the forested condition for most metrics. The simulated basin average BIBI score is 29 (i.e., "Fair").

3.6.3 1977 Land Use Benchmark (LU1977)

Similar to the 65/10 benchmark, the LU1977 benchmark results failed to meet water quality standards for all metrics. Pollutant loadings are multiples greater than forested conditions and simulated BIBI scores are reflective of a highly disturbed stream system with a basin average score of 19 (i.e., "Poor"). These simulated conditions suggest beneficial uses are not achievable.

3.7 Mitigation Scenario Simulation Results

Results for the seven mitigation scenarios are summarized in the following sub-sections by metric: BIBI, water quality exceedances and loadings, and gravel disturbances. Benchmark scenario results are also included for convenience and comparison.

3.7.1 Simulated BIBI Results

Simulated BIBI scores indicating stream health were calculated from hydrologic statistics derived from simulated discharges at each catchment outlet. Table 21 provides a summary of the simulated scores by catchment for benchmark and mitigation scenarios. For this study, the maximum basin-wide average BIBI score (38) was generated from the fully

forested scenario, while the lowest scores (16 and 17) were generated from the future unmitigated and existing condition scenarios, respectively.

Simulated BIBI scores for any given scenario are generally similar among the numerous catchments with the worst scores located in pipe-dominated conveyances (i.e., catchments WA3008 and WA3027) and better than average scores found in the Totem Lake tributary basin (catchments WA3010, WA3011, WA3012, and WA3014) which has significant amounts of natural storage. The standard deviation among the catchments for each scenario ranged from 2.2 to 4.6—the best catchment level scores were generally 7 points above the basin-wide average and the worst scores 6 points below the basin-wide average.

Table 21 Summary of simulated BIBI scores by catchment.

Catchment	FORESTED	65/10	LU1977	LU2002	FUTURE	LEVEL2	LID40	ECY08	LID40+	LID80	LVL2WET	CISTERNS
WA3001	39.8	30.6	21.1	18.7	17.1	24.2	21.4	36.2	22.1	29.7	24.6	18.7
WA3002	31.3	23.2	16.0	14.4	13.6	19.1	18.1	31.8	18.1	26.0	20.0	15.2
WA3003	31.4	24.2	16.6	14.7	13.7	19.8	18.0	35.0	18.0	26.1	20.1	15.4
WA3004	40.2	30.6	21.0	18.8	17.0	24.2	21.4	36.7	22.4	29.6	24.9	18.9
WA3005	35.4	25.6	14.9	14.1	13.6	19.4	18.3	30.2	18.3	26.3	20.4	15.2
WA3006	37.2	28.1	16.1	15.7	14.9	21.9	18.6	29.3	18.6	26.9	22.7	15.6
WA3007	34.7	24.3	14.6	13.8	13.6	18.3	18.4	30.6	18.4	26.6	19.2	15.2
WA3008	32.3	21.6	13.6	13.1	12.9	16.0	18.0	26.6	18.0	26.6	17.2	15.6
WA3009	38.0	28.4	17.1	15.2	14.7	18.3	19.1	29.6	19.1	27.2	19.9	15.6
WA3010	40.3	35.2	29.0	24.1	22.2	29.4	26.4	39.8	26.4	34.6	29.8	23.7
WA3011	40.3	29.7	26.8	15.7	14.6	23.1	18.4	35.2	18.4	27.8	23.7	16.2
WA3012	40.1	36.2	29.4	24.8	23.3	30.4	27.4	41.7	27.4	35.6	30.6	24.8
WA3013	39.4	30.6	19.1	15.9	15.4	26.3	19.6	36.9	19.6	28.3	27.2	17.0
WA3014	41.1	31.4	22.6	16.8	15.8	25.8	20.0	41.0	20.0	28.7	25.9	18.1
WA3015	40.3	29.6	19.1	17.0	15.8	22.9	19.4	36.1	20.9	28.3	23.3	17.2
WA3016	39.0	26.2	19.0	16.2	15.4	19.7	19.9	35.7	21.4	29.1	20.4	18.1
WA3017	40.6	27.8	21.1	16.3	15.4	20.8	19.1	38.9	21.0	28.2	21.2	17.2
WA3018	40.3	29.9	18.3	17.4	15.9	23.0	19.4	35.1	21.7	28.2	23.4	17.2
WA3019	38.8	29.2	18.9	16.6	15.0	22.6	19.6	32.7	19.6	27.6	22.9	16.0
WA3020	41.1	30.7	18.9	18.6	16.2	24.3	20.0	39.1	23.0	29.3	24.4	18.3
WA3021	41.0	30.8	19.3	18.9	16.3	24.4	20.4	40.8	23.8	29.7	24.7	18.7
WA3022	38.7	28.1	17.3	15.7	15.1	23.9	19.7	37.0	19.7	27.6	24.3	16.6
WA3023	40.4	27.7	17.1	15.9	15.3	24.0	19.3	40.4	19.3	27.4	24.7	16.4
WA3024	41.4	30.1	17.6	18.3	14.9	22.2	18.8	31.3	18.8	26.6	24.0	18.0
WA3025	40.2	27.4	16.3	15.2	14.8	24.0	19.1	36.3	19.1	27.2	24.7	16.1
WA3026	39.8	29.1	18.6	16.6	15.9	24.8	20.1	40.3	20.1	28.1	25.1	17.2
WA3027	35.3	26.3	14.4	14.1	12.4	22.7	16.3	26.4	16.3	24.7	22.9	12.8
WA3028	39.9	29.2	18.7	16.6	16.0	24.8	20.4	40.9	20.4	28.4	25.0	17.4
WA3029	30.0	27.3	29.8	21.9	21.2	27.6	24.0	31.1	24.0	28.8	29.6	23.4
WA3030	40.8	31.0	19.8	16.1	15.2	25.6	19.6	41.1	19.6	28.1	25.9	16.7
Average	38.3	28.7	19.4	16.9	15.8	23.1	20.0	35.5	20.5	28.2	23.8	17.4

Category	Very poor	10-17	Poor	18-27	Fair	28-37	Good	38-45	Excellent	46-50

Basin-wide average BIBI scores for the mitigation scenarios grouped mostly into one category of stream health—poor. Two of the scenarios showed marked improvements above the existing conditions category of very poor/poor by elevating stream health either to the low end of fair (LID80) or to the high end of fair (ECY08). Only the ECY08 mitigation scenario elevates the basin-wide average BIBI to a score above the identified target of 35—a threshold level indicated to be necessary for sustainable healthy stream systems (Karr et al., 2003). The LEVEL2 and LVL2WET scenarios produced similar results with simulated BIBI scores of 23 and 24, respectively. Although BIBI levels are raised by nearly 7 points from existing conditions, the poor category of stream health resulting from the LVL2WET scenario is notable given this scenario represents current (2005 SMMWW) stormwater standards. The LID40 and LID40+ scenarios resulted in nearly identical BIBI scores near 20, while the CISTERN scenario showed the least improvements by maintaining poor existing conditions into the future (Figure 12).

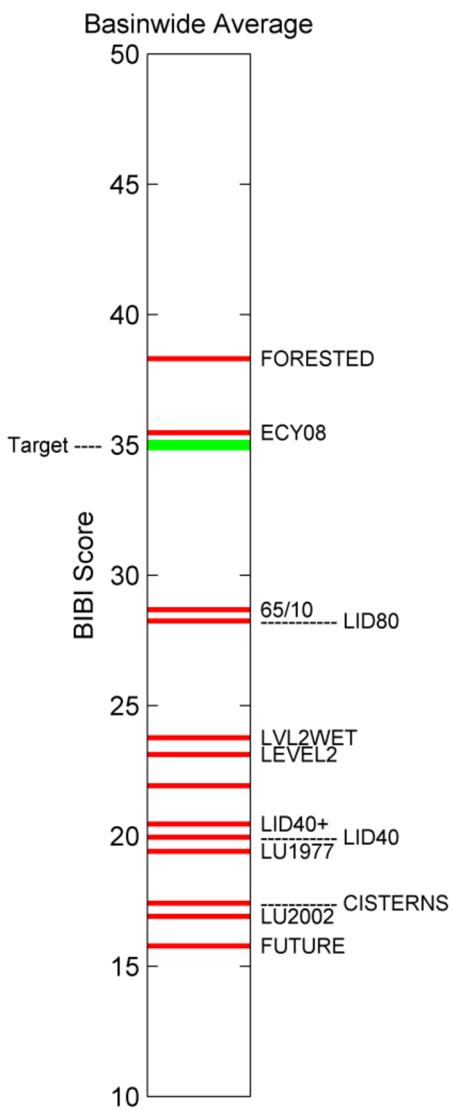

Figure 12 Simulated basin-wide average of BIBI scores per scenario.

3.7.1.1 Percent of Potential Maximum

As detailed in the methods section 2.9.2, 'percent of potential maximum' is used to measure the effectiveness of each mitigation scenario by normalizing their simulated BIBI results against the maximum result produced by the forested condition (Figure 13). The results show that the best performing scenario (ECY08) is impressively effective (93%), as might be expected given that this scenario is designed to match peaks and durations produced under forested conditions for a broad range of runoff events beginning at 8% of the 2-year storm.

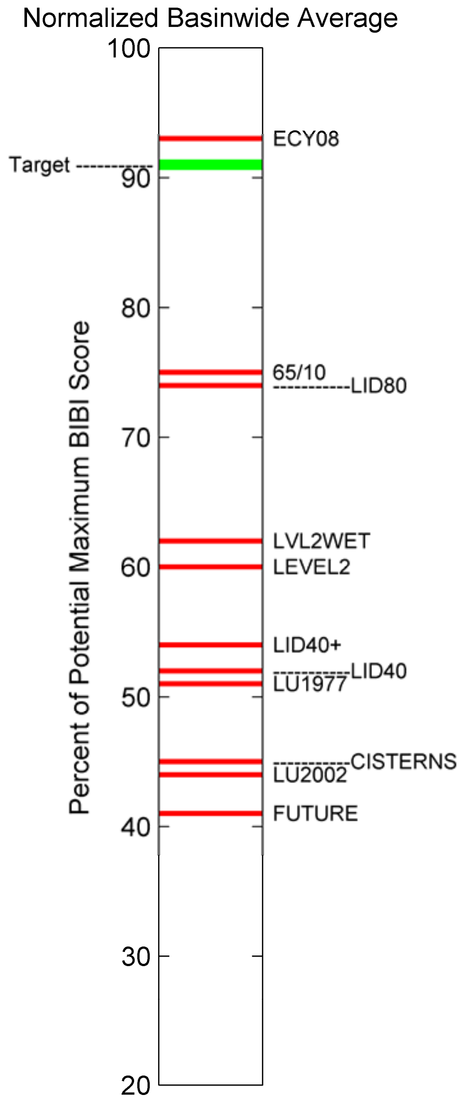

Figure 13 Simulated basin-wide average percent of potential maximum BIBI per scenario.

3.7.1.2 Simulated BIBI Results per Individual Hydrologic Metric

In addition to using catchment and basin-wide averages of BIBI scores that are based on averaging the BIBI values determined for each of the nine hydrologic metrics together, it is also instructive to evaluate how the individual metrics respond to the mitigation scenarios at a basin-wide scale. It is possible that a low outlier among the nine hydrologic metrics could indicate an actual condition limiting aquatic health and this might not be recognized if only the ensemble average is evaluated. Among the nine metrics, two of them (High Pulse Count and High Pulse Range) were consistently projecting lower BIBI scores than the other seven. In fact, simulated BIBI scores for High Pulse Range are categorized as very poor for all scenarios except for ECY08, Forested, and 65/10 (Table 22).

Table 22 Simulated BIBI responses by NFP metric and by scenario.

Metric	FOREST	65/10	LU1977	LU2002	FUTURE	LEVEL2	LID40	ECY08	LID40+	LID80	LVL2WET	CISTERN
LPC – Low Pulse Count	35	24	20	19	17	20	26	37	29	36	20	19
LPD – Low Pulse Duration	33	19	18	17	17	19	17	23	16	17	19	13
HPC – High Pulse Count	43	27	13	11	11	17	12	33	12	21	17	11
HPD – High Pulse Duration	35	27	20	19	19	28	21	44	22	33	29	19
HPR – High Pulse Range	38	28	12	10	10	10	10	24	10	15	10	10
FR – Flow Reversals	43	29	25	24	23	27	26	41	26	30	28	25
TQ - TQ_{mean}	39	38	27	20	17	32	26	44	27	40	33	28
RBI – R-B Index	36	32	17	12	11	24	18	35	19	31	25	13
P2YR – 2yr:Winter	42	33	22	19	17	30	23	38	23	32	31	18
Average	38	29	19	17	16	23	20	35	20	28	24	17

Category	Very poor 10-17	Poor 18-27	Fair 28-37	Good 38-45	Excellent 46-50

3.7.2 Simulated Water Quality Results

Simulated water quality results are evaluated using two methods, 1) the number of exceedances in chronic and/or acute concentrations per year, and 2) the annual loading rates. Both methods average 60 years of continuous simulations and are compared to modeled conditions for the three benchmark scenarios: FOREST, 65/10, and LU1977 for the mainstem of Juanita Creek with the exception of LEVEL2. Dry ponds are assumed to have no water quality benefit, thus LEVEL2 is omitted for this part of the reporting. Dissolved copper acute exceedances are based on hourly intervals, whereas chronic exceedances are four-day intervals. Thus every hour concentrations are above the acute criteria is an exceedance and every four days the concentration is above the chronic concentration is an exceedance. Water temperature, dissolved oxygen, and fecal coliforms are all based on daily intervals.

3.7.2.1 Exceedances

The forested benchmark scenario had no exceedances of dissolved copper, 65/10 had three events with enough hours of acute to average one hour of exceedances per year, and LU1977 had on average 13 per year consisting mainly short durations exceeding acute criteria. Simulated water temperature and fecal coliforms consistently exceeded criteria each year for all three benchmarks, with increasing exceedances with increasing development (Table 23). Exceedances in dissolved oxygen occurred in both the 65/10 and LU1977 benchmark scenarios, while FOREST produced none. The 65/10 scenario had exceedances with 1-day minimum concentrations that were a result of daily differences slightly greater than (-) 0.2 mg/L which is part of the criteria defining an exceedance.

Table 23 Benchmark scenario values for annual exceedances (acute and chronic combined) for dissolved copper, water temperature, dissolved oxygen, and fecal coliforms.

Parameter	Avg. # exceedances per year		
	FORESTED	65/10	LU1977
Dissolved Copper	0.0	1.0	13.4
Water Temperature[1]	46	50	60
Dissolved Oxygen	0.0	60	68
Fecal Coliforms	101	163	332

[1]Riparian shading kept constant among scenarios

<u>Dissolved Copper</u>

All mitigation scenarios reduced the number of exceedances in dissolved copper concentrations by 60% or more when compared to unmitigated future conditions with the exception of the CISTERN scenario which resulted in a slight increase (Table 24). This increase is likely a result of long detention times and no effective removal of solids allowing for desorption of copper in the sediments to dissolve back into solution before being released in the summer months. This return of copper to solution also occurs in the LVL2WET scenario but is compensated by the scenario's removal efficiency of solids and the copper attached to the solids. Only the ECY08 scenario combining dry ponds, wet ponds, and rain gardens fully mitigated all concentrations reducing the number of exceedances to zero (i.e. equivalent to forested conditions). Commensurate with the amount of impervious surfaces captured, the two scenarios utilizing rain gardens for mitigation (LID40 and LID80) reduce dissolved copper concentrations exceedances by 63% and 87%, respectively.

Table 24 Simulated average annual exceedances for dissolved copper.

Parameter	Average # of Exceedances per year							
	LU2002	FUTURE	LID40	ECY08	LID40+	LID80	LVL2WET	CISTERN
Dissolved Copper	92	99	37	0.0	33	13	3.8	105

The number of exceedances per year mostly is comprised of hourly acute exceedances, not days. Thus, one event can be comprised of multiple acute exceedances but is not

differentiated in the results. As an example, an average annual number of exceedances for LID40 is 37, but that could result from 2 to 3 events during the year.

Water Temperature

As previously reported from the monitoring conducted during this study, water temperatures in the various stream segments were above defined criteria for one or more months during the year. However, air temperatures during that summer (2009) were above normal, with record breaking days observed at Sea-Tac and other locations in the state which resulted in approximately 87 exceedances for that year (approximately 25% of the time). The simulated frequency of long-term temperature exceedances for current (LU2002) and future unmitigated conditions (FUTURE) is lower and estimated to occur approximately 17% of the time assuming current riparian shading conditions (Table 25).

Mitigation scenarios do not include any changes to riparian shading. Any effects seen can be attributed to either increasing base flows, and/or heating of large open bodies of water in ponds. The CISTERN scenario assumes no direct solar radiation reaches water surfaces but allows for heat transfer due to ambient air temperature. The mitigation scenarios with larger amounts of infiltration (ECY08 and LID80) reduce frequency of exceedances by more than 50 percent from future conditions. For the LVL2WET scenario, stream temperatures increase as a result of stagnate, large open bodies of water comprising the wetponds.

The intent of the modeling was to evaluate the effect of the mitigation scenarios without any changes to riparian buffers. Assuming an effective shade of 90 percent would most likely mitigate all exceedances for a stream system of this type (i.e. restoration of base flow, steeper gradient, faster travel times and lower potential of thermal inputs), but was not verified in this study.

Table 25 Simulated average annual exceedances for water temperature, dissolved oxygen, and fecal coliforms.

Parameter	Average # of Exceedances per year							
	LU2002	FUTURE	LID40	ECY08	LID40+	LID80	LVL2WET	CISTERN
Water Temperature[1]	63	69	44	29	28	20	74	30
Dissolved Oxygen	2.2	12	0.9	0.1	0.1	0.0	95	6.0
Fecal Coliforms	327	336	289	167	276	196	259	324

[1]Riparian shading kept constant among scenarios

Dissolved Oxygen

Dissolved oxygen will be near saturation and driven by air temperature for this type of stream system. However, mitigation scenarios with significant residence time will be additionally influenced by algae growth and respiration, organic decay, etc. In the simulated wetponds, 1-day minimum dissolved oxygen concentrations would routinely drop to 2 mg/L during summer months—affecting most, the LVL2WET scenario which features large wet ponds. The CISTERN scenario essentially had no algae growth (no light penetration), which would also suppress oxygen levels, but not nearly as much as stagnant wet ponds.

Fecal coliforms

As defined in section 2.6 describing removal efficiency, removals of fecal coliforms in the model are based solely the on the decay rate defined during calibration for the stream system and applied to the flow control facilities. In addition, large open bodies of water likely will be an attraction for wildlife and become fecal sources which are undifferentiated between human and animal in this study. Simulated unit area loading rates from forested conditions are lower than literature values (Figure 14) but still exceed defined thresholds (Figure 15).

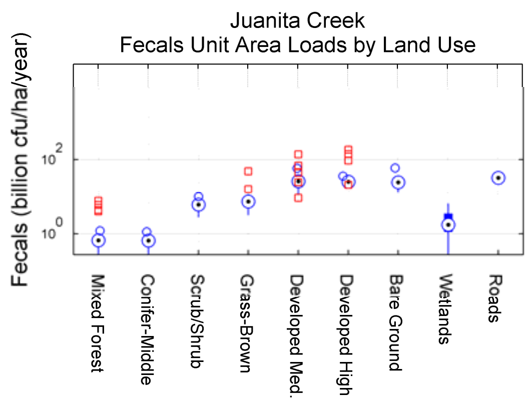

Figure 14 Unit area loading rates by land use for fecal coliforms. Blue symbols are simulated, red are from literature and previous studies.

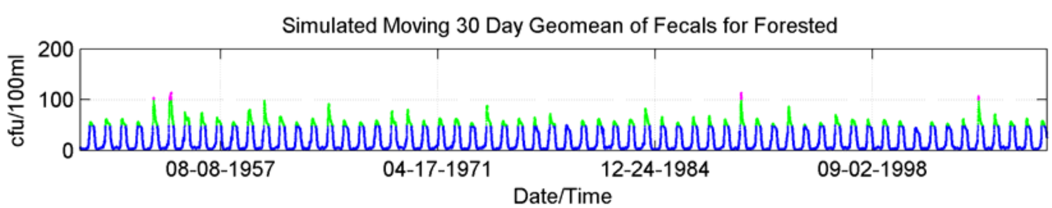

Figure 15 Simulated moving 30-day geometric means of fecal coliforms for forested (FOREST) conditions.

All mitigation scenarios produced exceedances of fecal coliform standards (Table 26). The ECY08 scenario showed the greatest reduction in the number of exceedances when compared to future unmitigated conditions (Future) with the LID80 scenario nearly as effective. The remaining scenarios reduced exceedances from 5 to 23 percent depending on the amount of flow control applied.

Table 26 Simulated average annual exceedances for fecal coliforms.

Parameter	Average # of Exceedances per year							
	LU2002	FUTURE	LID40	ECY08	LID40+	LID80	LVL2WET	CISTERN
Fecal Coliforms	327	336	289	167	276	196	259	324

The following three tables summarize how well the various mitigation scenarios meet the three benchmarks of forested (Table 27), 65/10 (Table 28), and LU1977 (Table 29). Cells filled with “Yes” are equal to or below the target scenario exceedances, and highlighted in green. Cells filled with “No” signify a scenario that does not achieve exceedance levels, and

are highlighted in red. Cells filled with "No*" do not meet target levels but are reduced from projected future conditions, and are also highlighted in green. The more relaxed the standard, the more targets are achieved with the exception of dissolved oxygen targets for 65/10. As the number of exceedances increase among the benchmarks, the relative comparison of the mitigation scenarios become more favorable. With the exception of LVL2WET and CISTERN, all mitigation scenarios had less exceedances then compared to LU1977 benchmark conditions.

Table 27 Simulated mitigation scenarios meeting forested conditions targets for dissolved copper, water temperature, dissolved oxygen, and fecal coliforms.

Parameter	Meet Forested Conditions Benchmark							
	LU2002	FUTURE	LID40	ECY08	LID40+	LID80	LVL2WET	CISTERN
Dissolved Copper	No	No	No*	Yes	No*	No*	No*	No
Water Temperature	No	No	No	Yes	Yes	Yes	No	Yes
Dissolved Oxygen	No	No	No	No	No	Yes	No	No
Fecal Coliforms	No	No	No	No	No	No	No	No

*Annual exceedances are significantly reduced from projected future conditions.

Table 28 Simulated mitigation scenarios meeting 65/10 conditions targets for dissolved copper, water temperature, dissolved oxygen, and fecal coliforms.

Parameter	Meets 65/10 Conditions Benchmark							
	LU2002	FUTURE	LID40	ECY08	LID40+	LID80	LVL2WET	CISTERN
Dissolved Copper	No	No	No*	Yes	No*	No*	No*	No
Water Temperature	No	No	No	Yes	Yes	Yes	No	Yes
Dissolved Oxygen	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes
Fecal Coliforms	No	No	No	No	No	No	No	No

*Annual exceedances are significantly reduced from projected future conditions.

Table 29 Simulated mitigation scenarios meeting 1977 conditions targets for dissolved copper, water temperature, dissolved oxygen, and fecal coliforms.

Parameter	Meet 1977 Conditions Benchmark							
	LU2002	FUTURE	LID40	ECY08	LID40+	LID80	LVL2WET	CISTERN
Dissolved Copper	No	No	No*	Yes	No*	Yes	Yes	No
Water Temperature	No	No	Yes	Yes	Yes	Yes	No	Yes
Dissolved Oxygen	No	No	Yes	Yes	Yes	Yes	No	No
Fecal Coliforms	Yes	No	Yes	Yes	Yes	Yes	Yes	Yes

*Annual exceedances are significantly reduced from projected future conditions.

3.7.2.2 Annual Loadings

In absence of mitigation, the projected largest increase in loadings is a near doubling of solids (i.e. TSS) from existing (LU2002) to future unmitigated (FUTURE) conditions. All other parameter loading rates remain similar between current and future conditions. The most effective mitigation scenarios for solids and metals were the LID80 scenario with rain gardens capturing 80 percent impervious surfaces and the ECY08 scenario which is a combination of rain gardens and dry/wet ponds (Table 30). However, the scenarios that include wet ponds (ECY08 and LVL2WET) increase ammonia loads as a result of simulated organic matter decaying into inorganic nitrogen and phosphorus species and the nitrate reduction via denitrification processes with low oxygen levels. Removal efficiencies are assumed zero for cisterns because of the absence of any algae growth or organic sediment. Any simulated reductions are a consequence of fate and transport processes simulated in the stream system occurring during the summer month releases.

Table 30 Simulated average annual loads.

Scenario		Solids[1]	Metals[2]		Nitrogen[2]			Phosphorous[2]	
		TSS	DCu	TCu	NH_4	NN	TN	PO_4	TP
Targets	FORESTED	0.1	0.1	0.1	5.0	35.4	100.8	1.9	7.7
	65/10	0.6	0.3	0.4	4.8	45.3	129.2	3.6	11.2
	LU1977	4.4	0.7	1.3	7.5	280.5	386.0	8.4	17.8
Unmitigated	LU2002	9.8	1.3	2.6	8.3	155.4	274.5	10.2	21.2
	FUTURE	17.8	1.4	3.3	8.4	138.0	258.2	11.0	22.4
Mitigations	LID40	5.1	0.9	1.7	7.1	116.7	203.0	7.5	15.5
	ECY08	0.7	0.5	0.5	11.2	78.3	129.3	5.9	9.2
	LID40+	5.2	0.9	1.7	7.4	116.7	203.9	7.5	15.5
	LID80	1.5	0.5	0.8	6.7	108.8	185.7	4.6	11.1
	LVL2WET	3.9	1.3	2.1	9.8	123.4	194.5	12.6	18.8
	CISTERN	10.7	1.5	3.0	7.1	134.7	246.3	11.0	21.5

[1]Loads are in metric tons per year (mt/yr)

[2]Loads are in kilograms per year (kg/yr)

<u>Annual Loading Targets</u>

None of the mitigation strategies reduce annual loadings to those modeled under hypothetical forested or 65/10 benchmarks, with the exception of total phosphorus which is reduced to below 65/10 levels applying the ECY08 and LID80 scenarios. Estimated annual loadings are less than those estimated under the LU1977 benchmark scenario for at least two parameters for every mitigation scenario (Table 31). The LID80 scenario had lower loadings for all parameters. It is worth noting that the simulated nitrate and total nitrogen loading rates for LU1977 are significantly higher than all other scenarios. This is a result of large portions of the basin assumed to have active pasture lands (based on photo interpretation done generating the 1977 land use scenario) with sources higher in nitrogen.

Table 31 Simulated annual loads meeting benchmarks.

Target Scenario	Scenario	Solids	Metals		Nitrogen			Phosphorous	
		TSS	DCu	TCu	NH_4	NN	TN	PO_4	TP
Forested	LU2002	No	No	No	No	No	No	No	No
	FUTURE	No	No	No	No	No	No	No	No
	LID40	No	No	No	No	No	No	No*	No
	ECY08	No*	No*	No*	No	No	No	No*	No*
	LID40+	No	No	No	No	No	No	No*	No
	LID80	No*	No*	No*	No	No	No	No*	No*
	LVL2Wet	No*	No	No	No	No	No	No	No
	CISTERNS	No	No	No	No	No	No	No	No
65/10	LU2002	No	No	No	No	No	No	No	No
	FUTURE	No	No	No	No	No	No	No	No
	LID40	No	No	No	No	No	No	No*	No
	ECY08	No*	No*	No*	No	No	No	No*	Yes
	LID40+	No	No	No	No	No	No	No*	No
	LID80	No*	No*	No*	No	No	No	No*	Yes
	LVL2Wet	No*	No	No	No	No	No	No	No
	CISTERNS	No	No	No	No	No	No	No	No
LU1977	LU2002	No	No	No	No	Yes	Yes	No	No
	FUTURE	No	No	No	No	Yes	Yes	No	No
	LID40	No	No	No	Yes	Yes	Yes	Yes	Yes
	ECY08	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes
	LID40+	No	No	No	Yes	Yes	Yes	Yes	Yes
	LID80	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
	LVL2Wet	Yes	No	No	No	Yes	Yes	No	No
	CISTERNS	No	No	No	Yes	Yes	Yes	No	No

*Does not meet targets, but annual loads are significantly reduced.

3.7.3 Simulated Gravel Disturbances and LWD Densities

Using the methods described in section 2.7.4, average annual gravel disturbances were determined for mitigation and benchmark scenarios for three selected stream reaches (Sites 107, 105, and 203). Actual flow rates competent to mobilize gravels at each of the three sites varied to a minor degree around a typical value of 30 cfs. The number of disturbances, with an assumed D_{50} equal to 16.5 mm, range from near zero in the headwater reach to an average disturbance greater than 30 times per year near the mouth for half of the scenarios (LID40, LID40+, and CISTERN). Several scenarios for the mid-basin and upper tributary site produced conditions closer to target ranges, while the most aggressive mitigation scenarios over-shot the target causing disturbances less than 1 per

year. Except for near the mouth (Site 107), the fully forested (FOREST) scenario does not produce high enough flow rates to mobilize gravels using existing channel geometries with ideal gravel sizes. In addition, the 65/10 scenario will not move gravels in the upper site (site 203), moves gravels only marginally in the mid-basin site (site 105), and over-produces disturbances in the lower site (site 107) near the mouth. Average annual gravel disturbances are summarized in Table 32, Table 33, and Table 34 below, with green cells highlighting where the target range of 1 to 3 per year is met. Also included in the tables are flow rates associated with average frequencies of 1 per year (Upper Q) and 3 per year (Lower Q), respectively.

Scenarios that project the number of disturbances greater than three per year are further evaluated by augmenting stream channel roughness with large wood (LWD). Using estimated equivalencies of average channel roughness (Manning's n) to linear densities of placement for LWD, a channel roughness capable of reducing the number of disturbances back to the target range of 1 to 3 per year is estimated. The greater the number of disturbances per year, the more pieces of LWD per 1000 feet is necessary. Existing channel roughness at the three sites is estimated as n= 0.036, .035, and .035. The modified channel n-values required to achieve the target disturbance range and the density of LWD placements necessary to achieve those roughnesses are listed in Table 32, Table 33, and Table 34.

For comparative purposes only, gravel disturbances and LWD densities are included for unmitigated existing (LU2002) and future (FUTURE) conditions.

Table 32 Estimated gravel disturbances and LWD placement density for Site 107 (near the mouth). Yellow cells indicate the flow rate used for analysis.

Scenario	Site 107 Gravel Disturbance			Using modified Wilcock	
	Lower Q (cfs)	Upper Q (cfs)	# of Disturbances	LWD Density* (# / 1000 ft)	Modified channel-n (n_{exist} 0.036)
FORESTED	14.3	23.3	0.5	--	< 0.036
65/10	34.6	43.9	5.4	3	0.040
LU1977	103.9	144.9	30.1	9	0.045
LU2002	141.1	199.3	33.2	8	0.044
FUTURE	189.0	260.7	34.2	6	0.042
LID40	107.3	145.8	30.8	9	0.045
ECY08	40.0	51.7	6.3	3	0.040
LID40+	108.2	147.0	31.1	9	0.045
LID80	55.7	71.3	17.2	6	0.042
LVL2WET	43.1	53.1	8.8	3	0.040
CISTERN	136.2	197.2	33.0	9	0.045

*Represents density necessary to achieve 1 to 3 disturbances per year

Table 33 Estimated gravel disturbances and LWD placement density for Site 105 (mid-basin mainstem). Yellow cells indicate the flow rate used for analysis.

Scenario	Site 105 Gravel Disturbance				
			Using standard Shields		
	Lower Q (cfs)	Upper Q (cfs)	# of Disturbances	LWD Density* (# / 1000 ft)	Modified channel-n (n_{exist} 0.035)
Forest	5.7	12.6	0.0	--	< 0.035
65/10	20.5	26.9	0.1	--	< 0.035
LU1977	68.1	97.7	8.1	2	0.040
LU2002	88.8	127.4	13.8	7	0.044
FUTURE	123.5	167.3	19.7	21	0.050
LID40	65.2	88.4	7.3	2	0.040
ECY08	21.4	28.9	0.1	--	< 0.035
LID40+	66.5	89.6	7.8	5	0.042
LID80	34.0	43.6	0.6	--	< 0.035
LVL2WET	25.1	31.0	0.2	--	< 0.035
CISTERN	87.7	130.4	13.1	7	0.044

*Represents density necessary to achieve 1 to 3 disturbances per year

Table 34 Estimated gravel disturbances and LWD placement density for Site 203 (north branch headwater tributary). Yellow cells indicate the flow rate used for analysis, green cells indicate achieved targets.

Scenario	Site 203 Gravel Disturbance				
			Using standard Shields		
	Lower Q (cfs)	Upper Q (cfs)	# of Disturbances	LWD Density* (# / 1000 ft)	Modified channel-n (n_{exist} 0.035)
FORESTED	1.6	4.9	0.0	--	< 0.035
65/10	7.8	9.9	0.0	--	< 0.035
LU1977	29.5	46.6	2.9	0	0.035
LU2002	33.9	55.1	4.2	3	0.040
FUTURE	56.9	76.6	10.0	18	0.049
LID40	21.0	29.0	0.8	1	0.037
ECY08	9.8	12.6	0.0	--	< 0.035
LID40+	21.8	29.9	1.0	0	0.035
LID80	12.6	16.5	0.1	--	< 0.035
LVL2WET	9.8	12.4	0.1	--	< 0.035
CISTERN	36.8	61.0	5.0	3	0.040

*Represents density necessary to achieve 1 to 3 disturbances per year

3.7.4 Mitigation Costs

As detailed in the methods section 2.8, this cost analysis includes all significant costs required to implement each mitigation scenario, including land purchases using prices specific to the Juanita Creek basin, construction costs, and operation and maintenance costs accrued over an assumed 40 year life cycle. Methods used to size the dry ponds in the regulatory ECY08 scenario were not fully optimized to reduce footprints and volumes of the facilities.

The estimated total net present (2011) value for the seven scenarios (Table 35 and Figure 16) evaluated ranged from $200 million to $1.4 billion dollars, which equates to $30 to $200 million per square mile or $5 to $35 million per year (over a 40 year time span).

Two scenarios, ECY08 and LID80, far surpass the others in terms of cost. In both cases, the primary cost driver is the installation and maintenance of many small rain gardens across the landscape necessary to achieve the required performance. In particular, operations and maintenance costs attributed to private homeowners for rain gardens comprise approximately 50% of the total costs for these two scenarios. Both scenarios utilize rain gardens to treat the majority (80%) of impervious surfaces in the basin (ECY08 utilizes conventional ponds as well).

The LID40 and LID40+ scenarios have nearly identical costs-- again mostly driven by the rain gardens, which for these scenarios treat 40% of the total basin impervious surfaces.

A common theme among the least expensive scenarios was low operations and maintenance costs. These include LEVEL2, LVL2WET, and CISTERN scenarios.

Table 35 Net present value costs per scenario, based on 40 year lifespan.

		O & M[3]		
Scenario	Capital[3]	Private[1]	Public[2]	Total[3]
LEVEL2	$207.52	$0.00	$2.98	$210.50
LID40	$184.57	$339.74	$65.76	$590.06
ECY08	$559.00	$679.48	$134.45	$1,372.93
LID40+	$185.32	$339.74	$65.95	$591.01
LID80	$369.13	$679.48	$131.51	$1,180.12
LVL2WET	$210.92	$0.00	$3.87	$214.79
CISTERN	$177.71	$0.00	$79.46	$257.17

[1]Private costs are burdens on the private citizens
[2]Public costs are burdens on the government jurisdictions
[3]Estimated costs are in millions of dollars

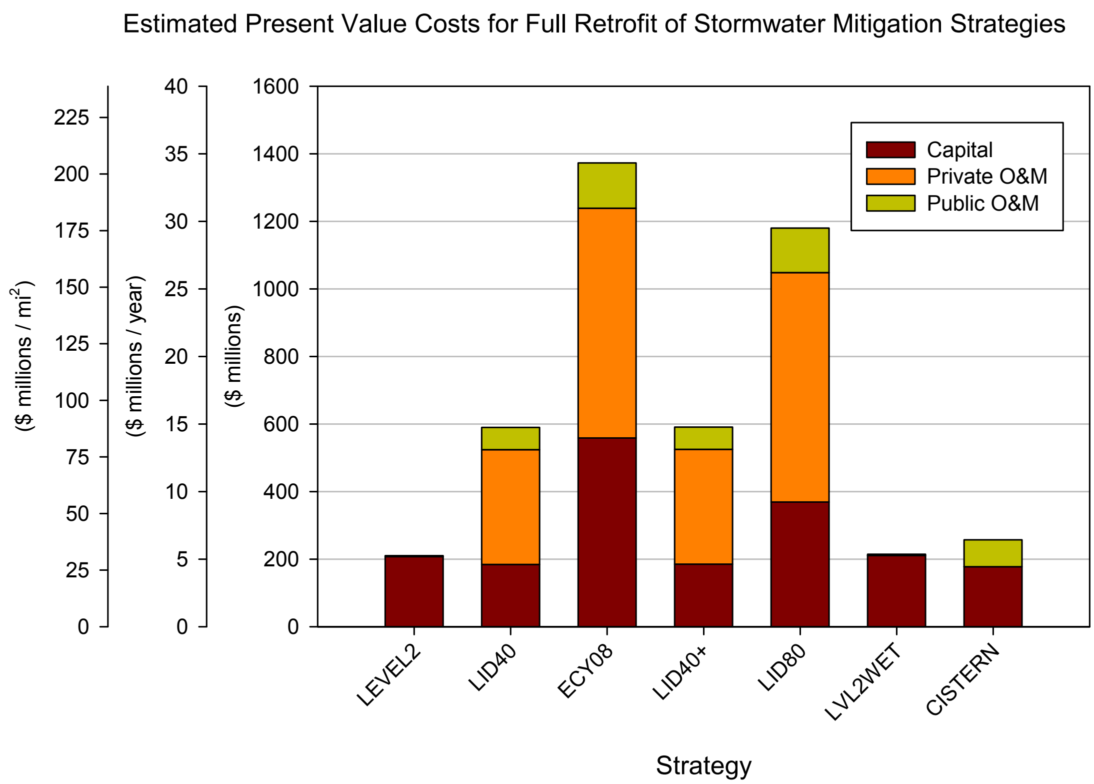

Figure 16 Estimated present value costs for full retrofit mitigation strategies.

3.7.5 Mitigation Cost Effectiveness

Of the seven mitigation scenarios evaluated, the two most expensive scenarios, LID80 and ECY08, routinely stand out as the most generally effective. The remaining mitigation scenarios had lower overall levels of effectiveness. Interestingly, the LVL2WET scenario appears to be a 'best bang for the buck' scenario even though its overall level effectiveness is deemed poor.

Cost effectiveness is summarized by metric in Table 36 below. Further detail is provided in the following Figure 17 through Figure 31.

Table 36 Summary of cost effectiveness responses for the mitigation scenarios.

Metric	Effectiveness	Figure
BIBI	ECY08 and LID80 best mitigate altered hydrology degrading projected biotic responses, but only ECY08 projects a BIBI score of 35- identified as one element necessary for achieving restoration of full beneficial uses.	Figure 17
Dissolved Copper	ECY08 and LID80 are most effective, but LVL2Wet highly effective and considerably less expensive.	Figure 18

Metric	Effectiveness	Figure
DO	All scenarios are similarly effective with the exception of wet ponds that experience low oxygen levels during warmer months.	Figure 19
Temperature	All scenarios are similarly effective with the exception of wet ponds that act as shallow lakes more sensitive to air temperatures and solar radiation. None of the mitigation strategies evaluated riparian buffer shading.	Figure 20
Fecal	ECY08 and LID80 are most effective and most expensive. Reducing fecal concentrations are a result of flow controls via infiltration pathways.	Figure 21
TSS	Similar to dissolved copper, ECY08 and LID80 are most effective, but LVL2Wet highly effective and considerably less expensive.	Figure 22
Total Copper	ECY08 and LID80 most effective and most costly	Figure 23
Nitrogen	ECY08 shows to be the most effective for nitrates and total nitrogen, but true responses are more unknown given the variable dynamics in nutrient uptake for the wet ponds.	Figure 24 Figure 25 Figure 26
Phosphorus	ECY08 and LID80 are most effective, but like nitrogen, true responses are more unknown given the variable dynamics in nutrient uptake for the wet ponds.	Figure 27 Figure 28
Gravel Disturbances	Any scenario not effective is mostly due to over-mitigation by reducing stream energies below levels capable of mobilizing gravels. This is more prevalent in headwater study reach (Site 203).	Figure 29 Figure 30 Figure 31

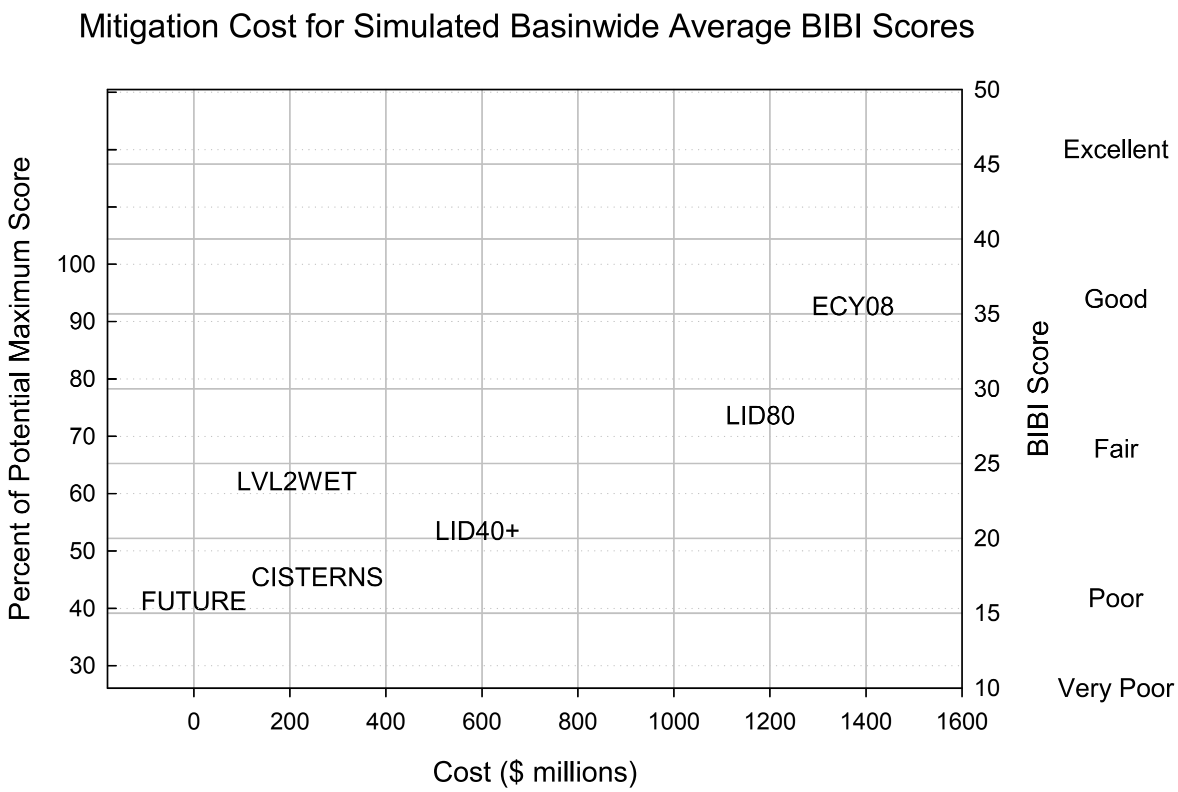

Figure 17 Cost effectiveness for basinwide average BIBI score. (LEVEL2 ≈ LVL2WET, LID40 ≈ LID40+)

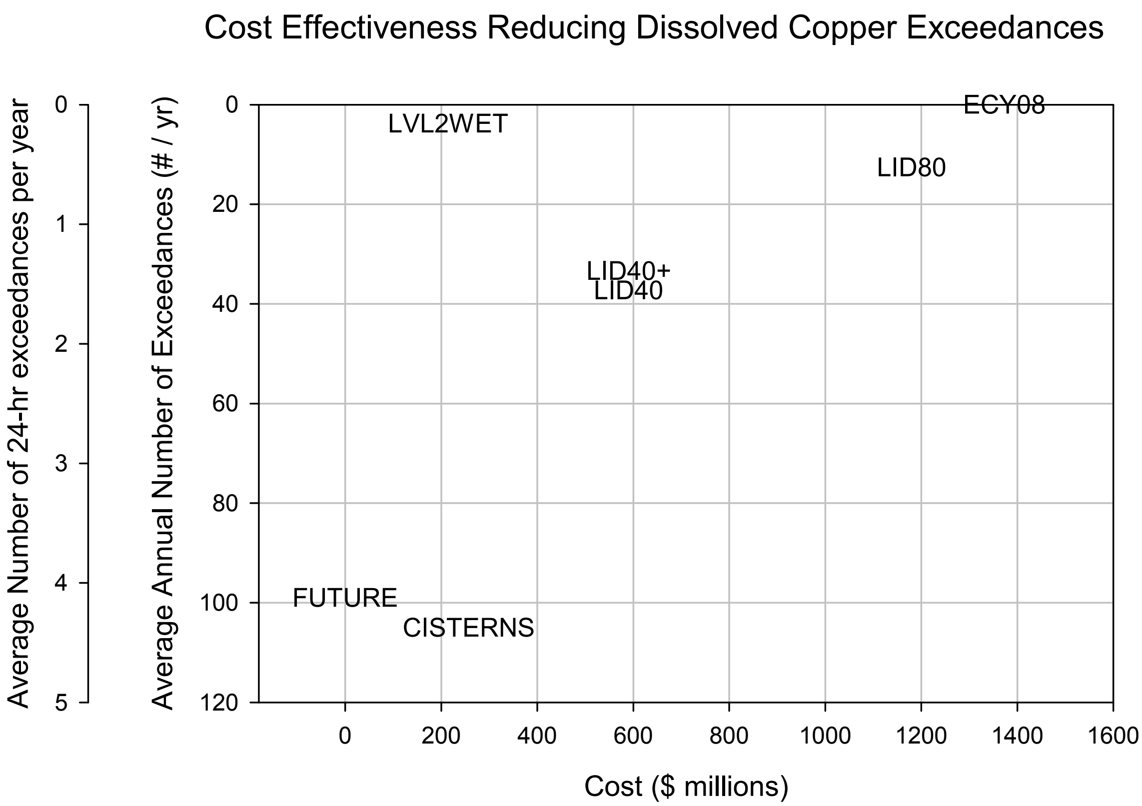

Figure 18 Cost effectiveness for reducing dissolved copper exceedances (LEVEL2 not assessed for removal efficiencies).

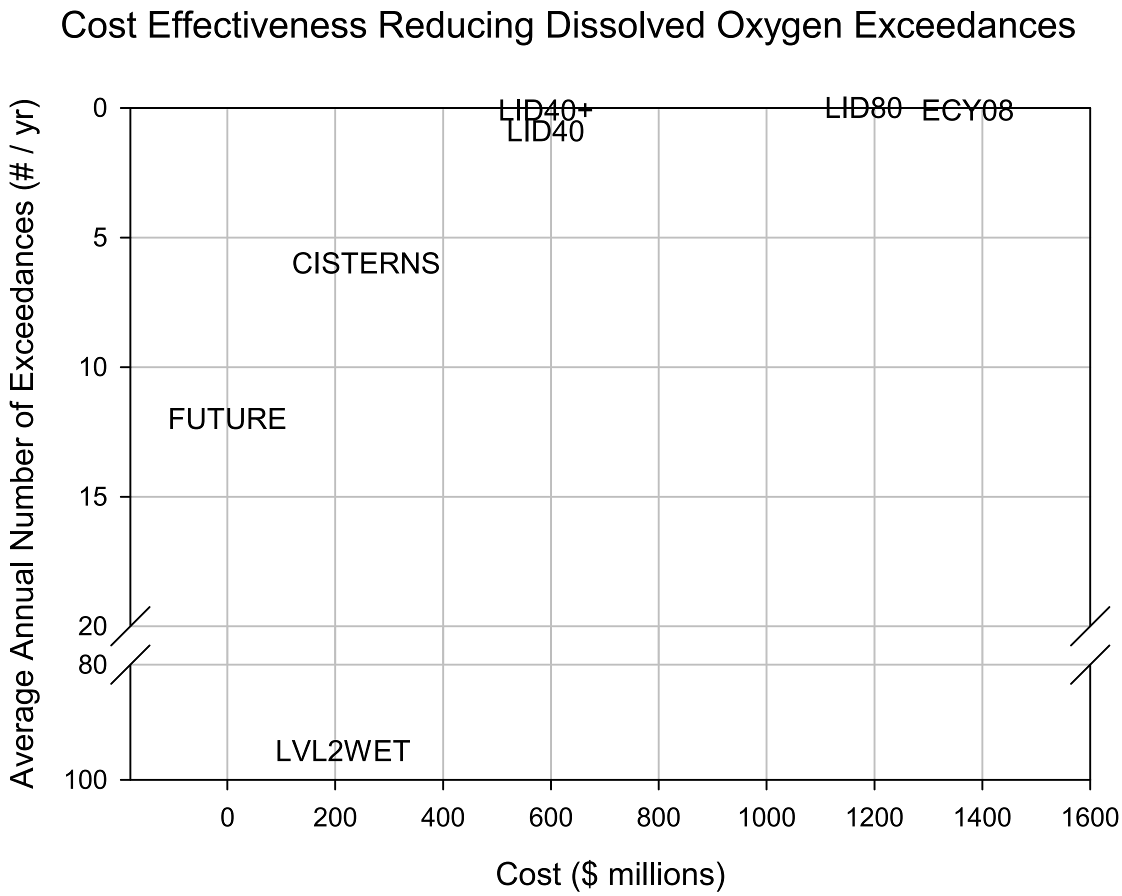

Figure 19 Cost effectiveness for reducing dissolved oxygen exceedances.

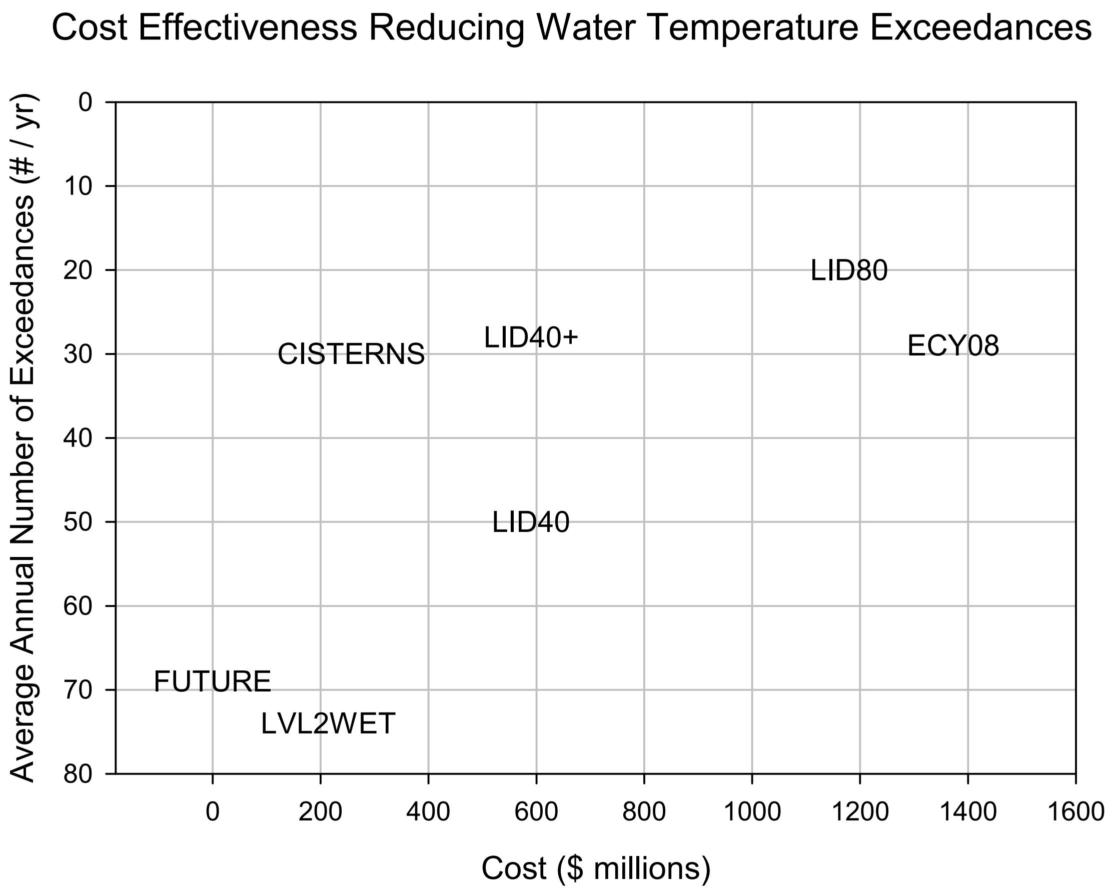

Figure 20 Cost effectiveness for reducing water temperature exceedances

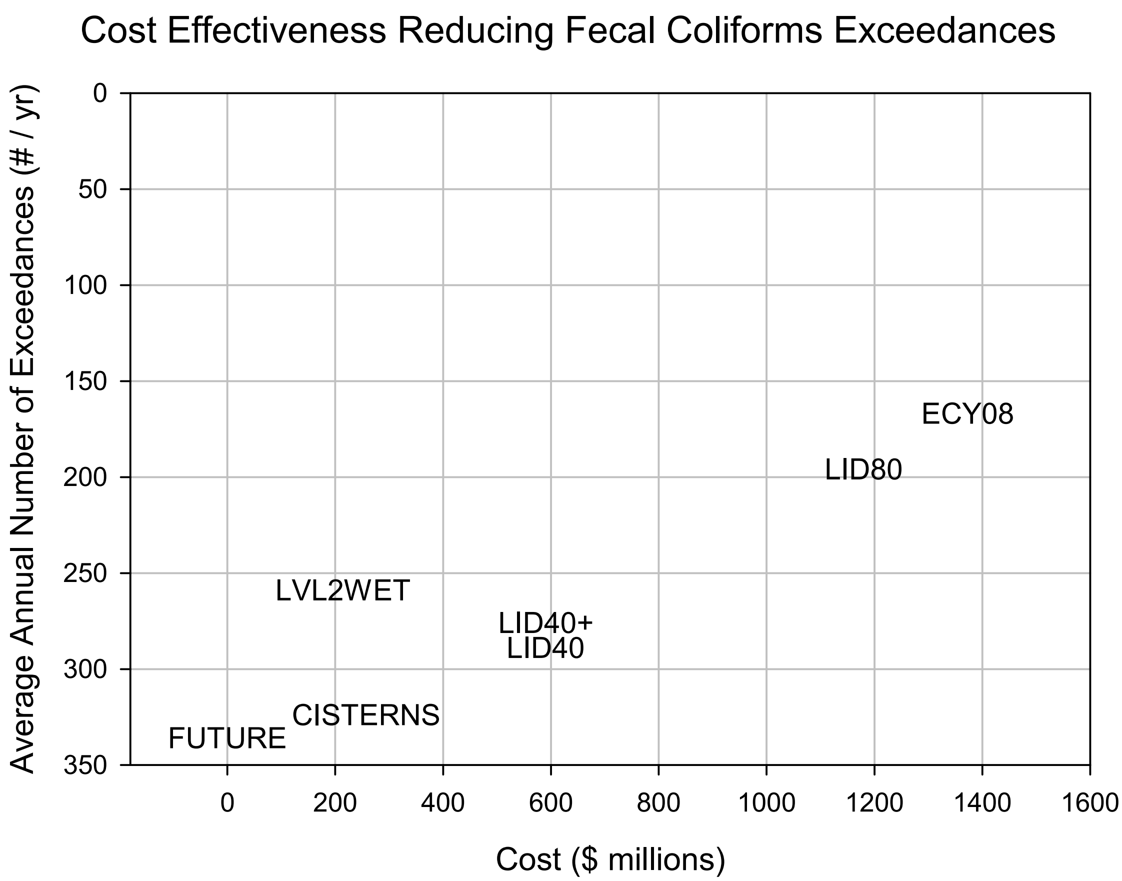

Figure 21 Cost effectiveness for reducing fecal coliform exceedances.

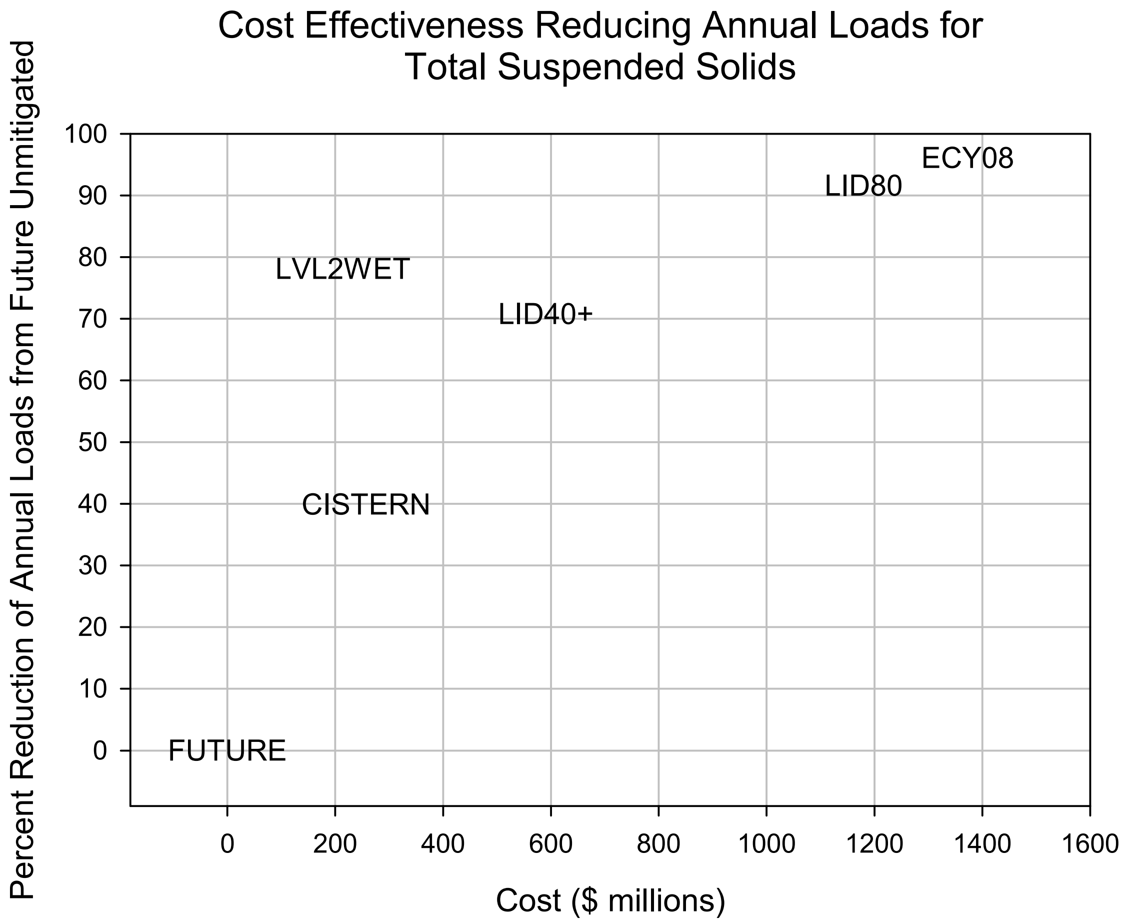

Figure 22 Cost effectiveness for reducing annual loads of TSS (LID40 ≈ LID40+).

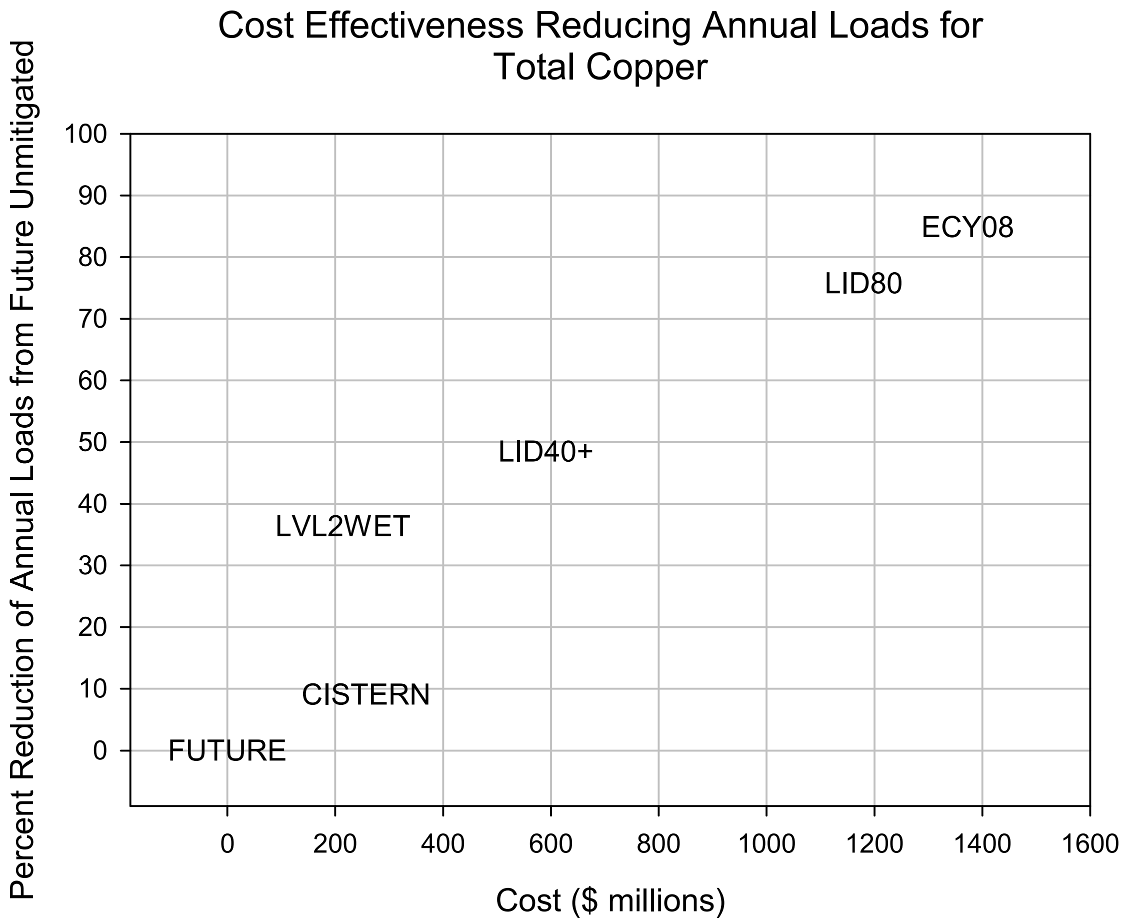

Figure 23 Cost effectiveness of reducing annual loads for total copper (LID40 ≈ LID40+).

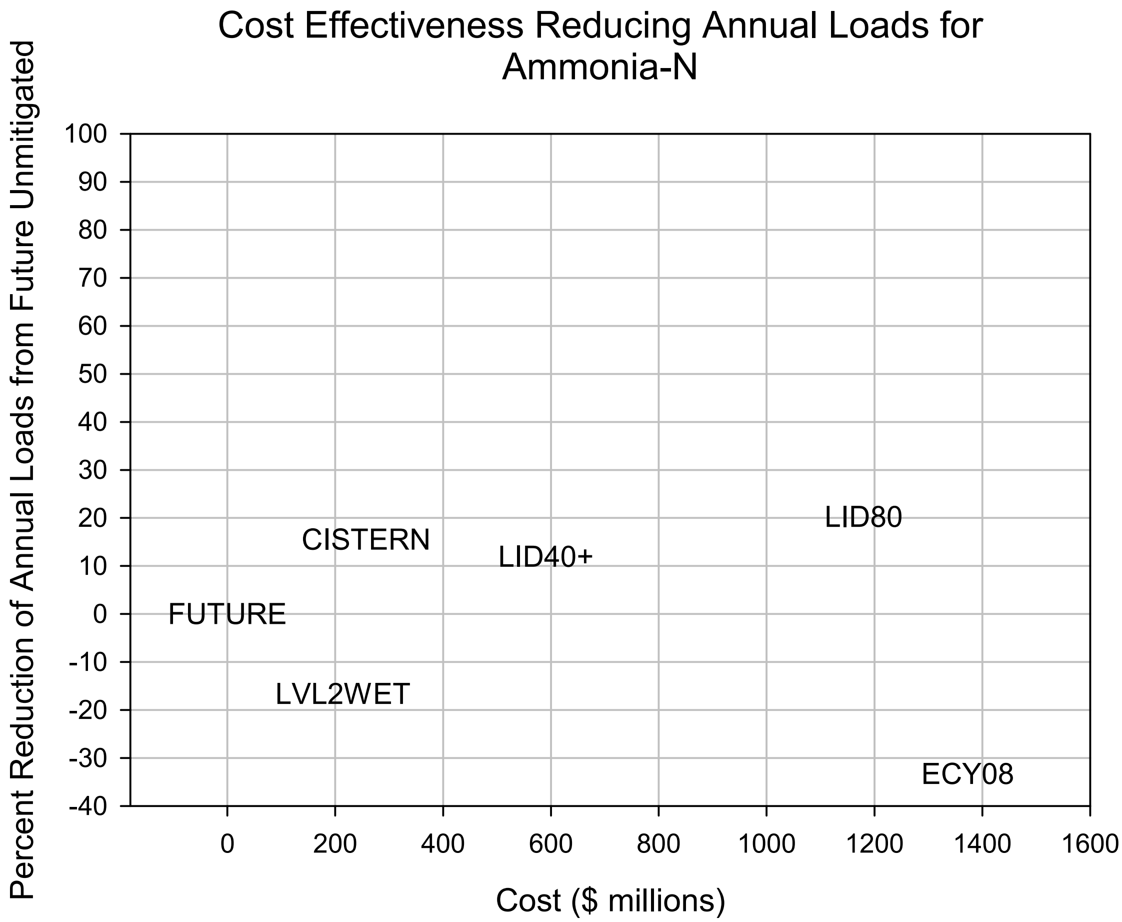

Figure 24 Cost effectiveness of reducing annual loads for ammonia-n (LID40 ≈ LID40+).

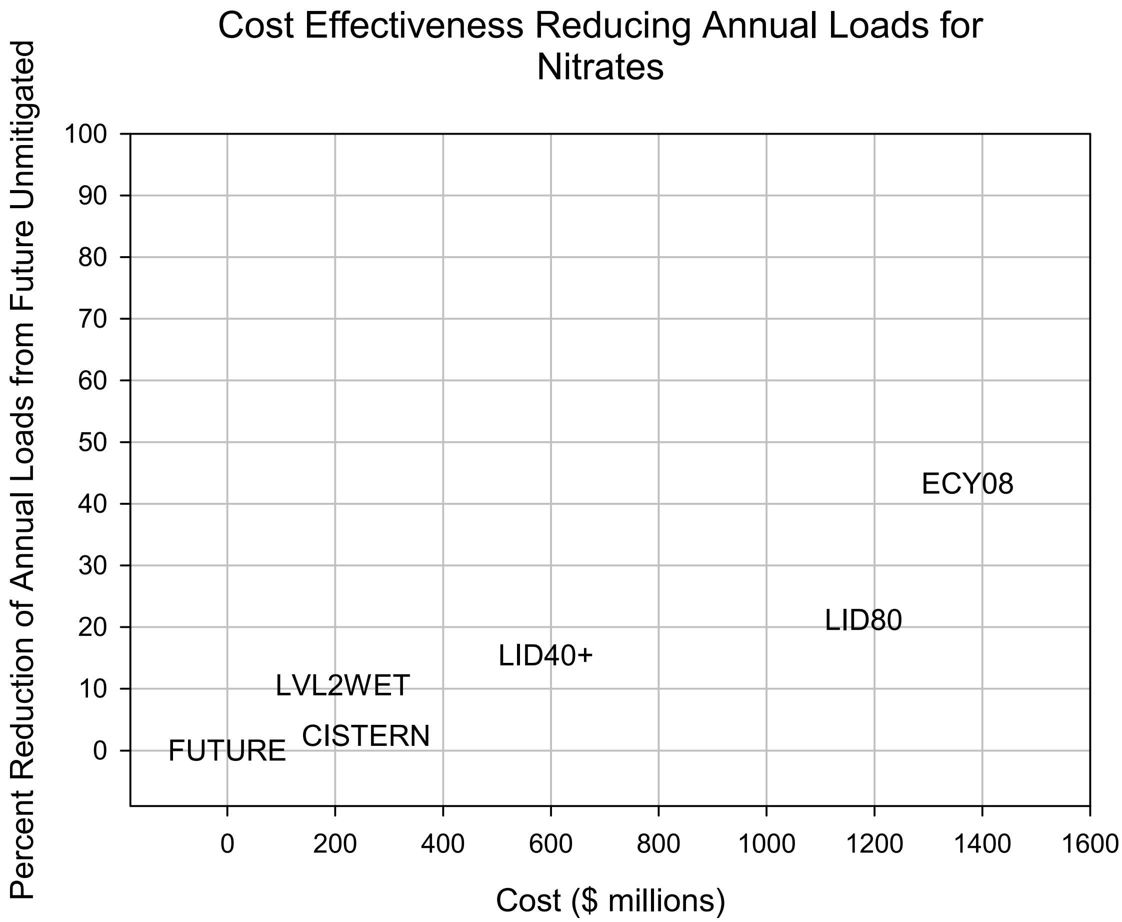

Figure 25 Cost effectiveness of reducing annual loads for nitrates (LID40 ≈ LID40+).

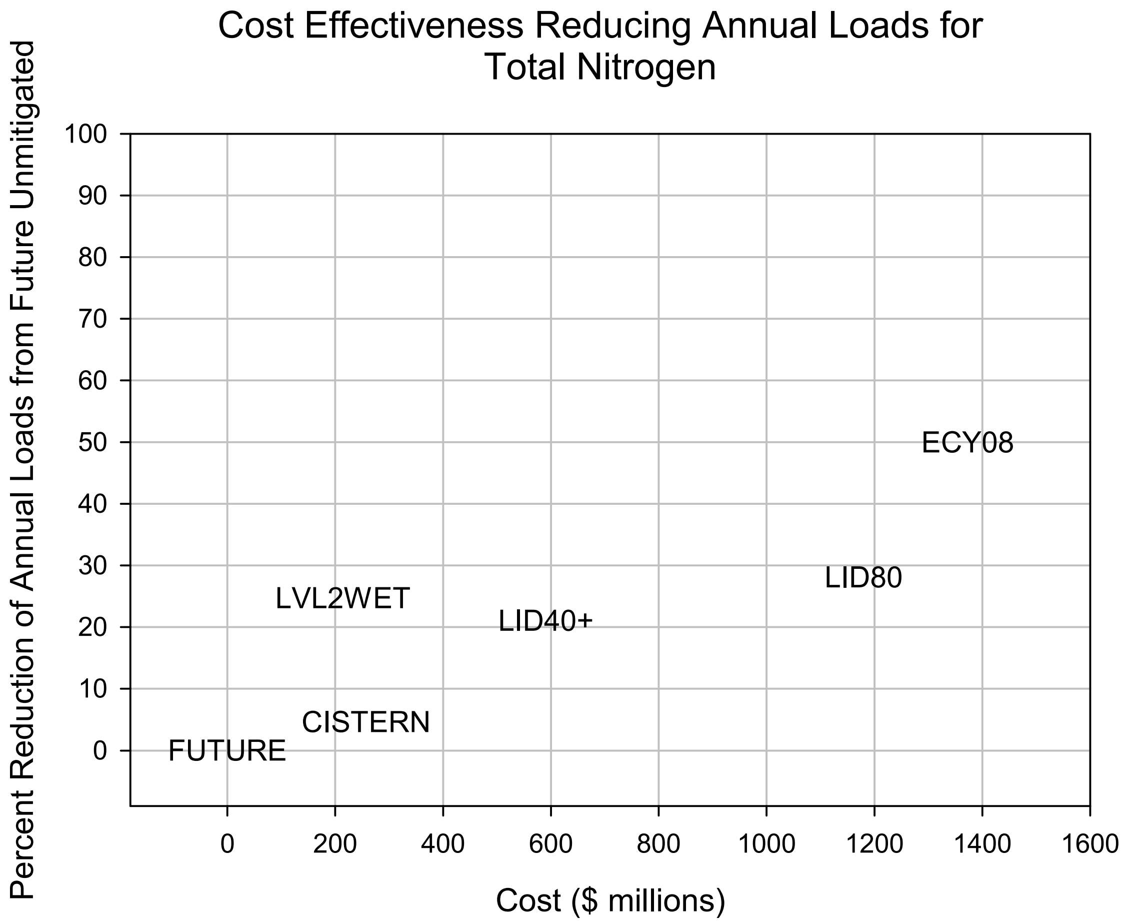

Figure 26 Cost effectiveness of reducing annual loads for total nitrogen (LID40 ≈ LID40+).

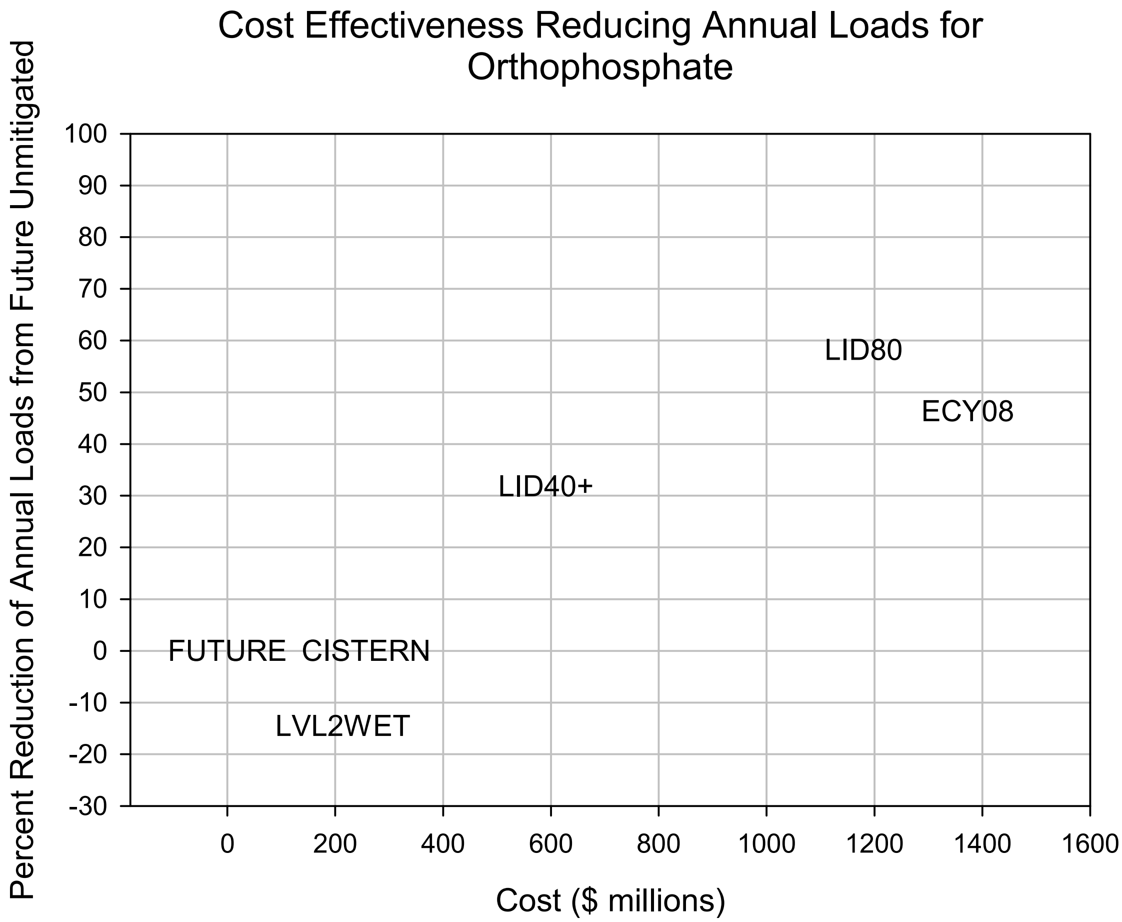

Figure 27 Cost effectiveness of reducing annual loads for orthophosphate (LID40 ≈ LID40+).

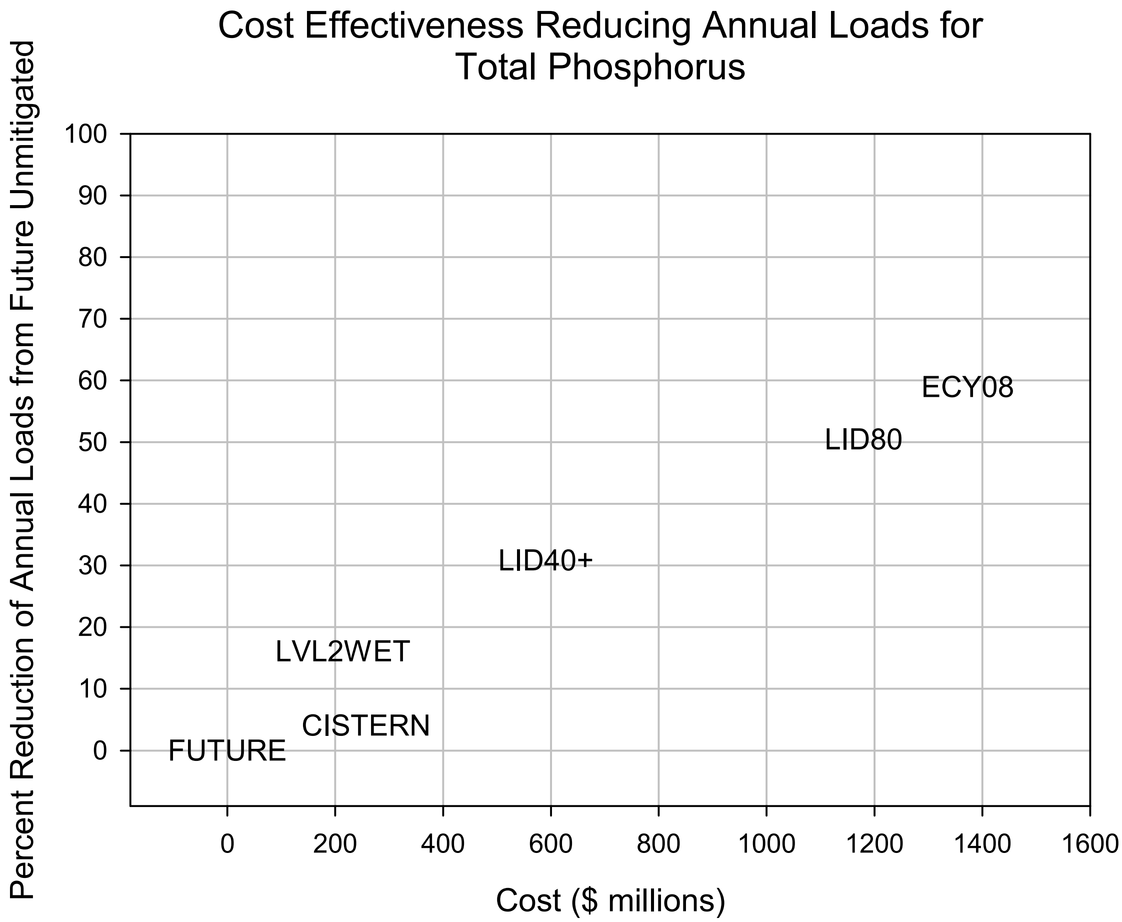

Figure 28 Cost effectiveness of reducing annual loads for total phosphorus (LID40 ≈ LID40+).

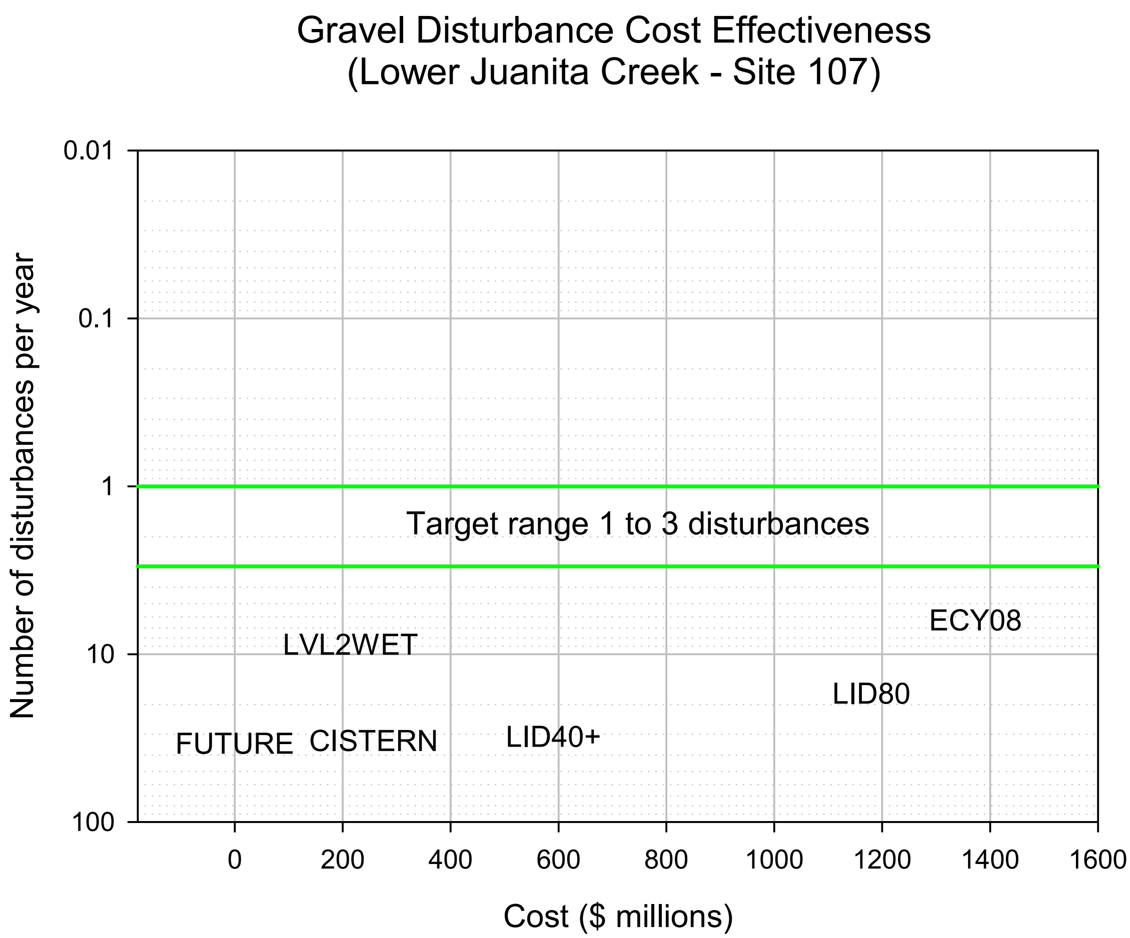

Figure 29 Cost effectiveness for gravel disturbances for lower Juanita, site 107, LID40 ≈ LID40+.

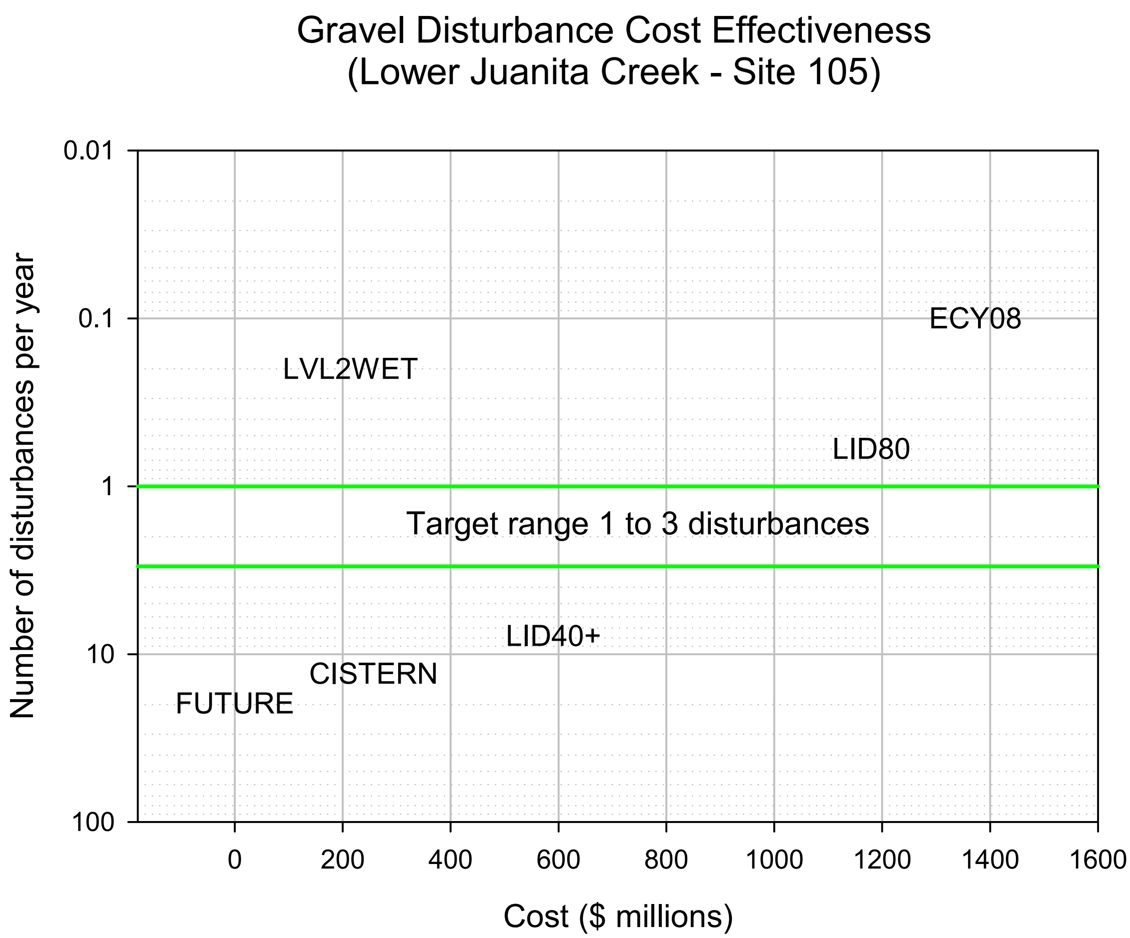

Figure 30 Cost effectiveness for gravel disturbances for middle Juanita, site 105, LID40 ≈ LID40+.

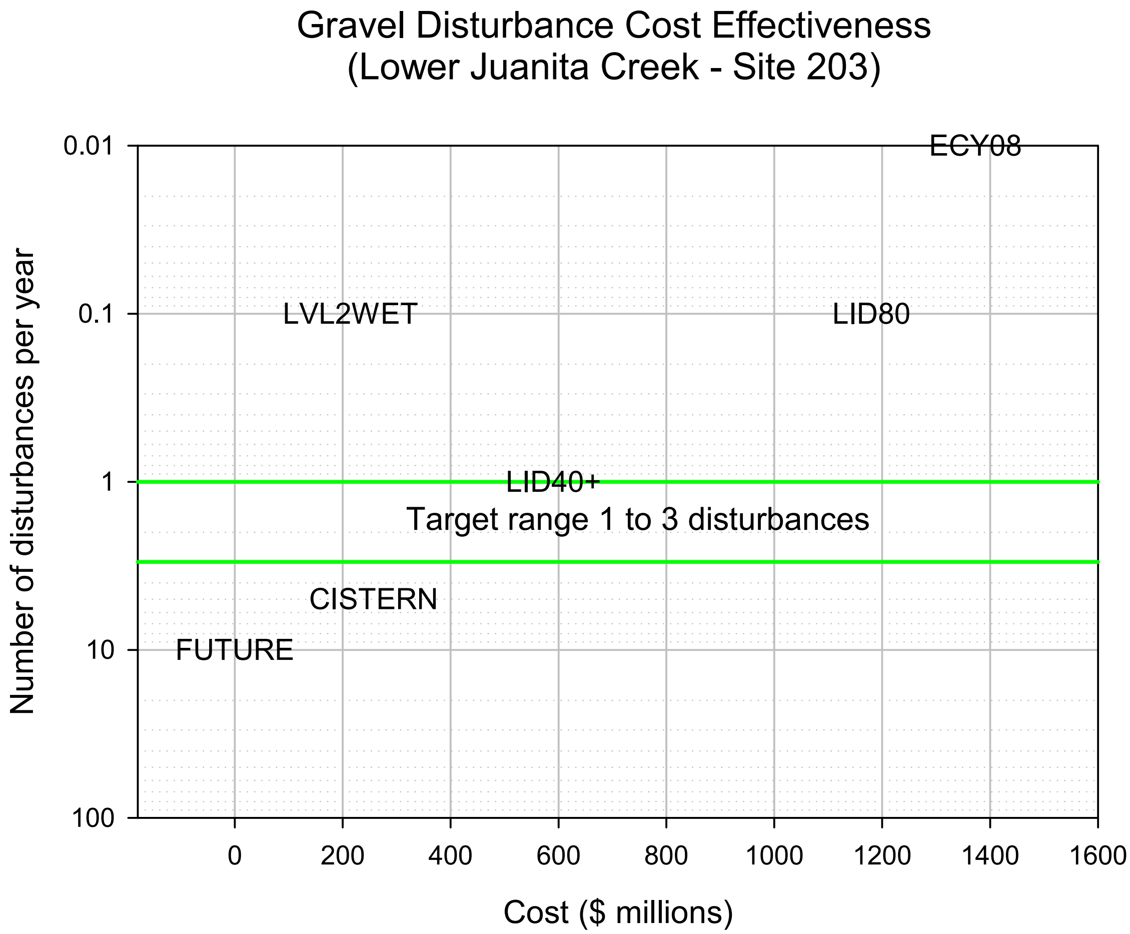

Figure 31 Cost effectiveness for gravel disturbances for upper Juanita, site 203, ECY08 had zero disturbances, set to 0.01 for display purposes, and LID40 was similar to LID40+ with 0.8 disturbances per year.

4.0. DISCUSSION

Hydrologic conditions are the key driver determining and setting the foundation for the health of stream systems (e.g., Poff et al. 1997, Richter 1996, Instream Flow Council 2002). Modified hydrology that results from urbanization can be seen in the exacerbation of the flashiness of stream flows---with corresponding deleterious effects on overall stream health. These effects can be observed (and measured) in lower BIBI scores, degraded water chemistry, and increases in frequency of stream bed mobilization.

In the Juanita Creek stream system, this study found that a basin-wide retrofit to the current stormwater standards required by King County and Ecology (LVL2WET) fails to achieve flow and water quality conditions supportive of beneficial uses. In fact, of seven mitigation scenarios evaluated, we feel confident in asserting that only one, ECY08, meets the stormwater restoration goal. Along with significant improvements made to water quality, the ECY08 scenario was unique among the scenarios evaluated in exceeding the targeted BIBI score of 35—a suggested lower threshold for salmon population viability. In addition, ECY08's predicted BIBI (36) was nearly equal to that generated under fully forested conditions (38)—achieving 93% of this potential maximum score.

It is possible that the predicted BIBI scores for ECY08 (and other scenarios) are conservative (under- predicting BIBI) when other limiting factors are addressed as part of the mitigation or through other restoration efforts. This is suggested by the observation that the original paired BIBI/hydrology data used for creating the predictive regressions likely included suppressing factors (e.g., degraded water quality, lack of riparian buffer, scarcity of large wood, bank instability, etc.) not accounted for in the regression relationships.

Infiltration of stormwater runoff was instrumental in meeting the ECY08 scenario performance standard. In concert with conventional pond facilities, the scenario employed low impact development BMPs (i.e., rain gardens) to treat and infiltrate stormwater runoff from the majority (80%) of the impervious surfaces in the Juanita Creek basin—which has a roughly even split of outwash and till underlying soil types. High infiltration rates assumed in the rain gardens in outwash soil areas was critical in meeting the desired performance. To achieve the same performance in basins dominated by low permeability soils, extensive use of soil amendments or greater reliance on conventional facilities (i.e., ponds) would be necessary.

In addition to being the best performing scenario, ECY08 was also the most expensive, mostly a result of the costs associated with the aforementioned LID BMPs. In particular, the costs for this scenario were driven by the operations and maintenance needs of thousands of individual rain gardens distributed across the landscape—the majority of which will likely be located on private properties. While the assumption that most of these costs will be shouldered by individual property owners may suggest this strategy as being more 'affordable' to public agencies, it comes with a corresponding caution regarding how much reliance should be placed on the performance of privately maintained small scale BMPs -- whose long term functionality, inspection, and maintenance requirements are not yet well understood.

Implementation strategies should evaluate whether publicly maintained regional infiltration facilities are practicable as these could provide the required infiltration performance at reduced costs, due to a presumptive economy of scale realized in construction and maintenance of fewer, larger facilities. This suggestion, however, includes the caveat that potential cost savings resulting from a regional facility approach will come at the price of allowing impacts from untreated stormwater to occur at all locations upstream of the regional facilities.

An interesting result of the gravel disturbance analysis was that highly mitigative scenarios including ECY08, LID80, and LVL2WET produced disturbances below the targeted range of 1 to 3 per year at two of the three sites studied. This suggests that a basin fully restored to the desired hydrologic conditions with amended gravel supportive of Coho salmonids will likely require channel modifications (e.g. benched channels with narrower inner channels where gravels can be mobilized by smaller events and upper benches to pass less frequent, larger peaks) to achieve gravel disturbances within the target range. The available flow energy per unit width under these mitigation scenarios is insufficient to mobilize median D_{50} gravels frequently enough in the existing widened, altered channels. This issue is further compounded by the likely necessary addition of large wood to the system, which presently is sorely lacking (NHC 2010). Any such addition would further reduce the energy in the system available to move gravels. It is possible that the stream channels would 'heal' and narrow over time as the hydrologic regime is gradually restored in response to the gradual implementation of mitigation projects.

Despite the fact that hydrologic mitigation is necessarily the foundation of any restoration effort, a comprehensive approach to stream restoration would include: implementing programs to recover buffer and restore stream shading as a strategy to address temperature issues in the stream system, water quality treatment of developed surfaces, bank stabilization projects to improve water quality and reduce fine sediments that impact salmonid spawning habitat, addition of large wood to achieve densities within the range typical for a natural stream system to improve habitat, and channel modifications in response to the mitigated hydrology.

If communities accept restoration of beneficial uses in streams in the urbanized Puget Sound region as an important goal, large-scale retrofitting efforts are critical in order to mitigate the hydrologic impacts caused by existing development. Updated stormwater regulations, like the ECY08 standard described in this study, applied to new and re-development projects may ultimately succeed in accomplishing this goal, but without aggressive retrofitting, this approach could take decades or longer.

This study advances a new paradigm with regard to the analytic methods used for evaluating stormwater impacts and measuring the effectiveness of mitigation strategies. The study provides an effective quantitative template for examining retrofit options in the Puget Sound region and beyond.

5.0. RECOMMENDATIONS

Based upon the results of the study, the following recommendation is provided to improve flow and water quality conditions to levels identified as critical for supporting aquatic beneficial uses (e.g. fish use) within the Juanita Creek basin stream system:

> Adopt a stormwater performance standard that, at minimum, matches that of the ECY08 mitigation scenario evaluated in this report. Achieving this standard may be accomplished through stormwater regulations applied to new and re-development projects as well as retrofitting existing development. Without an aggressive capital retrofit program, however, it could take many decades for a full retrofit to be accomplished given the slow rates of new and redevelopment occurring in the basin.

<u>Recommended Next Steps:</u>

1. Prepare a detailed plan that includes a time bound implementation strategy and performance targets; and that identifies specific capital projects. Use this report and appendices as a resource for the plan. Consider using a test sub-basin to study the efficacy of the proposed ECY08 mitigation scenario before implementing capital retrofit projects basin-wide. Use the results of the study to inform how to adapt a basin-wide mitigation strategy. Selection criteria for the test sub- basin should include: size (i.e., pick a drainage area that both supports a perennial stream where BIBI and flow can be measured while minimizing other potential limiting factors such as water temperature, dissolved oxygen, etc.); existing BIBI score (i.e. pick an impacted area that the model predicts could be improved significantly); and land availability and cost for the required new stormwater infrastructure (green and gray).

2. Using hydraulic modeling, determine the amount of LWD that could be placed in the stream system considering the limitations of flooding or other local hydraulic constraints.

3. Determine the amount of LWD necessary to achieve the target gravel disturbance metric in the remaining stream reaches (beyond the three modeled in the study) under future conditions with proposed mitigated hydrology and amended gravel size.

4. Determine alternative permutations of LID BMP's (e.g. rain gardens) and detention that result in a hydrologic response comparable to the ECY08 scenario. These alternatives may be more cost effective and/or practicable to achieve the predicted responses in some settings depending upon factors such as land cost, public acceptance of LID, etc.

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APPENDIX A. WATER QUALITY MONITORING REPORT

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Appendix A: Water Quality Data Report – Juanita Creek Stormwater Retrofit Project

August 2012

King County
Department of
Natural Resources and Parks
Water and Land Resources Division
201 South Jackson Street, Suite 600
Seattle, WA 98104

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Appendix A: Water Quality Data Report – Juanita Creek Stormwater Retrofit Project

Prepared for:

Stormwater Section.
King County Water and Land Resources Division
Department of Natural Resources and Parks

Submitted by:

Eric W. Ferguson
King County Water and Land Resources Division
Department of Natural Resources and Parks

Citation:

King County. 2012. Water Quality Data Report – Juanita Creek Stormwater Retrofit Project. Prepared by Eric W. Ferguson, Water and Land Resources Division. Seattle, Washington.

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EXECUTIVE SUMMARY

As part of the Juanita Creek Stormwater Retrofit planning project, a water quality monitoring study was designed to assess current baseflow and stormwater quality in Juanita Creek and support the development of a watershed hydrologic and water quality model. Water quality samples were collected during seven storm events and three base flow events over a period of 14 months – October 2008 to November 2009 from eight stations. Over the same time period, continuous (15-minute) monitoring of stream flow was conducted at six sites and four of these sites also recorded continuous water temperature.

Evaluation of previous monitoring data by the Washington State Department of Ecology has resulted in listing Juanita Creek as having impaired beneficial uses for aquatic life based on data for dissolved oxygen and water temperature, and impairment of beneficial uses for humans based on data for fecal coliform bacteria – an indicator of the potential presence of human pathogens. Data collected for the Juanita Creek Stormwater Retrofit planning project are consistent with those listings. However, this study also found levels of pH, copper and zinc that exceed state water quality criteria for the protection of aquatic life.

Dissolved oxygen (DO) concentrations were below the applicable state criterion of 9.5 mg/L during both base and storm events. Twenty-six of the 52 samples collected from the baseflow events and 64 of the 142 samples from storm events had DO values below 9.5 mg/L. The wetland sites, 4GI (wetland inlet) and 4GO (wetland outlet), had the lowest DO concentrations during baseflow events. Dissolved Oxygen concentrations during sampled storms were typically lower at more locations than during baseflow events. All of the Storm 6 (10/16-17/2009) event samples had DO levels below the criterion with concentrations ranging from 0.7 to 3.8 mg/L. It is unknown at this time if the data collected during Storm 6 was due to an equipment problem or a flushing occurrence that happens at the first fall storm.

The water temperature data collected as part of this study are a combination of grab samples and continuous sensor probes at stream flow gauge locations. The temperature criteria are based on the 7-day average of the daily maximum temperature. At the four sites with continuous temperature data the number of days that the ore Salmonid Rearing criterion of 16.0 °C was exceeded ranged from 35 to 128 days. The largest number of days that the temperature criterion was exceeded was at Station 27J-6G on Upper Juanita Creek. The Supplemental Spawning criterion of 13.0 °C applies only to the mainstem of Juanita Creek. The temperatures measured on the mainstem of the creek exceeded this criterion for a total of 56 days. The grab (discreet) data was collected at all sites and supports the continuous temperature data.

The pH data for all baseflow and storm events were within the state criteria of greater than 6.5 and less than 8.5 except for five storm samples. Four samples were below the criterion of 6.5 at sites 5GF and 4GI. A result of 13.9 from site 5GF during Storm 6 appears to be anomalous when compared to the total dataset of 142 storm samples whose values ranged from 6.3 to 8.0.

Fecal coliform were detected in all baseflow and storm event samples. The state criteria are based on the protection of Extraordinary Primary Contact Recreation use. The criteria are that

the geometric mean of the samples from a particular location not exceed50 CFU/100 mL[1] and that not more than 10 percent of all the samples from a site exceed 100 colonies per 100 mL. The geometric mean of all baseflow samples was 64 CFU/100ml while the geometric mean of all storm event samples was 765 CFU/100ml. Both of these results are above the Extraordinary Primary Contact Recreation criterion of 50 CFU/100ml. Storm 5 (August 10-11, 2009) had the highest geometric mean of all sampled storms with 3,823 CFU/100ml and site 6G had the highest geometric mean of all sites with 2,479 CFU/100ml and a peak concentration at site 5GF of 23,000 CFU/100ml. For the baseflow sampling events, the June 20, 2009 event had the highest geometric mean value with 204 CFU/100ml and site 6G had the highest geometric mean of all sites with 622 CFU/100ml.

For dissolved metals (copper and zinc), no baseflow event samples exceeded either the acute or chronic criteria. However, of the 142 observed storm event dissolved copper concentrations measured during the study, two exceeded the acute criterion and 12 exceeded the chronic criterion. Similarly, six of the 142 measured storm event dissolved zinc concentrations exceeded the chronic and acute criteria. Moreover, the storm events and site locations with exceedances for dissolved zinc were different than the sites where exceedances of dissolved copper criteria were observed.

In general, the data collected were of sufficient quality to support the development of a watershed hydrologic and water quality model that can be used to evaluate the effectiveness of various stormwater retrofit strategies in improving water quality and quantity conditions in the Juanita Creek watershed.

[1] Fecal coliform concentrations are reported as numbers of colony forming units or CFU per 100 millilters (mL).

1.0. INTRODUCTION

King County Department of Natural Resources and Parks have entered into a partnership with City of Kirkland and Washington State Department of Transportation for a Washington State Ecology grant to investigate potential benefits of retrofitting the storm water system in the Juanita Creek basin. This basin is almost completely developed with residential and commercial land uses, and drains to Juanita Bay on the northeastern shore of Lake Washington (Figure 1).

Land use in the basin is comprised of mostly residential and commercial developments (including a major interstate freeway corridor, I-405), two relatively large forested wetlands, and one small lake (Totem Lake) receiving storm water runoff from a large portion of the commercial development in the eastern headwaters of the basin.

Evaluation of previous monitoring data by the Washington State Department of Ecology has resulted in listing Juanita Creek as having impaired beneficial uses for aquatic life and human recreational use. The listings for these impairments are based on data for dissolved oxygen (DO) and water temperature (aquatic life use impairment) and on data for fecal coliform bacteria[2] (human recreational use impairment). As part of the Stormwater Retrofit grant, watershed models will be developed to characterize the hydrologic regime and water quality for existing conditions and future conditions with retrofits targeted to improve Juanita Creek stream habitat and beneficial uses. This report describes the results of a water quality monitoring study that will be used to support the development and calibration of the watershed models and provide a snapshot of existing water quality conditions for a range of parameters over a number of locations within the basin.

1.1 Goals and Objectives

Monitoring of water quality conditions occurred over a 14 month period starting in October 2008 and ending in November 2009. The goal was to measure conditions during all four seasons during storm and non-storm (i.e. baseflow) events. This design was intended to capture the seasonal variability in concentrations for a number of constituents from variable size storms during different times of the year and during periods when streamflows were predominantly driven by shallow groundwater contributions (i.e., baseflow conditions).

Parameters analyzed in the field during baseflow and storm event sampling events included dissolved oxygen, pH, temperature, and specific conductance. Continuous (15-minute) temperature measurements were also made at a subset of the sampling sites. Samples collected in the field were also taken back to the laboratory for analysis of nutrients, alkalinity, biochemical oxygen demand, total and dissolved organic carbon, chlorophyll a, fecal coliform bacteria, and total and dissolved trace metals (copper and zinc). Nutrient analyses included ammonia nitrogen, nitrate plus nitrite nitrogen, total nitrogen, orthophosphate phosphorus, and total phosphorus. Stream gages were also established to continuously (15-minute interval) measure water levels (which were then converted to flow rates) and temperature.. Additionally, calcium and magnesium concentrations were measured so that hardness could be calculated for

[2] Fecal coliform bacteria are used as an indicator of the potential presence of human pathogens that include pathogenic bacteria and viruses.

use in the equations that accounts for the effect of hardness on dissolved metal toxicity. Further details of this study design can be found in the Quality Assurance Project Plan and Sampling and Analysis Plan developed as part of this grant (King County 2008).

The results of this study are used to meet the objectives of this grant by: 1) characterizing existing conditions and 2) provide the necessary level of spatial and temporal stream conditions to support development of a predictive watershed model to be used to evaluate benefits resulting from various stormwater retrofitting strategies.

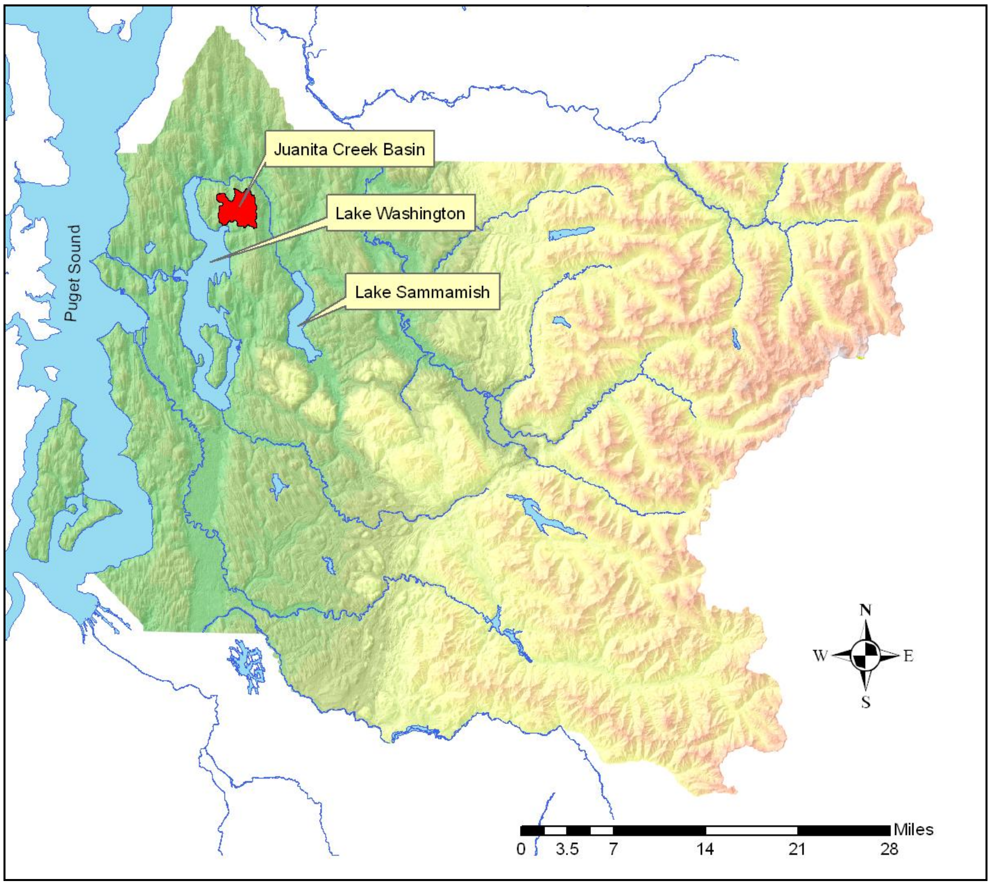


Figure 1. Map showing the location of the Juanita Creek basin in King County.

2.0. WATER QUALITY CRITERIA

Water quality criteria for surface waters are established to protect public health and maintain the public enjoyment of the waters and the propagation and protection of fish, shellfish, and wildlife as defined by Washington Administrative Code (WAC) 173-201A. As defined by code, beneficial uses include maintaining the quality of potable water and water essential to farming activities, preserving aesthetic values, and providing adequate water quality for aquatic life (fish and wildlife).

The federal Clean Water Act, adopted in 1972, requires that all states restore their waters to be “fishable and swimmable.” Section 303(d) of the Clean Water Act established a process to identify and clean up polluted waters. Every two years, all states are required to perform a *water quality assessment* of the quality of surface waters in the state, including all the rivers, lakes, and marine waters where data were available. As noted in Section 1.0, Juanita Creek has 303(d) listings for dissolved oxygen, water temperature, and fecal coliform.

The criteria that apply to Juanita Creek and that are used in this report are adopted by the state of Washington (WAC 173-201A-200) and are shown in Table 1.

Table 1. Water quality criteria for surface water in Washington State — WAC 173-201A.

Parameter	**Water Quality Criteria**	
	Core Rearing and Migration	Supplemental Spawning
Temperature[a]	16.0 °C	13.0 °C
Dissolved Oxygen[b]	9.5 mg/L	
pH	Range of 6.5 to 8.5	
	Extraordinary Primary Contact Recreation	
Fecal Coliform[c]	50 CFU/100 ml	
	Acute	Chronic
Ammonia[d]	24.1 mg/L	2.1 mg/L
Copper (dissolved)[e]	8.7 µg/L	9.3 µg/L
Zinc (dissolved)[e]	62.6 µg/L	85.6 µg/L

[a] = Temperature values shall not exceed these criteria based on the 7-day average of the daily maximum temperature. Supplemental Spawning criterion applies to the period from September 15th to May 15th as defined in Waters Requiring Supplemental Spawning and Incubation Protection For Salmonid Species (Ecology, 2011).

[b] = Dissolved oxygen values shall not be less than this criterion.

[c] = Fecal coliform bacteria concentrations should not exceed a geometric mean value of 50 CFU/100 mL and no more than 10 percent of the samples should exceed 100 CFU/100 mL. Or if less than 10 samples, no sample shall exceed 100 CFU/100 mL.

[d] = Formula based criteria – numbers shown are based on salmonids present with temperature 15° C and pH of 7.0. see WAC 173-201A for formula.

[e] = Formula based criteria – numbers shown are based on average hardness of 49.1 mg/L for storm events used in the acute criteria and an average hardness of 79.0 mg/L for baseflow events for the chronic criteria. see WAC 173-201A for formulas.

Units: C: Celsius; µg/L: microgram per liter; mg/L: milligrams per liter; CFU/100ml: colony forming units per 100 milliliters

3.0. STUDY DESIGN AND METHODS

Data were collected at a number of locations (eight in total) during storm and non-storm events which can be grouped into three categories: 1) continuously recording instruments during the entire study period, 2) parameters that were measured in the field, and 3) lab analyses performed on grab samples taken at each of the monitoring stations. Samples were collected during seven storm events over the course of two wet seasons and three base flow events within the same water year over a period of 14 months.

3.1.1 Sampling Locations

Locations of sample collections were located on all five major tributaries near the confluence with Juanita Creek main stem, and near the mouth of the basin itself. These six locations have an associated continuously measuring flow gauge site nearby (Table 2 and Figure 2). Two additional water quality sampling locations were added to assess a significantly large wetland that receives runoff from a commercial business district (i.e. shopping mall) via a small lake and neighboring high density residential development (Table 2 and Figure 2).

3.1.1.1 Continuous Temperature

Four of the six gauge locations had continuous (15-minute) temperature recorders (Table 2). Temperature data collection started in late October 2008 for the three upper Juanita Creek tributaries sites. Juanita Creek site (27a) near the mouth is an existing monitoring site and has had temperature and flow measured since 2003.

3.1.2 Sampling Criteria

Criteria for initiating sampling during storm and baseflow events are described below.

3.1.2.1 Storm Events

Sampling events for a storm were determined using a few considerations; 1) was the lab available for receiving samples collected, 2) when was the last storm sample collected, and 3) was the forecast predicting a storm of 0.25 inches (or greater) in 24 hours. When all three conditions were met and the storm forecast remained at 0.25 inches or greater, sampling would begin at the onset of rainfall. However, since the total amount of rainfall during the sampled storm could not be known until sampling was completed; two storms did not ultimately result in more than 0.25 inches of rainfall. Samples were collected and used in those situations. During the storm event, the intention was to collect three grab samples representing the beginning, middle, and end of the storm. However, some storms were either too short in duration to collect the third sample and/or technical difficulties with equipment reduced the number of samples.

3.1.2.2 Baseflow Events

Baseflow sampling targeted periods during which no precipitation had occurred within at least 3 days. Baseflow events were scheduled events that could have been delayed if these antecedent conditions changed. Three sampling events were planned to occur; one sampling event in each of three seasons: fall, winter, and summer.

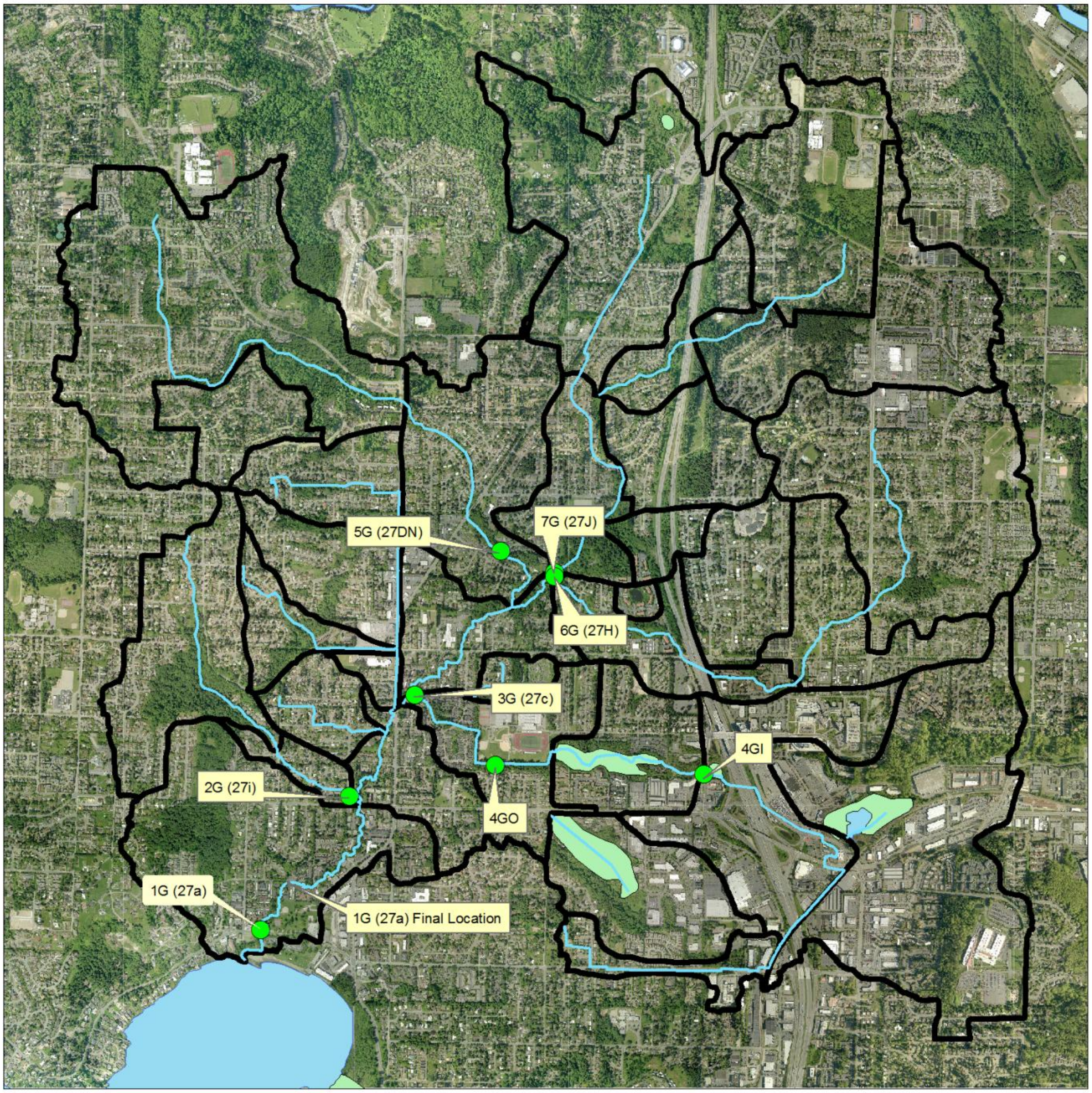


Figure 2. Map showing water quality and stream gauging locations within Juanita Creek Basin.

Note: The eight water quality sites are labeled. The six gauge locations are also designated with the site id in parenthesis with an example given: 1G (27a) for water quality site — 1G at or near gauge — 27a.

Table 2. Monitoring locations for water quality sampling and stream gauging.

Locator	Site Description	Gauge Sites	
		Flow	**Temperature**
J446_1G	Juanita Creek, at mouth	27a	27a
J446_2G	Lower Juanita trib - west	27i	No temperature
J446_3G	Lower Juanita trib - east	27c	No temperature
J446_4GI	Wetland Entrance	No flow gauge	No temperature
J446_4GO	Wetland Exit	No flow gauge	No temperature
J446_5GF	Upper Juanita - west trib	27dn	27dn
J446_6G	Upper Juanita - east trib	27h	27h
J446_7G	Upper Juanita - central trib	27j	27j

Baseflow samples were collected twice in one day at the eight sampling sites. Once in the morning (goal of within 1-hour of sunrise), and once in the early evening (target 5:pm)—such that if there were any consistent indications of potential diurnal variation present, this timing of sample collection might capture some aspect of it.

3.1.3 Sampling Methods

Sampling methods included the use of in situ probes capable of making instantaneous readings within the water column in the field, and grab samples that were collected and then transported to the King County Environmental Laboratory (KCEL) for analysis. With either method, consistent sample handling procedures were necessary to maintain sample integrity and provide data that is as defensible and as high a quality as possible under the sampling conditions. Details of the sampling methods used are provided in King County (2008).

3.1.3.1 Field Measurements

Field measurements for temperature, dissolved oxygen, specific conductance, and pH using Hydrolab sondes was be done according to KCEL SOP # 205v4 (Field Measurement using an Attended Hydrolab – (see King County 2008).

3.1.3.2 Grab Samples

Samples were collected from the thalweg, within free-flowing stream sections, and away from channel boundaries. Where access was from a bridge or roadway because of loss of access, the sample was collected from the upstream side. These procedures are described in KCEL Standard Operating Procedure 214v3 (see King County 2008).

Field filtration was done for dissolved metals and nutrients. Filtration occurred within 15 minutes of sample collection.

4.0. RESULTS

This section summarizes the data collected from October 2008 through November 2009 – 14 month period. The results are summarized separately for baseflow and storm sampling events. An overall summary of the conditions experienced sampling events is provided (Summary of Events) and then results are summarized by the type of measurement (Field vs. Laboratory).

4.1 Summary of events

A total of three baseflow events and seven storm events were collected (Table 3). Figure 3 provides a graphical representation of the precipitation, flow and timing of sampling events.

4.1.1 Baseflow events

The baseflow events were conducted on October 28, 2008, January 20, 2009 and June 10, 2009. A summary of these events are presented in Table 3. These represent different time periods (fall, winter, spring/summer) as outlined in King County (2008). The criteria outlined in section 3.1.2.2 were met before each base flow event.

The collection of the samples were taken over two time periods during the same day with the initial set of samples during the morning and the second set of samples were taken in the late afternoon. Each event was to have one field replicate collected, although this was not always the case as described below. In total, 51 samples were collected during baseflow events (Table 3).

4.1.2 Storm events

Storm samples were collected from seven events from February through November 2009. Table 3 presents a summary of the storm events and their sample collection details.

Three of these events did not collect three rounds of samples as outlined in the QAPP (King County 2008). Storm 1 only had one round of samples with no field replicate due to the lack of sufficient precipitation. This storm did not track as predicted with less rainfall occurring within the basin. Storms 2 and 4 only had two rounds of sample collection for similar reasons. In total, 142 samples were collected during storm events (Table 3).

4.2 Field Measurements

Field measurements for temperature, dissolved oxygen, specific conductance, and pH were done using calibrated Hydrolab sondes. Temperature was also measured continuously at four of the six gauge locations.

4.2.1 Baseflow Events

The baseline data of field parameters do show some variability depending on the parameter and the season of sample collection (Table 4, Table 5 and Appendix 1). Washington State does have surface water quality criteria for three of the four field parameters collected, Table 1.

Table 3. Summary of baseflow and storm sampling events and sample collection details.

Events	**Collect Date**	**Samples Collected^**	**Sample Duration[o]**	**Rainfall total (in)[•]**	**Max flow (cfs)***
Base 1	10/28/2008	2 rounds - 0 FREPs (16)	4 hours	0.00	2.9
Base 2	1/20/2009	2 rounds - 2 FREPs (18)	4 hours	0.00	6.5
Base 3	6/10/2009	2 rounds - 1 FREPs (17)	4 hours	0.00	3.4
Storm 1	2/25/2009	1 round - 0 FREPs (8)	3.5 hours	0.14	29.7
Storm 2	3/14/2009	2 rounds - 1 FREPs (17)	6 hours	0.06	21.5
Storm 3	5/04-5/05/2009	3 rounds - 2 FREPs (26)	12 hours	1.08	100.6
Storm 4	5/13-5/14/2009	2 rounds - 2 FREPs (18)	7 hours	0.11	17.3
Storm 5	8/10-8/11/2009	3 rounds - 2 FREPs (26)	12 hours	0.40	18.1
Storm 6	10/16-10/17/2009	3 rounds - 2 FREPs (26)	10.5 hours	0.30	43.8
Storm 7	11/16-11/17/2009	3 rounds - 1 FREPs (25)	12.5 hours	0.92	71.4

^ = refers to total number of samples collected during an event in parenthesis. Storm events were planned to have 3 rounds with 2 field replicates (FREPs) with a total of 26 samples. Baseflow events were scheduled to have 2 rounds of samples with 1 field replicate for a total of 17 samples.

[o] = refers to total time during sample collection.

[•] = refers to total rainfall during sample collection.

***** = refers to maximum flow measured at gauge 27a (mouth of Juanita Creek) during the sample duration.

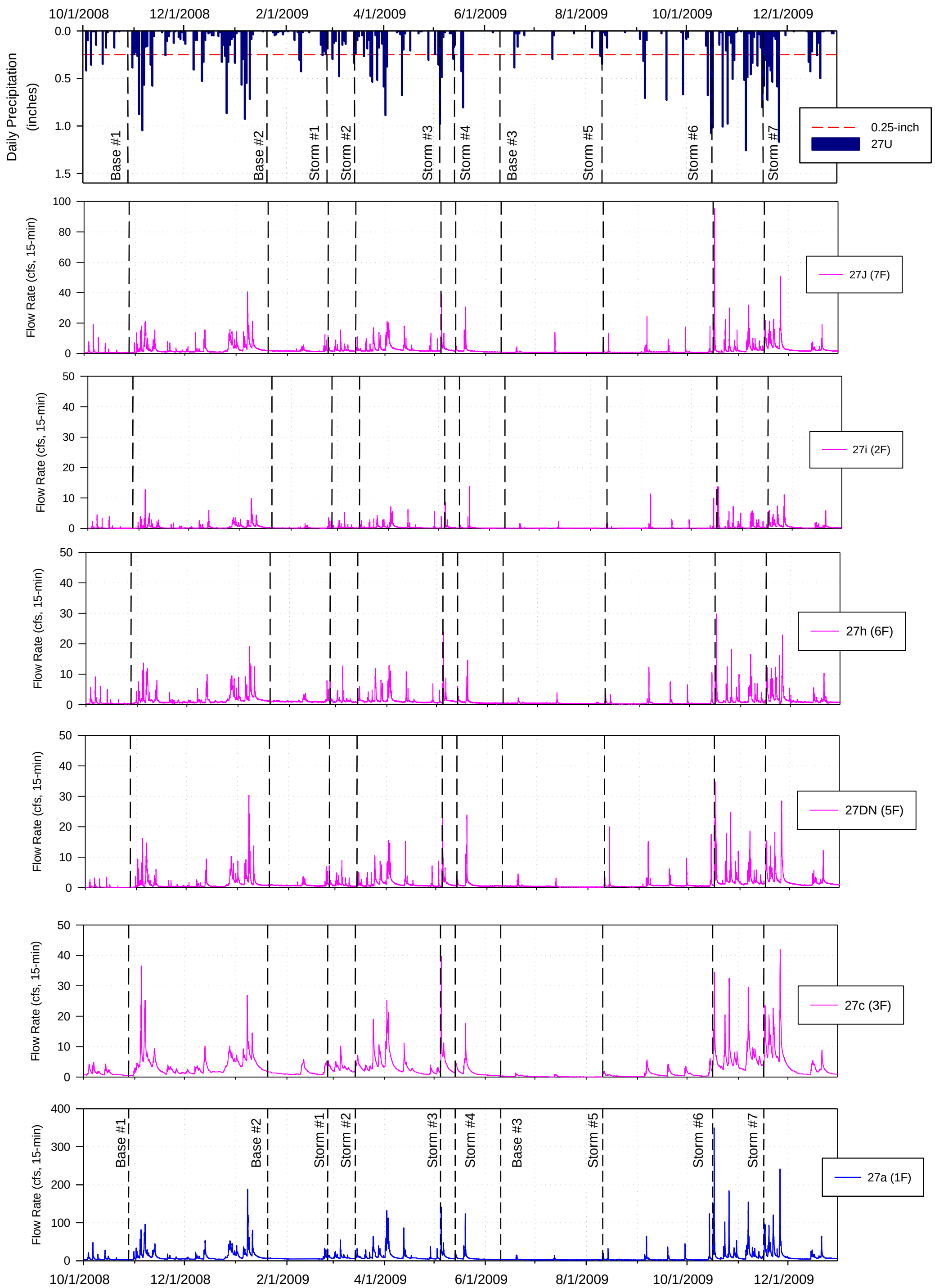


Figure 3. Summary of daily accumulated precipitation, 15-minute flow rates, and timing of sampling events.

Note: Gage flow data are 15 minute intervals while precipitation are daily totals..

4.2.1.1 Dissolved Oxygen

Dissolved oxygen data collected during base events ranged from 0.7 to 14.2 mg/L with an average value of 9.1 mg/L. All sites had at least one measured DO level below the state criterion of 9.5 mg/L (Table 5 and Table 1). The DO data from all sites show a pattern of greatest values during the winter event (01/20/2009) and the lowest values during the spring/summer event (06/10/2009). Almost all of the DO data from the spring/summer base event were below the 9.5 mg/L criterion. DO concentrations measured at Station 4GI were all below 4.0 mg/L. In total, 27 of the 52 baseflow event samples had DO values below the state criterion of 9.5 mg/L (Table 5, Table 1, and Appendix 1).

4.2.1.2 Temperature

Temperature data were collected by grab (discreet) and continuous methods. The state criterion for stream temperature is based on a 7-day average of daily maximum temperatures (7-DADMax) (Table 1). Continuous temperature data were collected at four of the six gauge sites (Table 2). Continuous temperature data collection started at the three upper tributary gauge sites in late October 2008 and continuous temperature data has been collected at the the gauge site at the mouth since 2003. At the four sites with continuous temperature data the number of days that the Core Salmonid Rearing criterion of 16.0 °C was exceeded ranged from 35 to 128 days (Table 6 and Figure 4). The largest number of days that the temperature criterion was exceeded was a Station 27J-6G on Upper Juanita Creek (Figure 4).

Juanita Creek also has a Supplemental Spawning temperature criterion for the mainstem of 13° C assessed from September 15th until May 15th (Ecology, 2011). Two periods of assessment were done during this 14 month study. They are: 9/15/2008 to 05/15/2009 and 9/15/2009 to 11/30/2009. The Supplemental Spawning criterion does not apply to the upper tributaries of Juanita Creek, but these stations were included in this assessment due to their potential influence on the mainstem reach of the creek. During the first period, 9/15/2008 to 05/15/2009, two sites (27a and 27h) had 7-DADMax that exceeded the standard of 13 °C (Table 6 and Figure 4). During the second period of assessment from 9/15/2009 to 11/30/2009, all four sites had at least 18 days over the standard with site 27h (6G) having the most (29 days) (Table 6 and Figure 4).

The grab (discreet) data was collected at all sites and supports the continuous temperature data findings. The temperature data collected during base events ranged from 2.5 to 19.7 °C with an average value of 9.8 °C. This temperature data also showed a pattern which is opposite of the dissolve oxygen with lowest values occurring the winter event and highest values during the spring/summer event (Table 5). This pattern is consistent with the seasonal variation observed in the continuous temperature data. Lower temperatures in the morning and higher temperatures in the evening sample (see Table 5) are also consistent with the diel variability observed in the continuous temperature data. Every sample collected during the afternoon round during the spring/summer base event (06/10/2009) had temperatures over 16 °C (Table 5).

Table 4. Summary of base event water quality for all parameters. *State water quality criteria are presented in table 1.

PARMNAME	Avg-result	Min	Max	StDev	Total Samples	Detected	MDL	UNITS	threshold exceed
Ammonia Nitrogen	0.04	0.01	0.34	0.07	51	41	0.005	mg/L	**0**
Biochemical Oxygen Demand	4.02	2.88	5.16	1.61	51	2	2.0	mg/L	N/A
Calcium, Total,	16,251.0	12,100	21,500	2,264.5	51	51	10	ug/L	N/A
Chlorophyll a	1.03	0.51	3.06	0.61	51	17	0.5	ug/L	N/A
Copper, Dissolved	0.82	0.33	2.67	0.38	51	50	0.1	ug/L	**0/0**
Copper, Total	1.05	0.39	2.57	0.57	51	51	0.1	ug/L	**0/0**
Dissolved Organic Carbon	4.19	1.93	7.39	1.25	51	51	0.5	mg/L	N/A
Fecal Coliform	276.31	1.00	5,000	722.0	51	51	>1	CFU/100ml	**51***
Hardness, Calc	79.0	50.7	109	17.2	51	51	0.066	mg CaCO3/L	N/A
Magnesium, Total	9,331.0	4,950	15,400	3,039.2	51	51	10	ug/L	N/A
Nitrite + Nitrate Nitrogen	1.00	0.02	2.53	0.70	51	50	0.01	mg/L	N/A
Orthophosphate Phosphorus	0.03	0.01	0.09	0.02	51	51	0.002	mg/L	N/A
Pheophytin a	1.86	1.60	2.12	0.25	51	4	1	ug/L	N/A
Total Alkalinity	79.0	50.40	108	16.08	51	51	1	mg CaCO3/L	N/A
Total Nitrogen	1.23	0.26	2.68	0.67	51	51	0.05	mg/L	N/A
Total Organic Carbon	4.45	1.77	8.84	1.49	51	51	0.5	mg/L	N/A
Total Phosphorus	0.059	0.02	0.17	0.03	51	51	0.005	mg/L	N/A
Total Suspended Solids	3.60	0.50	18.0	4.00	51	46	0.5	mg/L	N/A
Zinc, Dissolved	4.97	1.40	31.7	6.69	51	51	0.5	ug/L	**0/0**
Zinc, Total	5.91	1.50	30.8	6.80	51	51	0.5	ug/L	**0/0**
Field Parameters									
pH, Field	7.31	6.48	8.05	0.46	54	54	0.1	pH	**0**
Temperature, Field	9.82	2.49	19.73	4.86	54	53	0.1	deg C	**11**
Dissolved Oxygen, Field	9.06	0.71	14.20	3.30	54	54	0.5	mg/L	**27**
Specific Conductance	202.1	149	272	34.33	54	54	0.5	µS/cm	N/A

Table 5. A summary of field parameters collected during base events. Bold red refers to sample results over state criteria of water quality — WAC 173-201A.

Site	Event	flow (cfs)	Date	pH	Specific Conductance (μS/cm)	Dissolved Oxygen (mg/L)*	Temperature (deg C)
1G	base 1	2.8	10/28/2008	7.3	210	10.6	9.0
		2.9		7.4	210	10.7	10.0
	base 2	6.5	1/20/2009	7.2	198	12.4	5.0
		6.5		7.2	198	12.4	5.0
		6.5		7.3	199	12.0	5.6
	base 3	3.4	6/10/2009	7.4	213	**8.8**	14.0
		3.4		7.5	214	**8.6**	17.2
2G	base 1	0.1	10/28/2008	7.9	272	11.5	8.8
		0.1		8.0	268	10.3	10.4
	base 2	0.2	1/20/2009	7.7	221	14.2	3.9
		0.2		7.9	223	12.7	4.8
	base 3	0.1	6/10/2009	8.1	254	9.6	13.9
		0.1		8.0	254	**8.9**	17.1
3G	base 1	0.5	10/28/2008	6.8	161	**9.3**	8.7
		0.5		6.8	161	**9.4**	8.7
		0.5		6.8	160	**9.4**	9.6
	base 2	1.8	1/20/2009	6.8	152	13.4	4.2
		1.7		6.9	153	11.6	5.2
	base 3	0.4	6/10/2009	6.9	169	**8.0**	14.4
		0.3		6.9	172	**7.6**	17.0
4GI	base 1	No Flow	10/28/2008	6.6	168	**1.4**	8.7
				6.6	197	**1.2**	9.9
	base 2		1/20/2009	6.5	157	**3.6**	4.0
				6.6	159	**3.3**	4.6
				6.6	160	**3.2**	4.6
	base 3		6/10/2009	6.7	180	**1.0**	16.8
				6.6	182	**0.7**	18.4
4GO	base 1	No Flow	10/28/2008	6.9	157	**7.6**	6.9
				6.9	157	**7.8**	7.9
	base 2		1/20/2009	6.8	149	10.3	2.5
				6.9	149	10.1	3.3
	base 3		6/10/2009	7.2	181	**5.7**	16.7

Site	Event	flow (cfs)	Date	pH	Specific Conductance (µS/cm)	Dissolved Oxygen (mg/L)*	Temperature (deg C)
				7.2	182	**6.1**	19.7
				7.2	182	**6.2**	19.7
5GF	base 1	0.1	10/28/2008	7.7	253	11.1	8.0
		0.1		7.7	249	10.9	9.4
	base 2	0.9	1/20/2009	7.7	231	13.4	3.7
		1.0		7.7	232	12.8	4.6
	base 3	0.6	6/10/2009	7.7	252	**9.2**	13.6
		0.5		7.7	252	**9.0**	16.4
6G	base 1	0.3	10/28/2008	7.6	212	10.3	8.8
		0.4		7.6	192	9.6	10.8
	base 2	0.8	1/20/2009	7.5	205	11.9	6.1
		1.0		7.5	205	11.1	7.0
	base 3	0.4	6/10/2009	7.7	208	**9.0**	14.0
		0.4		7.8	208	**8.2**	19.7
J446_7G	base 1	0.7	10/28/2008	7.6	229	10.9	8.5
		0.6		7.6	227	10.0	10.0
		0.6		7.6	228	10.1	10.0
	base 2	1.9	1/20/2009	7.5	214	12.4	5.1
		1.9		7.6	214	11.6	5.9
	base 3	0.8	6/10/2009	7.8	226	**9.4**	13.0
		0.8		7.8	225	**9.4**	13.0
		0.8		7.8	228	**9.1**	16.2

"*" = refers to Washington State criterion for Dissolved Oxygen of 9.5 mg/L (see Table 1).

4.2.1.3 pH

The pH values of all samples collected during the baseflow events were within the state standard of 6.5 to 8.5 (Table 5 and Table 1).

4.2.1.4 Specific Conductance

Specific conductance data collected during baseflow events ranged from 149 to 272 µS/cm with an average value of 202 µS/cm. Typically, the lowest values were collected during the winter baseflow event (1/20/2009) and higher values in summer to early fall (Table 5).

Table 6. A summary of days over the temperature criteria.

Gauge sites	Site name	Date started	16°C Core Standard	13°C Spawning Standard^	
			10/01/2008 - 11/30/20009	9/15/2008- 5/15/2009	9/15/2009- 11/30/20009
27a (1G)	Juanita Creek - mouth	10/13/2003	89	36	23
27dn (5G)	Upper Juanita -west	10/28/2008	35*	0*	18
27h (6G)	Upper Juanita -central	10/23/2008	128*	26*	29
27j (7G)	Upper Juanita -east	10/24/2008	40*	0*	18

"^" = The supplement spawning standard for Juanita Creek is for main stem and does not apply to the upper tributaries. Data were assessed and compared for additional information.

"*" = These sites had data collection start in late October.

Note: Temperature values shall not exceed this criteria based on the 7-day average of daily maximum temperatures. Supplemental Spawning criteria is from September 15th to May 15th. see WAC 173-201A.

4.2.2 Storm Events

The storm data show some variability depending on the parameter and the storm event sampled **(**Table 7). All storm data including field parameters are presented in Table 8.

4.2.2.1 Dissolved Oxygen

Dissolved oxygen data collected during baseflow events ranged from 0.7 to 22.7 mg/L with an average value of 8.8 mg/L (Table 7). All samples from Storm 6 (10/16/2009) had DO data below the state criterion of 9.5 mg/L with no values above 4.0 mg/L (Figure 5 and Table 7). The DO data for the wetland sites, 4GI and 4GO, consistently had results below the state criterion. For storm samples, 64 of the 142 samples were below the state criterion of 9.5 mg/L (Table 8). DO concentrations during storms were the highest at site 2G. This is suspected to be the result of samples taken in a plunge-pool approximately 3-feet vertically below the invert of the culvert draining that sub basin.

4.2.2.2 Temperature

Temperature data collected during storm events ranged from 5.4 to 18.9 °C with an average value of 11.7 °C (Table 7). Temperature data from storm events reflected the time of year of the sample collection with winter storms being the coldest while spring/summer storm being the warmest (Figure 4). Twenty-five of the 26 samples collected during the Storm 5 event (08/10-11/2009) had temperature values over 16.0 °C (Table 8).

4.2.2.3 pH

The pH levels measured at all sites and storms were within the state criteria of greater than 6.5 and less than 8.5 except for 5 samples (Table 7 and Table 8). Four samples were below 6.5 while one sample was above 8.5. The result of 13.9 from site 5GF during Storm 6 appears to be anomalous when compared to the total dataset of 142 storm samples (Table 7) which ranged from 6.3 to 8.0.

4.2.2.4 Specific Conductance

Specific conductance data collected during storm events ranged from 33 to 217 µS/cm with an average value of 118 µS/cm (Table 7). The specific conductance data for a few sites (1G and 3G) did show a relationship of lower conductance with increasing stream flows (Figure 6). Site 1G, Juanita Creek at the mouth, has the maximum flows of any site measured and shows this association (Figure 6). This is due to the runoff of rain water with relatively low conductance (ionic strength) during storm events relative to the baseflow derived from groundwater that generally has higher conductance.

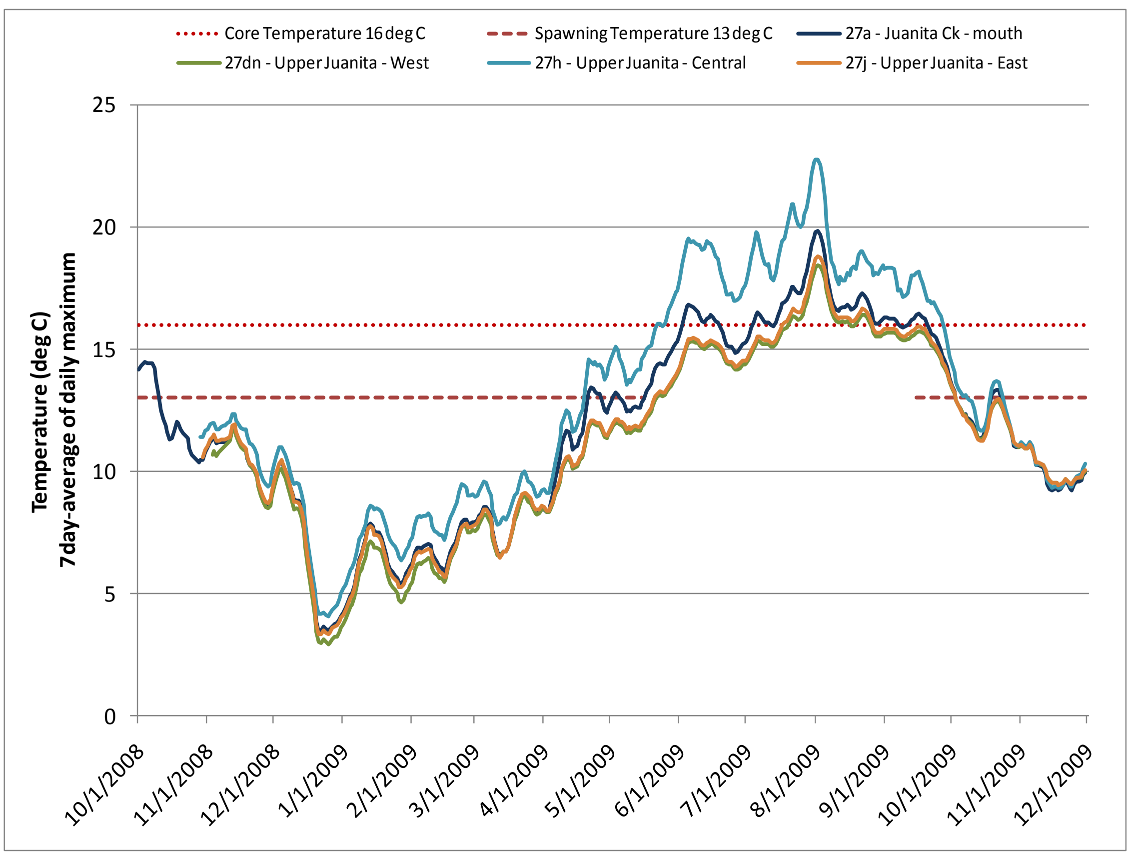


Figure 4. Temperature (7-day average of daily maximum) data for Juanita Creek.

Note: Temperature values shall not exceed the criterion based on the 7-day average of daily maximum temperatures. Spawning criterion is from September 15th to May 15th, see table 6.

Table 7. A summary of field parameters collected during storm events.

Parameter	Event	Average	Min	Max	StDev	Units
Specific Conductance	ALL	118.5	33	217	40.0	µS/cm
	storm 1	118.5	68	187	36.9	
	storm 2	155.2	127	202	20.1	
	storm 3	102.3	33	212	38.0	
	storm 4	134.5	96	177	21.1	
	storm 5	153.7	97	217	30.0	
	storm 6	93.1	44	162	34.2	
	storm 7	88.7	53	148	24.7	
Dissolved Oxygen	ALL	**8.8**	**0.7**	22.7	4.0	mg/L
	storm 1	11.0	**7.6**	12.1	1.6	
	storm 2	11.2	**7.6**	12.7	1.5	
	storm 3	9.8	**4.5**	12.0	2.1	
	storm 4	10.0	**3.3**	12.6	2.5	
	storm 5	**8.8**	**4.4**	11.5	2.1	
	storm 6	**2.4**	**0.7**	**3.7**	0.8	
	storm 7	11.4	**1.6**	22.7	4.8	
pH	ALL	7.1	**6.3**	**13.9**	0.65	*unitless*
	storm 1	7.3	7.0	7.7	0.27	
	storm 2	7.4	6.7	8.0	0.36	
	storm 3	6.9	6.5	7.2	0.22	
	storm 4	6.9	**6.4**	7.3	0.25	
	storm 5	7.3	6.7	7.6	0.25	
	storm 6	7.1	**6.3**	**13.9**	1.4	
	storm 7	7.0	6.6	7.5	0.21	
Temperature	ALL	11.7	5.4	18.9	3.3	deg C
	storm 1	7.2	6.8	7.7	0.31	
	storm 2	6.3	5.4	7.2	0.51	
	storm 3	11.4	10.2	12.2	0.59	
	storm 4	11.1	9.9	11.9	0.57	
	storm 5	17.0	15.8	18.9	0.76	
	storm 6	13.7	12.9	14.6	0.53	
	storm 7	10.0	7.2	11.0	0.69	

Note: Bold red refers to sample results over state water quality criteria

Table 8. Summary of storm event water quality for all parameters.

PARMNAME	Avg-result	Min	Max	StDev	Total Samples	Detected	MDL	UNITS	Exceedance Frequency *
Ammonia Nitrogen	0.09	0.0	1.3	0.17	142	135	0.005	mg/L	**0**
Biochemical Oxygen Demand	6.21	2.2	29.4	5.74	142	52	2	mg/L	N/A
Calcium, Total	10,668.7	4,280	20,800	3,128.9	142	142	10	ug/L	N/A
Chlorophyll a	4.98	0.7	27.4	5.16	142	91	0.5	ug/L	N/A
Copper, Dissolved	2.90	1.0	13.8	2.07	142	139	0.4	ug/L	**2/12**
Copper, Total	8.23	1.0	89.7	10.36	142	142	0.4	ug/L	
Dissolved Organic Carbon	7.81	3.3	40.1	4.43	142	142	0.5	mg/L	N/A
Fecal Coliform	2,144.3	5	23,000	3,569.7	142	142	>1	CFU/100ml	**142***
Hardness, Calc	49.06	17.7	98.2	15.26	142	142	0.066	mg CaCO3/L	N/A
Magnesium, Total	5,444.9	1,620	11,600	2,050	142	142	10	ug/L	N/A
Nitrite + Nitrate Nitrogen	0.56	0.0	2.2	0.46	142	142	0.1	mg/L	N/A
Orthophosphate Phosphorus	0.03	0.0	0.3	0.04	142	142	0.006	mg/L	N/A
Pheophytin a	7.72	1.2	25.4	6.13	142	33	1	ug/L	N/A
Total Alkalinity	44.01	13.2	85.3	15.15	142	142	1	mg CaCO3/L	N/A
Total Nitrogen	1.22	0.3	5.2	0.82	142	142	0.5	mg/L	N/A
Total Organic Carbon	12.18	3.8	54.4	9.28	142	142	0.5	mg/L	N/A
Total Phosphorus	0.15	0.0	1.1	0.15	142	142	0.005	mg/L	N/A
Total Suspended Solids	69.26	1.8	1,530.0	161.54	142	142	0.5	mg/L	N/A
Zinc, Dissolved	16.12	2.3	292.0	38.76	142	139	0.5	ug/L	**6/6**
Zinc, Total	40.85	4.2	341.0	52.38	142	142	0.5	ug/L	
Field Parameters									
pH, Field	7.11	6.3	13.9	0.65	146	146	0.1	No units	**5**
Temperature, Field	11.70	5.4	18.9	3.34	146	146	0.1	deg C	**25**
Dissolved Oxygen, Field	8.83	0.7	22.7	3.98	146	142	0.5	mg/L	**49**
Specific Conductance, Field	118.5	33	217	40.0	146	146	0.5	uS/cm	N/A

*State water quality criteria are presented in table 1. Dissolved metal data are presented as acute/chronic threshold.

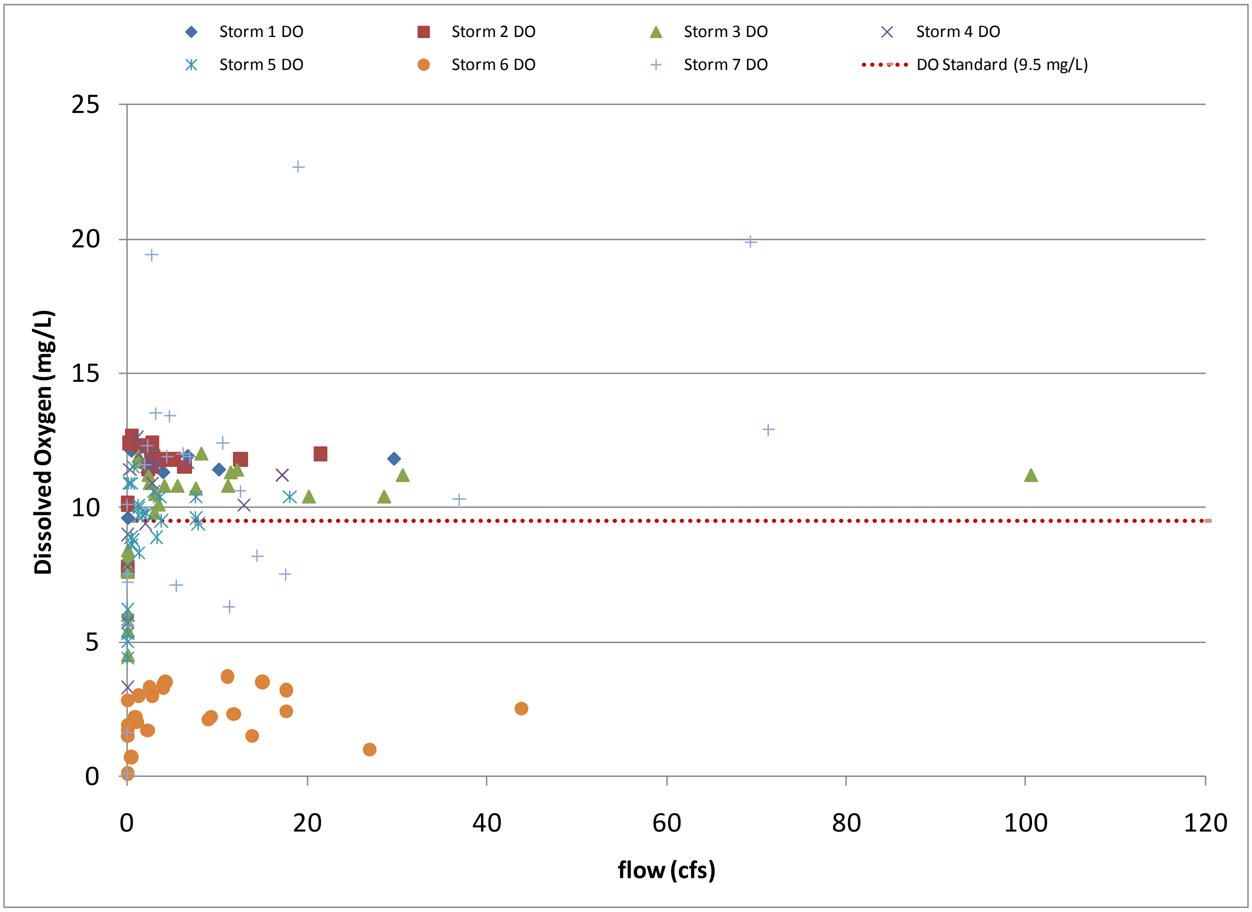


Figure 5. A scatter graph of dissolved oxygen and flow data for storm events.

Note: no flow measurements were taken at wetland sites (4GI and 4GO).

4.3 Lab Analysis

This section summarizes the data analyzed at the King County Environmental Laboratory for all samples collected. The method, detection limits and other laboratory analysis details can be found in the Sampling and Analysis plan, King County 2008.

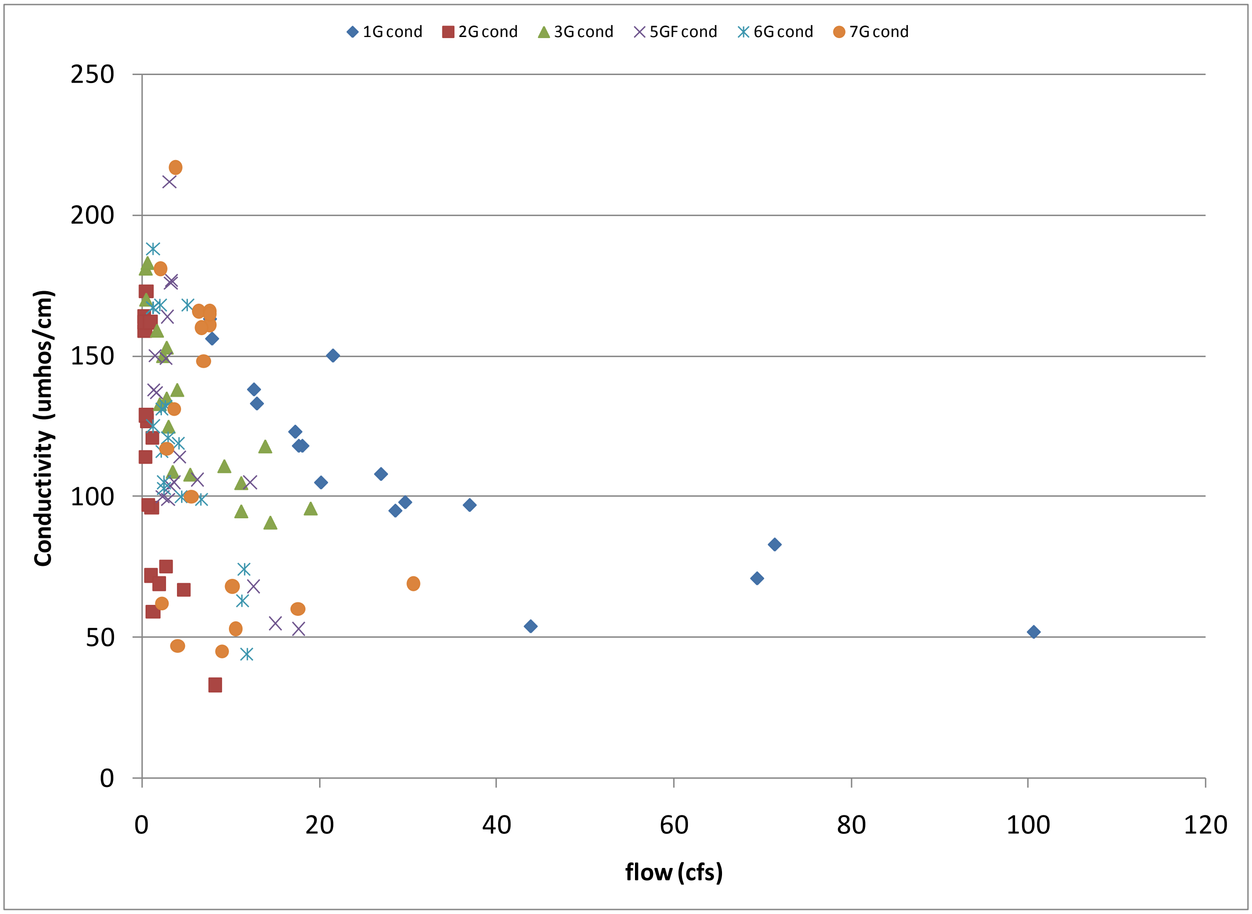


Figure 6. A scatter graph of specific conductance and flow data for storm events.

4.3.1 Conventional Parameters

Results for baseflow and storm event sampling for parameters analyzed in the laboratory are discussed below. All laboratory data are summarized in Table 4, Table 8, and Appendix 1.

4.3.1.1 Alkalinity

The baseflow event alkalinity ranged from 50.4 to 108 mg/L $CaCO_3$ and the storm event alkalinity ranged from 13.2 to 85.3 mg/L $CaCO_3$. The average values for the baseflow and storm events were 79.0 mg/L and 44.0 mg/L $CaCO_3$, respectively. Alkalinity values for all sites were lower for the storm samples than during baseflow events and this is likely caused by the increased runoff of rain water into the stream during storm events which has lower alkalinity (i.e., buffering capacity) than baseflow derived from groundwater during storm events. This is consistent with the specific conductance results presented above.

4.3.1.2 Total Suspended Solids

The baseflow event total suspended solids (TSS) values ranged from 0.5 to 18 mg/L and the storm event TSS values ranged from 1.8 to 1,530 mg/L. The average values for the base and storm events were 3.6 mg/L and 69.2 mg/L, respectively. Total suspended solids values for all sites were higher for the storm samples than during baseflow events except for site 4GI. This site

had similar values for both baseflow and storm events. The general increase in TSS values during storm events is likely caused by a combination of rainfall washoff from the land, resuspension and transport along the stream bed, and erosion and transport from the stream bank. However, the highest concentrations did not always occur at the highest observed flows for those sites with associated flow measurements (Figure 7).

4.3.1.3 Dissolved Organic Carbon

The baseflow event dissolved organic carbon (DOC) values ranged from 1.9 to 7.4 mg/L and the storm event DOC values ranged from 3.3 to 40.1 mg/L. The average values for the baseflow and storm events were 4.2 mg/L and 7.8 mg/L, respectively. Dissolved organic carbon values for all sites were higher for the storm samples than during baseflow events.

4.3.1.4 Total Organic Carbon

The baseflow event total organic carbon (TOC) values ranged from 1.8 to 8.8 mg/L and the storm event TOC values ranged from 3.8 to 54.4 mg/L. The average values for the base and storm events were 4.4 mg/L and 12.2 mg/L, respectively. Total organic carbon values for all sites were higher for the storm samples than during base events.

4.3.1.5 Biochemical Oxygen Demand

The baseflow event biochemical oxygen demand (BOD) values were all below the method detection limit of 2.0 mg/L, except for two samples with reported concentrations of 2.9 and 5.2 mg/L at Stations 4GI and 6G, respectively. The storm event BOD values ranged from below the laboratory method detection limit to 29.4 mg/L. Twenty-six of the 85 samples were below the method detection limit of 2.0 mg/L. Additionally, seven samples from Storm 2 were qualified with an “R” which refers not reported and text added stated ‘No result available due to contamination of dilution water’. Each site had at least three storm samples above the detection limit and the average values for storm events were 2.7 mg/L and 9.5 mg/L. Biochemical oxygen demand values for the storm samples were higher than baseflow event samples for all sites.

4.3.1.6 Hardness

The baseflow event hardness values ranged from 50.7 to 109 mg/L and the storm event hardness values ranged from 17.7 to 98.2 mg/L. The average values for the baseflow and storm events were 79.0 mg/L and 49.1 mg/L, respectively. Hardness, like alkalinity, was lower for the storms than during base events for all sites.

4.3.1.7 Fecal Coliform

The baseflow event fecal coliform (FC) values ranged from 1 to 5,000 CFU/100 ml and the storm event FC values ranged from 5 to 23,000 CFU/100 ml. The geometric mean of all

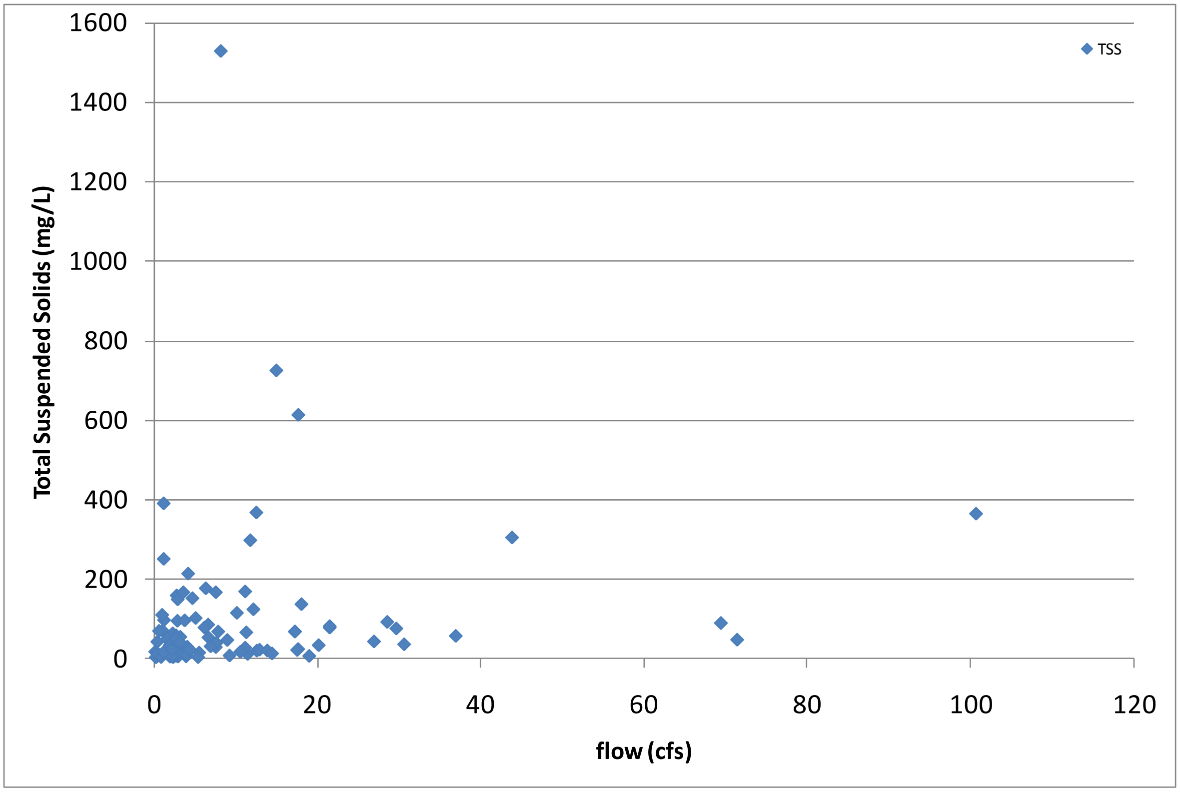


Figure 7. A graph of total suspended solids verses flow data for storm events.

baseflow event samples was 64 CFU/100 ml while the geometric mean of all storm event samples was 765 CFU/100 ml (Table 9). Both of these results are above the Extraordinary Primary Contact Recreation criterion of 50 CFU/100 ml

Table 9 shows the data by event and by site. The geometric mean values of all storm events and for each site during storm events exceeded the Ordinary Primary Contact Recreation of 100 CFU/100ml. The storm event 5 (08/10-11/2009) had the highest geometric mean of all storm events with 3,823 CFU/100 ml and site 6G had the highest geometric mean of all sites with 2,479 CFU/100ml, Table 9. For the baseflow events, Base 3 (06/20/2009) had the highest geometric mean value with 204 CFU/100ml and site 6G also had the highest geometric mean of all sites with 622 CFU/100ml, Table 9.

4.3.1.8 Chlorophyll a — Pheophytin a

The baseflow event values of chlorophyll and pheophytin ranged from 1 to 3 µg/L with the majority of the samples below the laboratory method detection limit. The number of samples below the limit of detection was 34 and 47 of 51 total samples for chlorophyll and pheophytin, respectively. For storm samples, the number of measured concentrations and the range of values increased for both parameters. Chlorophyll storm values ranged from 1 to 27 µg/L with an average value of 4.9 µg/L for 91 of 142 samples. The remaining samples (51) were below the limit of detection. Pheophytin storm values ranged from 1 to 25 µg/L with an average value of 7.7 µg/L for 33 of 142 samples while the remaining samples (109) were below the limit of detection.

Table 9. A summary of fecal coliform data (geometric means) from baseflow and storm sampling events.

Storm events		**Baseflow events**	
768		**65**	
Storm samples			
Event	Geometric mean	Site	Geometric mean
storm 1	**224**	J446_1G	**1,254**
storm 2	**197**	J446_2G	**808**
storm 3	**823**	J446_3G	**323**
storm 4	**688**	J446_4GI	**316**
storm 5	**3,823**	J446_4GO	**351**
storm 6	**1,322**	J446_5GF	**1,680**
storm 7	**385**	J446_6G	**2,479**
		J446_7G	**959**
Base flow samples			
Event	Geometric mean	Site	Geometric mean
base 1	**87**	J446_1G	**135**
base 2	17	J446_2G	47
base 3	**204**	J446_3G	17
		J446_4GI	10
		J446_4GO	15
		J446_5GF	**153**
		J446_6G	**622**
		J446_7G	**205**

Note: Data are geometric mean values over the state criterion are shown in bold.

4.3.1.9 Ammonia

The baseflow event ammonia values ranged from 0.01 to 0.34 mg/L and the storm event ammonia values ranged from 0.01 to 1.28 mg/L. The average values for the base and storm events are 0.04 mg/L and 0.09 mg/L, respectively. No samples exceeded the surface water criterion for ammonia.

4.3.1.10 Nitrate

The parameter analyzed was nitrate plus nitrite (identified as nitrate in the remainder of this report for convenience[3]). The baseflow event nitrate values ranged from 0.02 to 2.5 mg/L and

[3] Measured as nitrate plus nitrite nitrogen. Nitrite nitrogen concentrations are typically very low in well oxygenated ambient waters (Hem 1985) and are typically very near or below the analytical detection limit when specific measurements of nitrite nitrogen are made.

the storm event nitrate values ranged from 0.02 to 2.2 mg/L. The average values for the base and storm events are 1.0 mg/L and 0.6 mg/L, respectively. All sites except the wetland sites (4GI and 4GO) had lower average values during storm events relative to baseflow samples. Nitrate values from sites 4GI and 4GO were consistent for both storm and baseline events within the range of 0.1 to 0.2 mg/L.

4.3.1.11 Total N

The base event total nitrogen (TN) values ranged from 0.3 to 2.7 mg/L and the storm event TN values ranged from 0.3 to 5.2 mg/L. The average values for the base and storm events are 1.2 mg/L and 1.2 mg/L, respectively. The average and minimum values for both type of events were the same while the maximum values of storm event samples were greater than the baseflow events.

4.3.1.12 Orthophosphate Phosphorus

The baseflow event orthophosphate phosphorus (ortho P) values ranged from 0.008 to 0.09 mg/L and the storm event ortho P values ranged from 0.006 to 0.29 mg/L. The average values for the base and storm events are 0.03 mg/L and 0.03 mg/L, respectively. The average and minimum values for both types of events were similar while the maximum values measured during storm events were greater than the concentrations observed in baseflow events sampled.

4.3.1.13 Total Phosphorus

The baseflow event total phosphorus (TP) values ranged from 0.02 to 0.17 mg/L and the storm event TP values ranged from 0.02 to 1.08 mg/L. The average values for the base and storm events are 0.06 mg/L and 0.15 mg/L, respectively. All sites except for site 4GI had greater values during storm events than the base events. Site 4GI had similar values (average, minimum and maximum) for both storm and base events with a range of 0.02 to 0.18 mg/L.

4.3.2 Metals

The trace metals, copper and zinc, were analyzed as total and dissolved fractions.

4.3.2.1 Copper

Dissolved copper concentrations ranged from 0.33 to 2.67 µg/L for baseflow events and ranged from 0.95 to 13.8 µg/L for storm events. The average concentrations for base and storm events were 0.82 and 2.9 µg/L, respectively.

Total copper concentrations ranged from 0.39 to 2.57 µg/L for base events and ranged from 1.0 to 89.7 µg/L for storm events. The average concentrations for base and storm events were 1.1 and 8.2 µg/L, respectively.

No base event samples exceeded either the acute or chronic criteria (Table 10). Two of the 142 storm samples exceeded the acute criterion. These samples were from Storm 5 (08/20/2009) and occurred at sites 4GI and 5G. The chronic criterion was exceeded in 12 of the storm samples. Storm 5 and Site 4GI had the most samples over the standard - 9 and 6 samples, respectively. Storm 3 had two samples with values over the chronic criterion while Storm 7 only had one. The

other sites with sample values over the chronic criterion included 5GF and 6G with 2 samples each while sites 1G and 2G only had one sample.

4.3.2.2 Zinc

Dissolved zinc concentrations ranged from 1.4 to 31.7 µg/L for baseflow events and ranged from 2.3 to 292 µg/L for storm events. The average concentrations for base and storm events were 5.0 and 16.1 µg/L, respectively.

Total zinc concentrations ranged from 1.5 to 30.8 µg/L for base events and ranged from 4.2 to 341 µg/L for storm events. The average concentrations for base and storm events were 5.9 and 40.9 µg/L, respectively.

Dissolved zinc, like copper, did not have any baseflow event samples that exceeded either the acute or chronic criteria (Table 10). Six of the 142 storm samples exceeded the acute criterion . Each sample that exceeded the acute threshold also exceeded the chronic criterion. No other chronic standard exceedances occurred. The occurrence of these samples is different than those of copper. Storm 2 and site 1G had the most samples over the chronic criterion with 3 samples each. Storm 5 had two samples with values over the chronic criterion while Storm 7 only had one. The other sites with sample values over the chronic criterion included 4GI, 4GO and 5GF with one sample each.

Table 10. A summary of trace metal data collected during baseflow and storm events compared to the state acute and chronic criteria*. Number of samples over the criteria are shown in bold.

Dissolved Copper			**Dissolved Zinc**	
		Base		
Acute	**Chronic**		**Acute**	**Chronic**
0	0	Event/Site	0	0
		Storm		
Acute	**Chronic**		**Acute**	**Chronic**
2	**12**	Event/Site	**6**	**6**
0	0	Storm 1	0	0
0	0	Storm 2	**3**	3
0	**2**	Storm 3	0	0
0	0	Storm 4	0	0
2	**9**	Storm 5	**2**	**2**
0	0	Storm 6	0	0
0	**1**	Storm 7	**1**	**1**
0	**1**	J446_1G	**3**	**3**
0	**1**	J446_2G	0	0
0	0	J446_3G	0	0
1	**6**	J446_4GI	**1**	**1**
0	0	J446_4GO	**1**	**1**
1	**2**	J446_5GF	**1**	**1**
0	**2**	J446_6G	0	0
0	0	J446_7G	0	0

* = These criteria are formula based using the hardness of the sample. see WAC 173-201A for formulas. Average values can be found in Table 1.

5.0. SUMMARY

As part of the Juanita Creek Stormwater Retrofit planning project, a water quality monitoring study was designed to assess current baseflow and stormwater quality in Juanita Creek and support the development of a watershed hydrologic and water quality model. Water quality samples were collected during seven storm events and three base flow events over a period of 14 months – October 2008 to November 2009 from eight stations. Continuous (15-minute) monitoring of stream flow and water temperature was also conducted at six sites from October 2008 to December 2009.

The dissolved oxygen concentrations were below the state criteria of 9.5 mg/L during both base and storm events (Table 4 and Table 8). The wetland sites (4GI and 4GO) had the lowest DO concentrations during baseflow events. Low DO was observed at Station 4GI relative to the other locations sampled could be a result of very flat highly vegetated channels connecting between the wetland and Totem Lake upstream?. These types of water courses can have high levels of benthic oxygen demand which can be exacerbated by slow moving waters. One other sample collected at Station 3G was below the criterion (Table 5). The storm DO data had more sites with data below the state criterion (Table 8). All of Storm 6 (10/16-17/2009) event had DO data below the criterion with data ranging from 0.7 to 3.8 mg/L (Figure 5). It is unknown at this time if the data collected during Storm 6 was an equipment problem or a flushing occurrence that happens during the first fall storm. Other storm events had six of eight sites with low DO data. They include sites 3G, 4GI, 4GO, 5GF, 6G, 7G.

The water temperature data collected as part of this study were a combination of grab samples and continuously recording sensors. The temperature criteria are based on the 7-day average of daily maximum temperatures. All continuous temperature sites had temperatures that exceeded the criterion intended to protect salmonid rearing and migration (Table 6 and Figure 4). Site 27h (6G) had the highest temperatures and the most days over the standards. The grab (discreet) temperature data collected at all sites support the data collected at continuous sites.

The pH data for all baseflow and storm events were within the state criteria of greater than 6.5 and less than 8.5 except for 5 storm samples (Table 4, Table 7 and Table 8). Four samples were below 6.5 at sites 5GF and 4GI. The result of 13.9 from site 5GF during Storm 6 appears to be anomalous when compared to the total dataset of storm samples (142) (Table 7) which ranged from 6.3 to 8.0.

Fecal coliform were detected in all samples from baseflow and storm events. The geometric mean of all baseflow event samples was 64 CFU/100 ml while the geometric mean of all storm event samples was 765 CFU/100 ml, or approximately an order of magnitude greater during storm events (Table 9 and Appendix 1). Both of these results are above the Extraordinary Primary Contact Recreation criterion of 50 CFU/100 ml (Table 1 and Table 9). Storm 5 (08/10-11/2009) had the highest geometric mean of all storm events with 3,823 CFU/100 ml and site 6G had the highest geometric mean of all sites with 2,479 CFU/100 ml (Table 9). For the baseflow events, Base 3 (06/20/2009) had the highest geometric mean value with 204 CFU/100 ml and site 6G also had the highest geometric mean of all sites with 622 CFU/100 ml (Table 9).

For dissolved metals (copper and zinc), no baseflow event samples exceeded either the acute or chronic criteria (Table 4 and Table 10). For dissolved copper, two of the 142 storm samples exceeded the acute criterion. These samples were from Storm 5 (08/20/2009) and occurred at

sites 4GI and 5G. The chronic criterion was exceeded in 12 samples, with Storm 5 and Site 4GI having the most samples over this standard with 9 and 6 samples, respectively. Elevated copper levels measured at 4GI is consistent with the drainage area that includes major arterials and Interstate-405- copper from automobile brake pads is a potential source of copper in urban streams. Storm 3 had two samples with values over the chronic criterion while Storm 7 had one. The other sites with sample values over the chronic criterion included 5GF and 6G with two samples each while sites 1G and 2G only had one sample, Table 10.

For dissolved zinc, six of the 142 storm samples exceeded the acute criterion (Table 10). Each sample that exceeded the acute criterion also exceeded the chronic criterion. The occurrence of these standard exceedances is different than those for dissolved copper. Storm 2 and site 1G had the most samples over the criteria with 3 samples each (Table 10). Storm 5 had two samples with values over the chronic criterion while Storm 7 only had one. The other sites with sample values over the chronic criterion included 4GI, 4GO and 5GF with one sample each.

In general, the data collected were of sufficient quality to support the development of a watershed hydrologic and water quality model that can be used to evaluate the effectiveness of various stormwater retrofit strategies in improving water quality and quantity conditions in the Juanita Creek watershed.

6.0. REFERENCES

Ecology, 2011. Waters Requiring Supplemental Spawning and Incubation Protection For Salmonid Species. Washington Department of Ecology Publication 06-10-038, published 2006 – revised 2011.

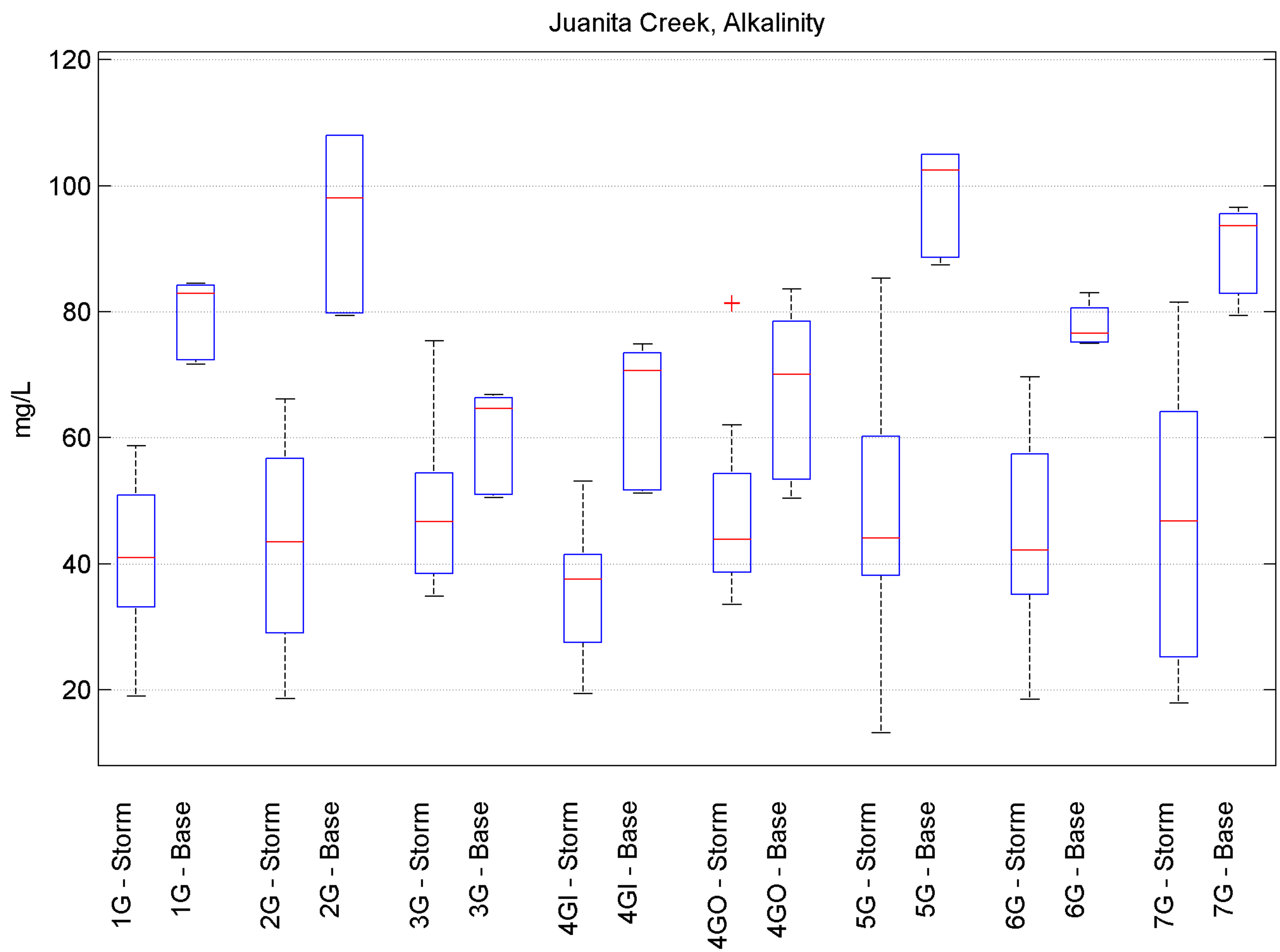
Hem, J.D. 1985. Study and interpretation of chemical characteristics of natural water. U.S. Geological Survey Water-Supply Paper 2254. http://pubs.usgs.gov/wsp/wsp2254/

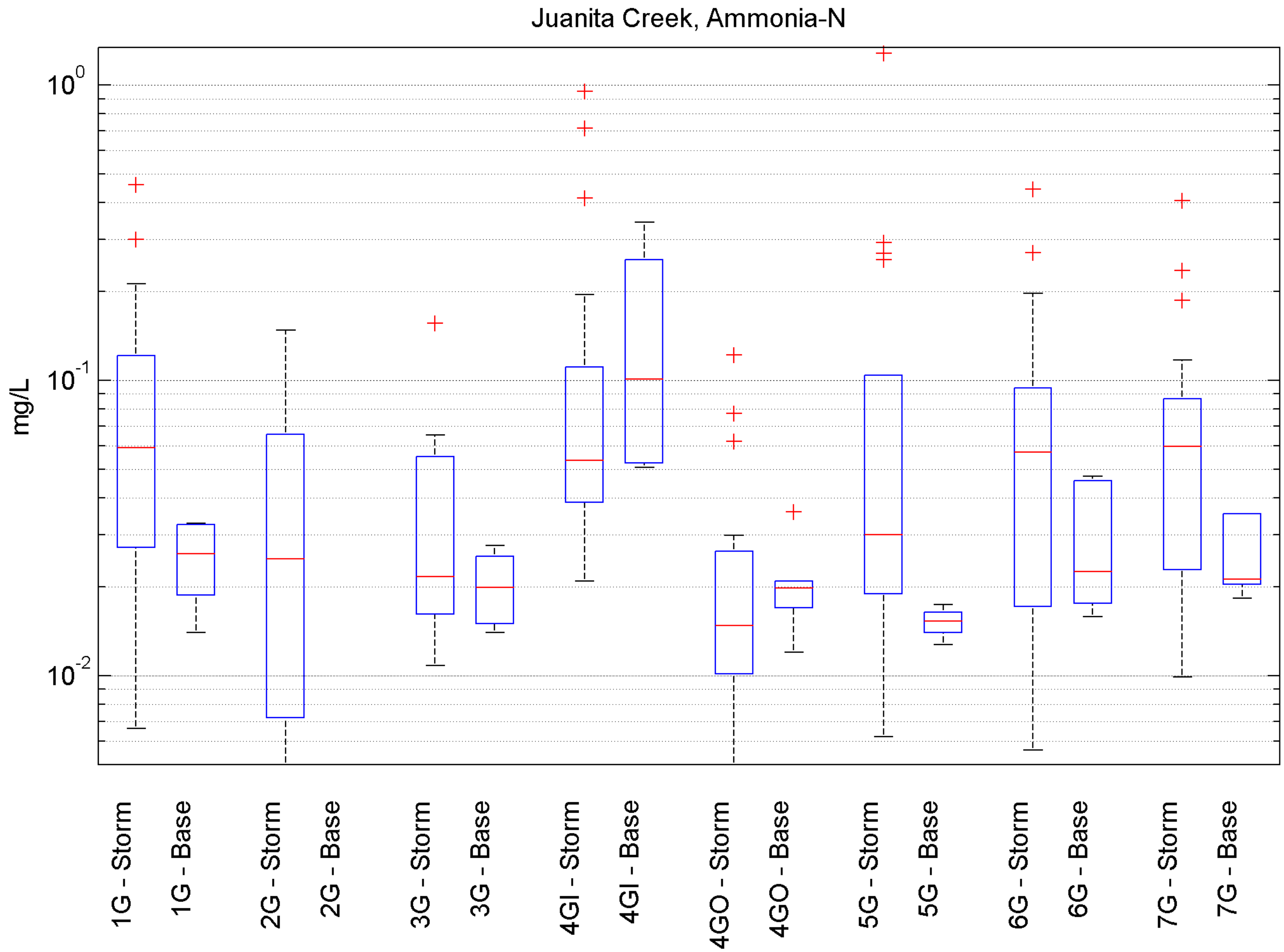
King County, 2008. Sampling and Analysis Plan and Quality Assurance Project Plan – A one year monitoring plan supporting Juanita Creek basin stormwater retrofitting analysis project. Water and Land Resources Division, Seattle, Washington.

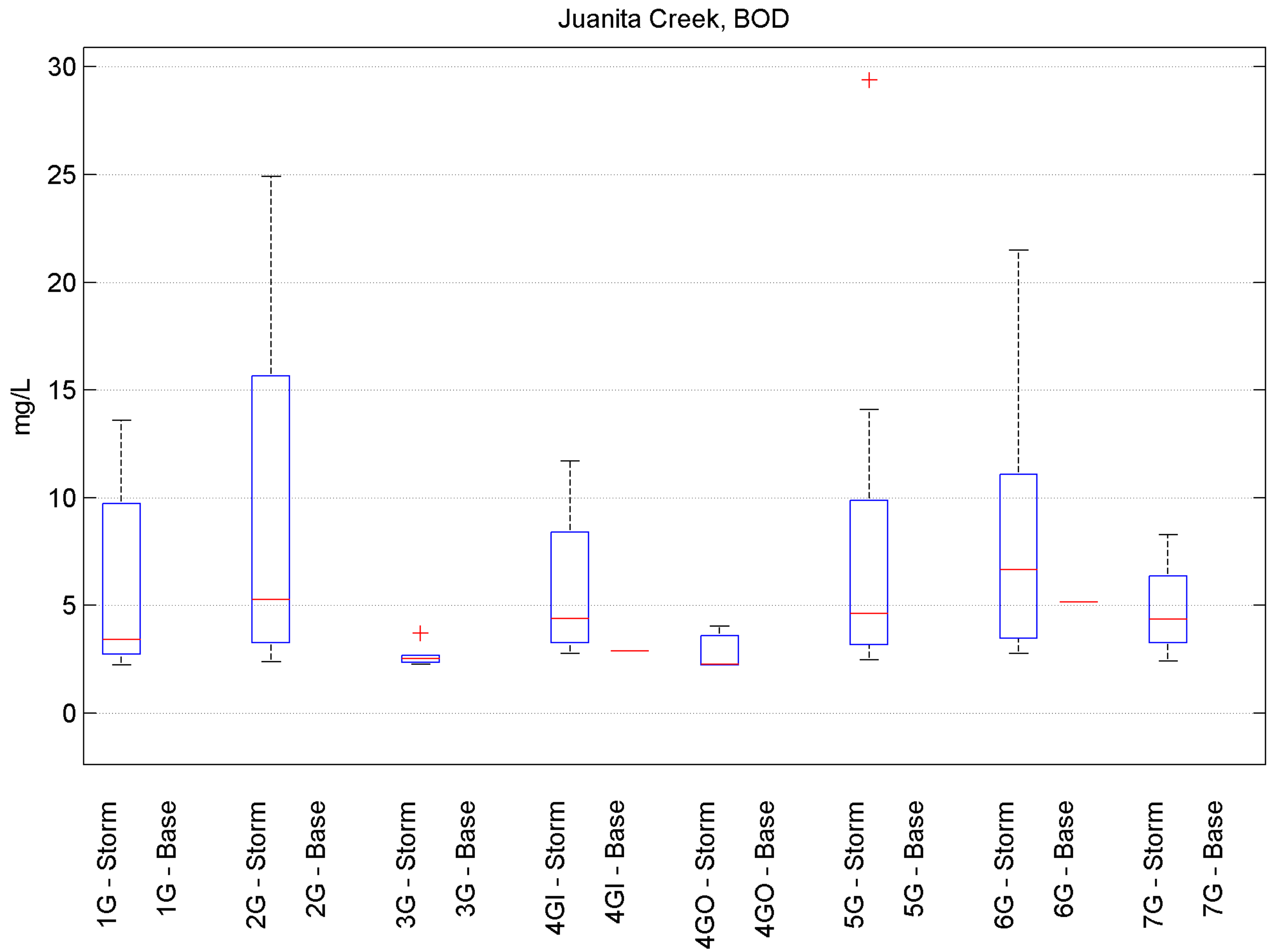
United States Geological Survey (USGS), 2011. Water Hardness and Alkalinity, USGS website for the guidelines of hardness classifications: http://water.usgs.gov/owq/hardness-alkalinity.html

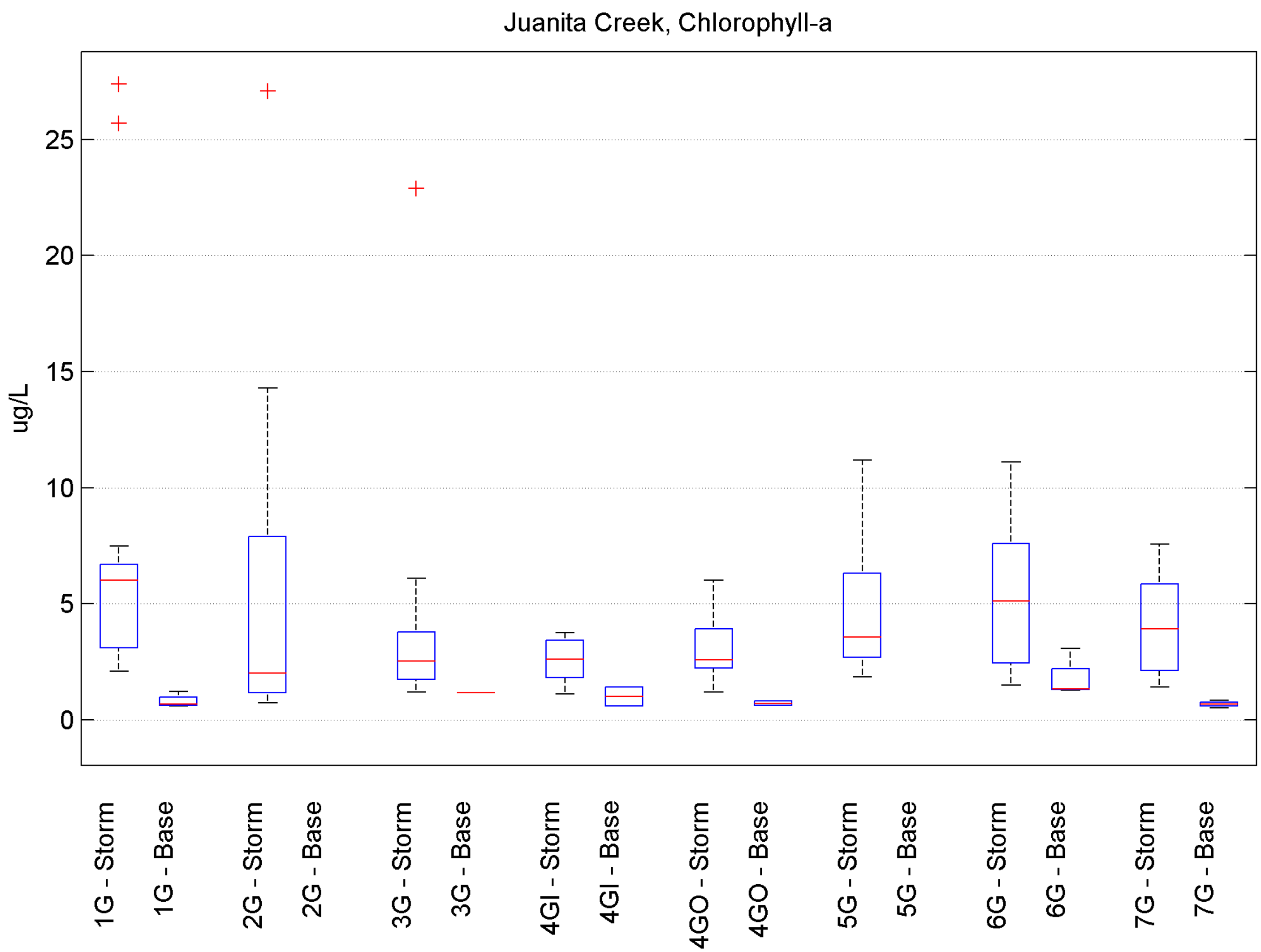
Appendix 1

Boxplots of water quality parameters (base and storm)

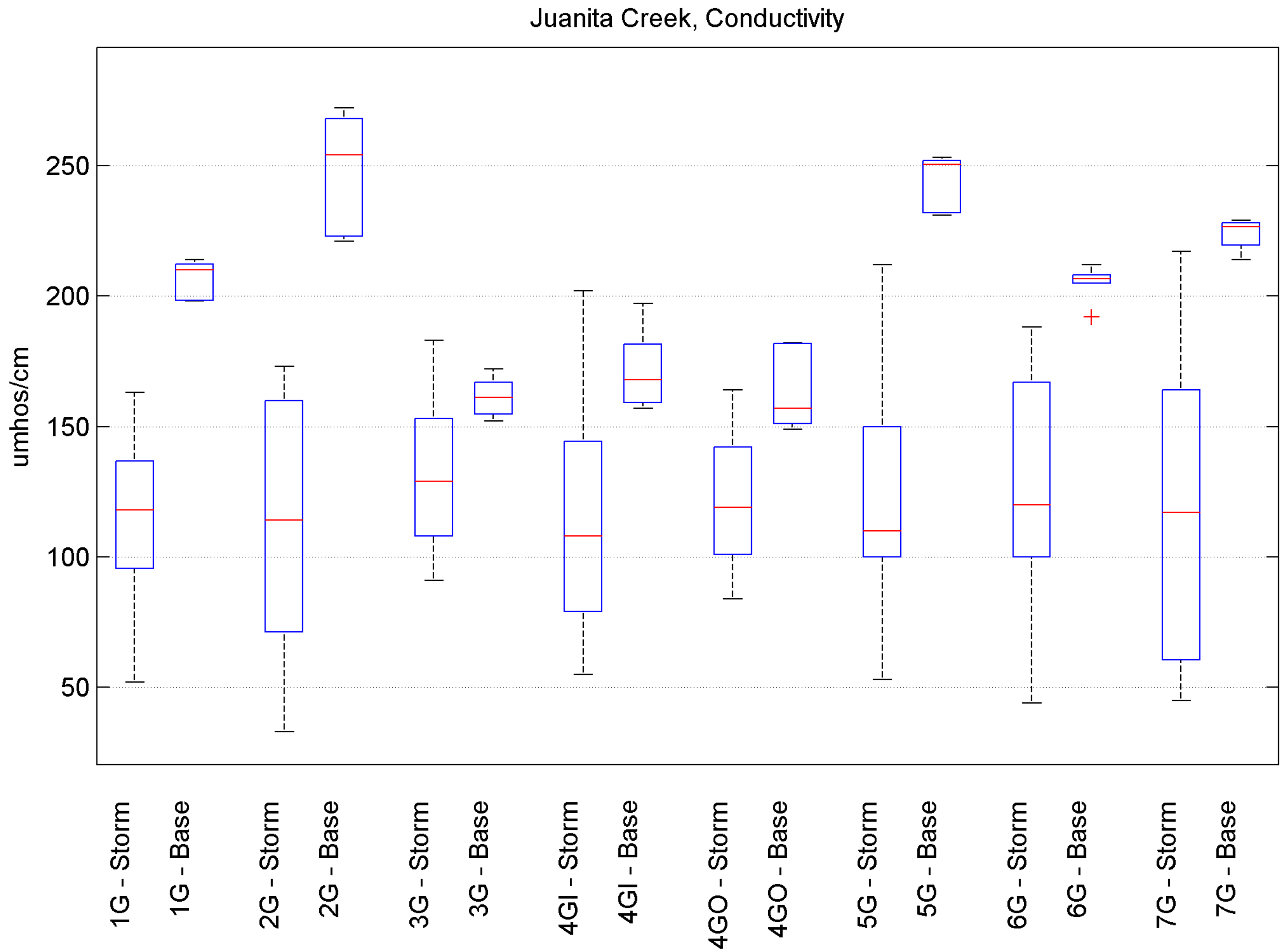

Juanita Creek, Alkalinity
mg/L
120
100
80
60
40
20
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base

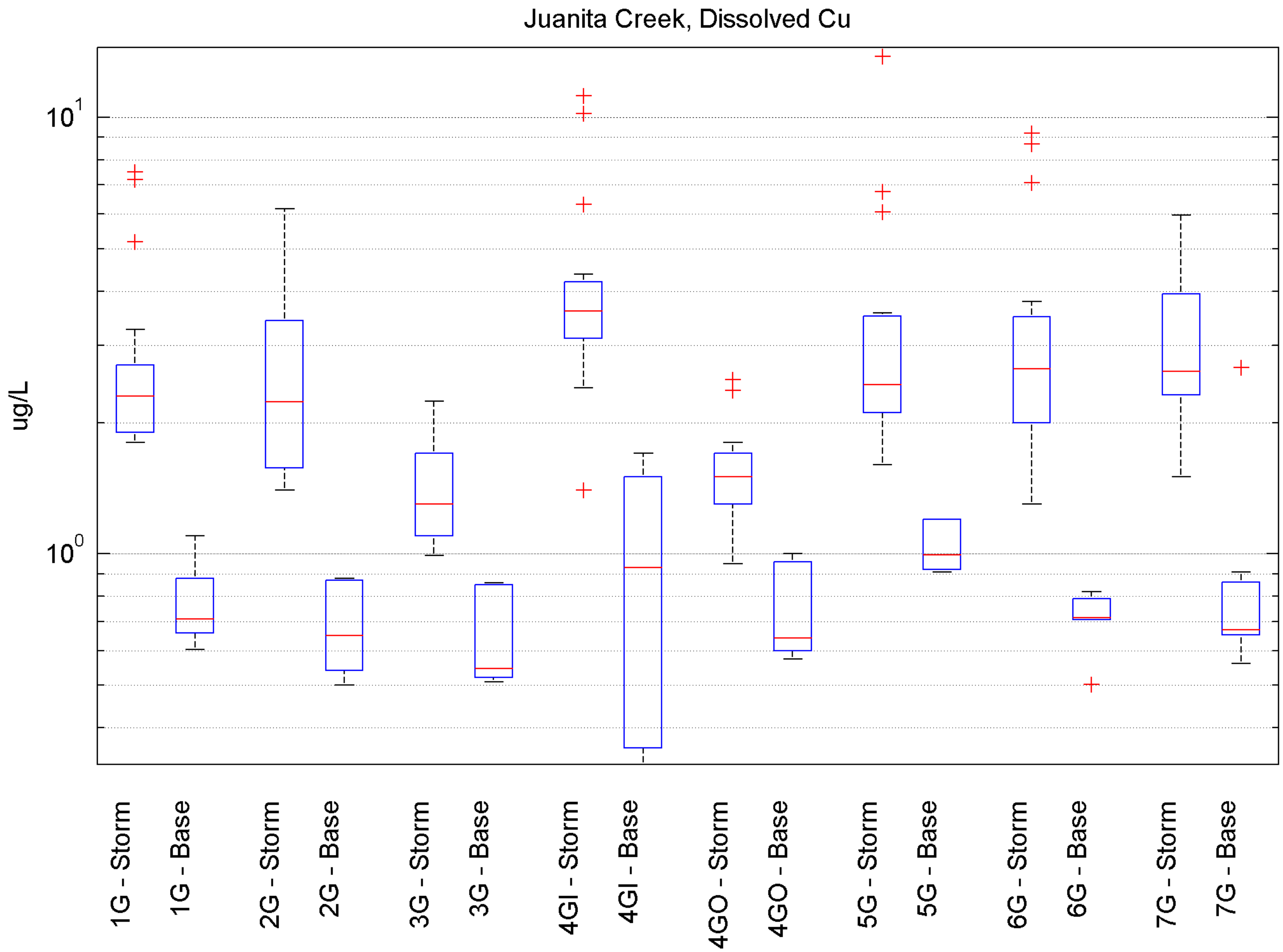

Juanita Creek, Ammonia-N
mg/L
10^0
10^{-1}
10^{-2}
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base


Juanita Creek, BOD
mg/L
30
25
20
15
10
5
0
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base

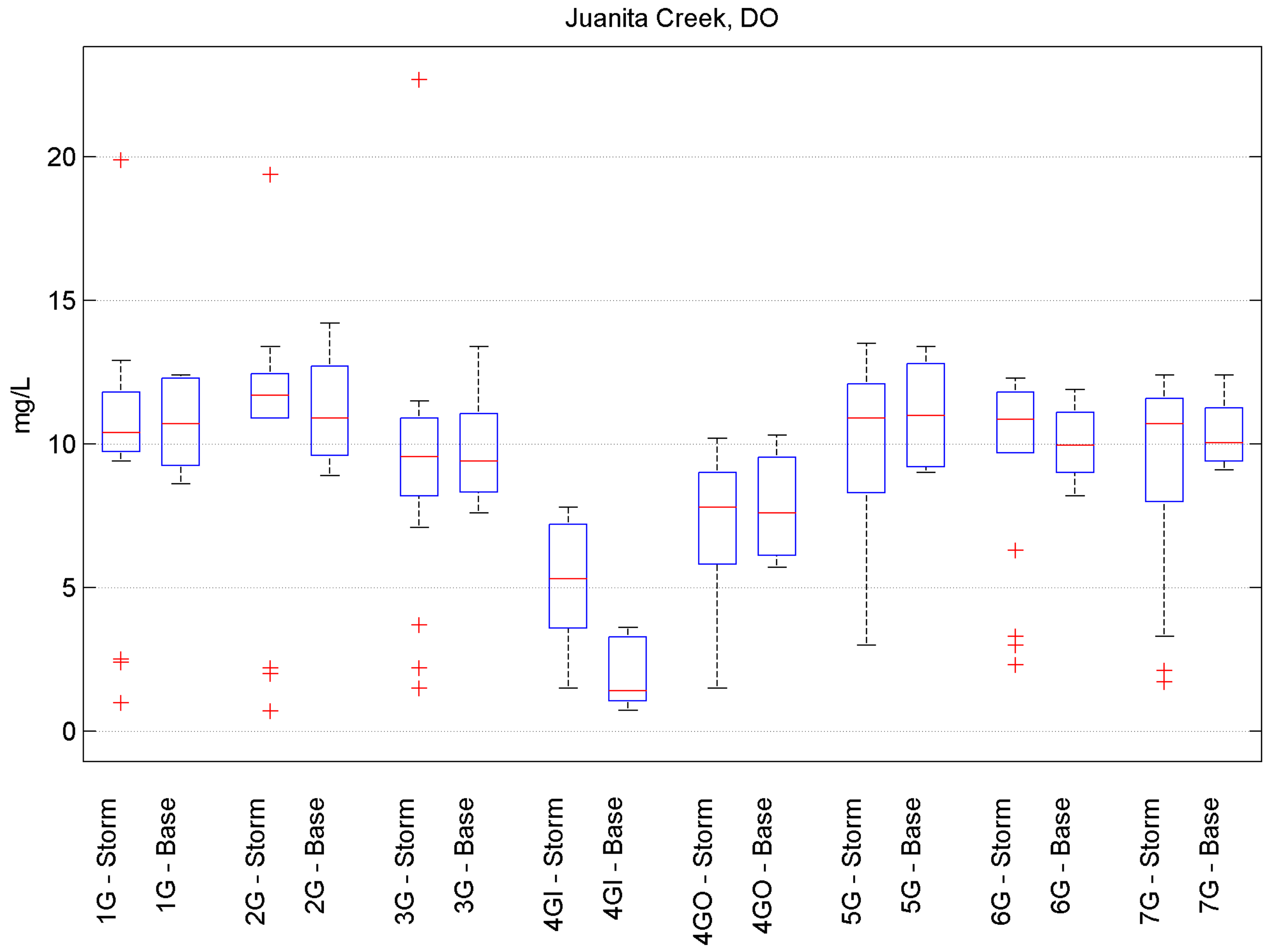


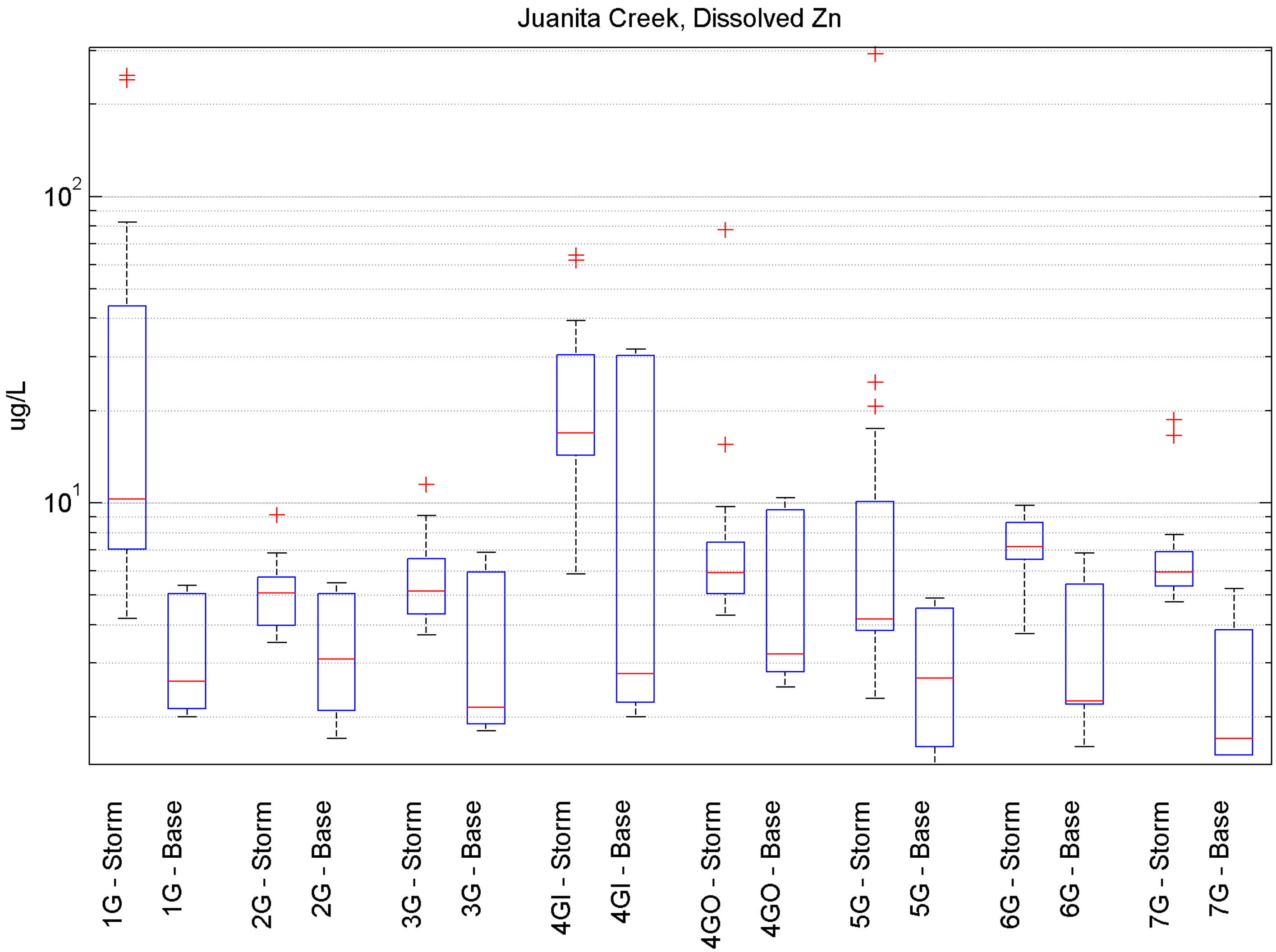
Juanita Creek, Chlorophyll-a
ug/L
0
5
10
15
20
25
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base

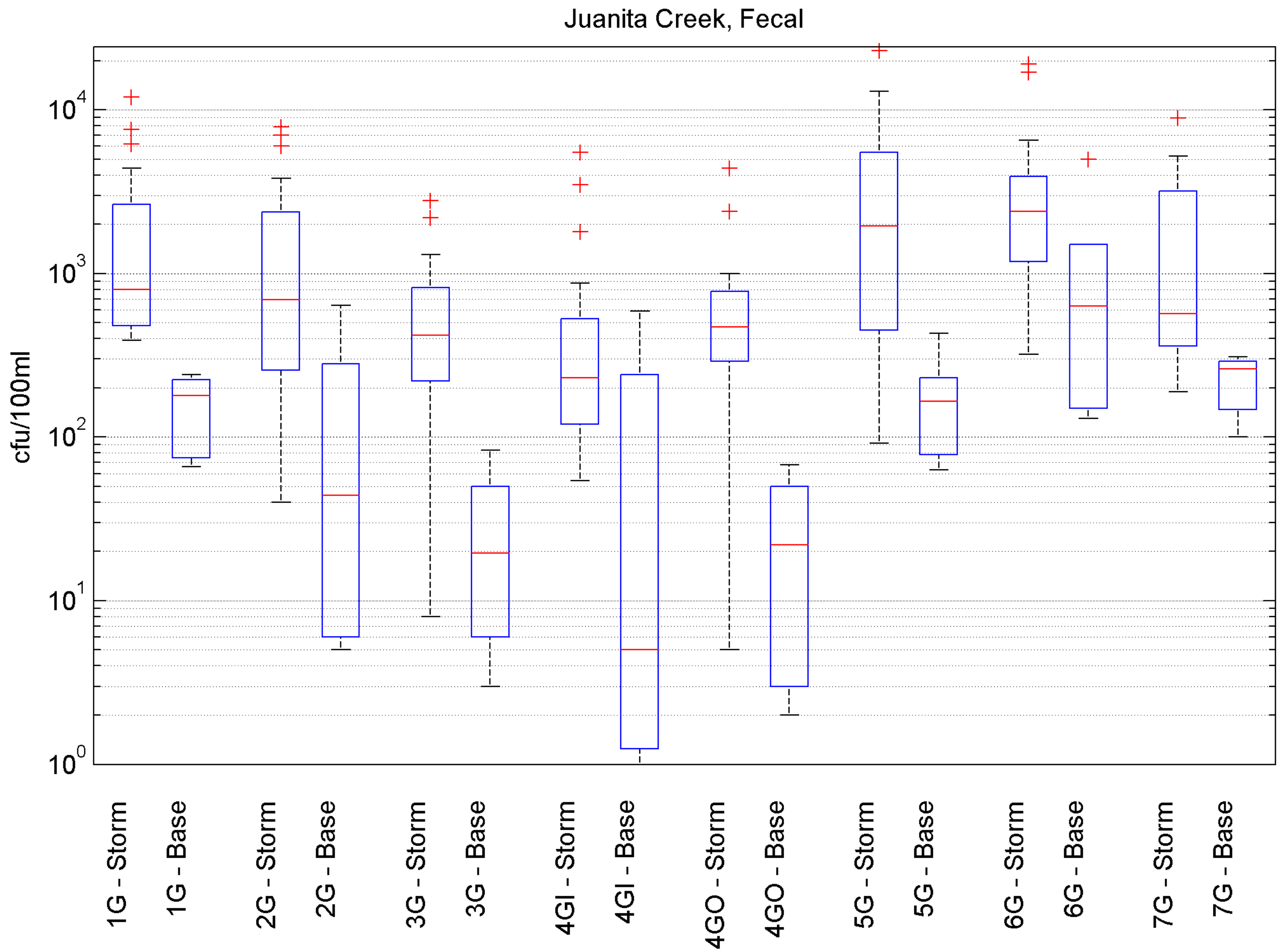

Juanita Creek, Conductivity
umhos/cm
250
200
150
100
50
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base



Juanita Creek, Dissolved Cu
ug/L
10^1
10^0
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base


Juanita Creek, DO
mg/L
0
5
10
15
20
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base


Juanita Creek, Dissolved Zn
ug/L
10^2
10^1
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base



Juanita Creek, Fecal
cfu/100ml
10^4
10^3
10^2
10^1
10^0
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
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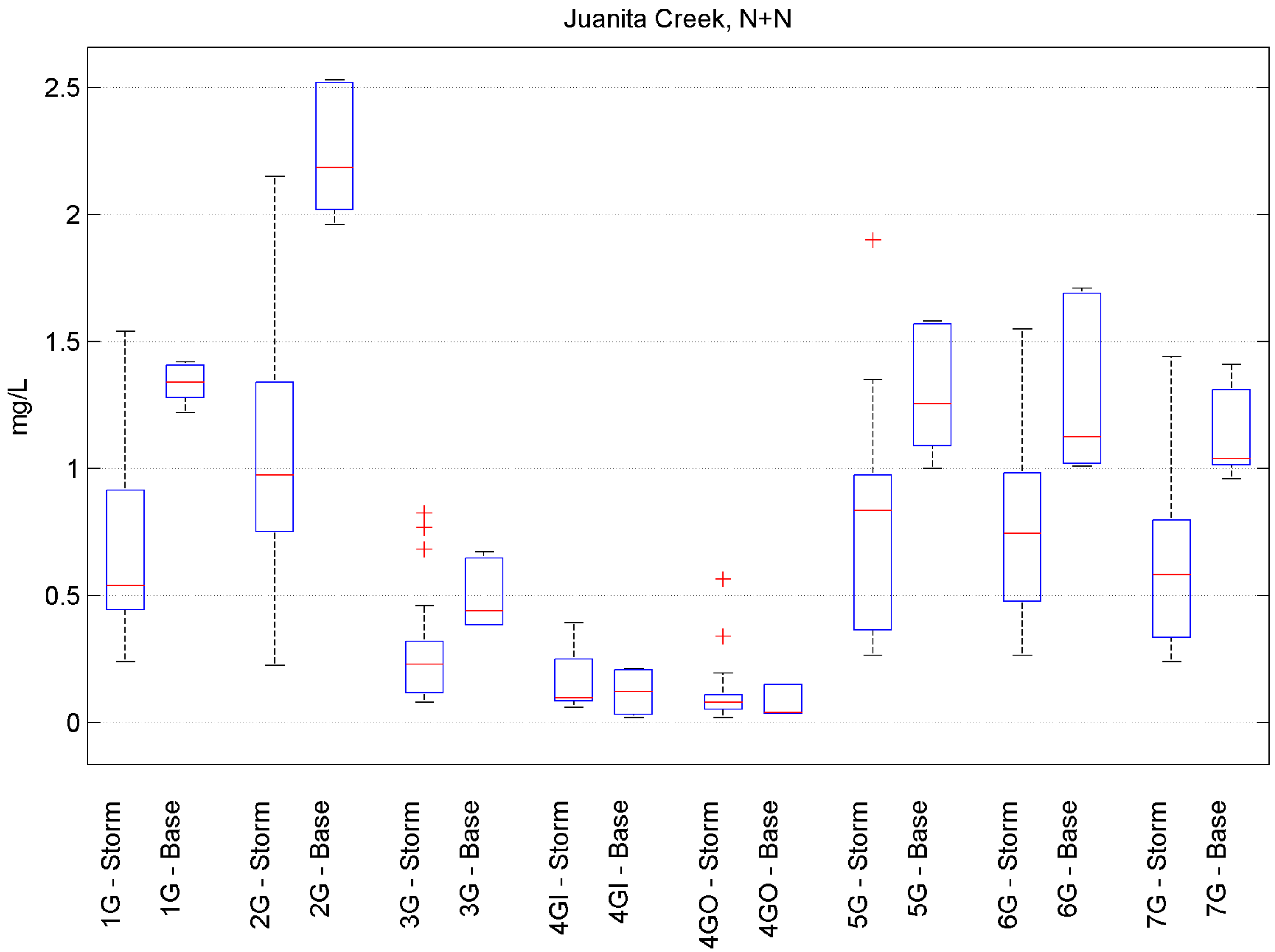
6G - Base
7G - Storm
7G - Base


Juanita Creek, N+N
mg/L
2.5
2
1.5
1
0.5
0
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
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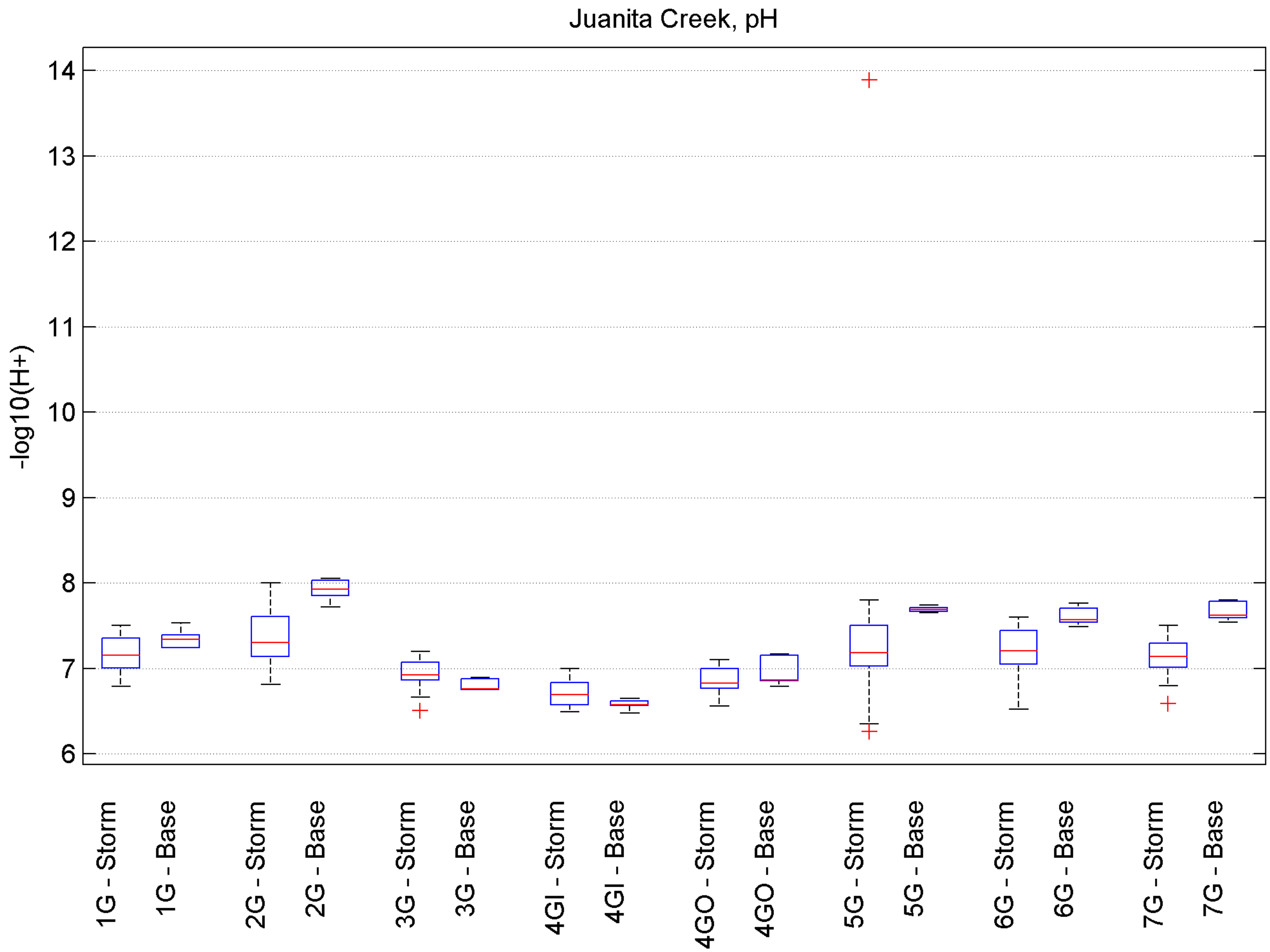
6G - Storm
6G - Base
7G - Storm
7G - Base


Juanita Creek, pH
-log10(H+)
14
13
12
11
10
9
8
7
6
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
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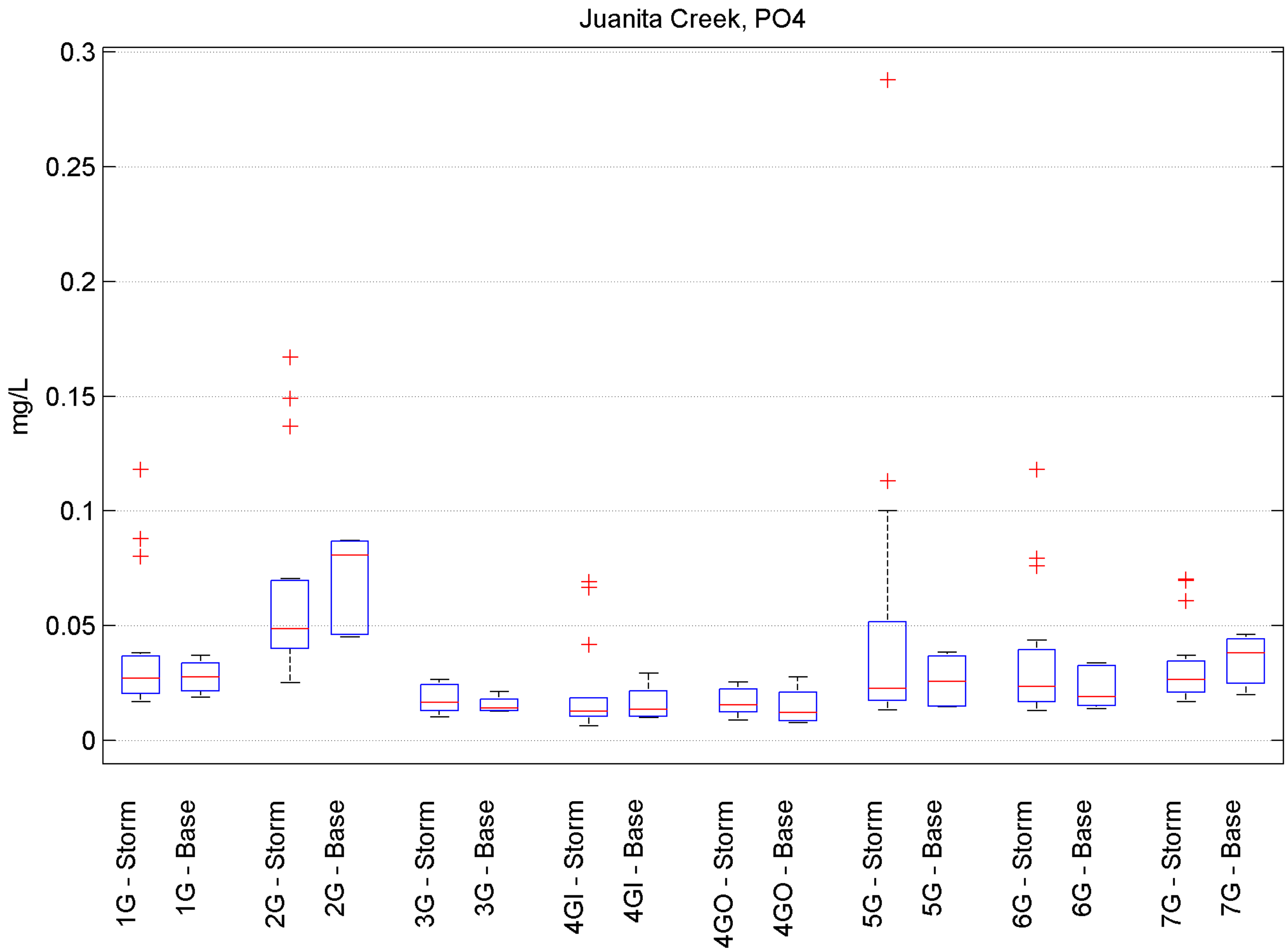
6G - Base
7G - Storm
7G - Base


Juanita Creek, PO4
mg/L
0.3
0.25
0.2
0.15
0.1
0.05
0
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
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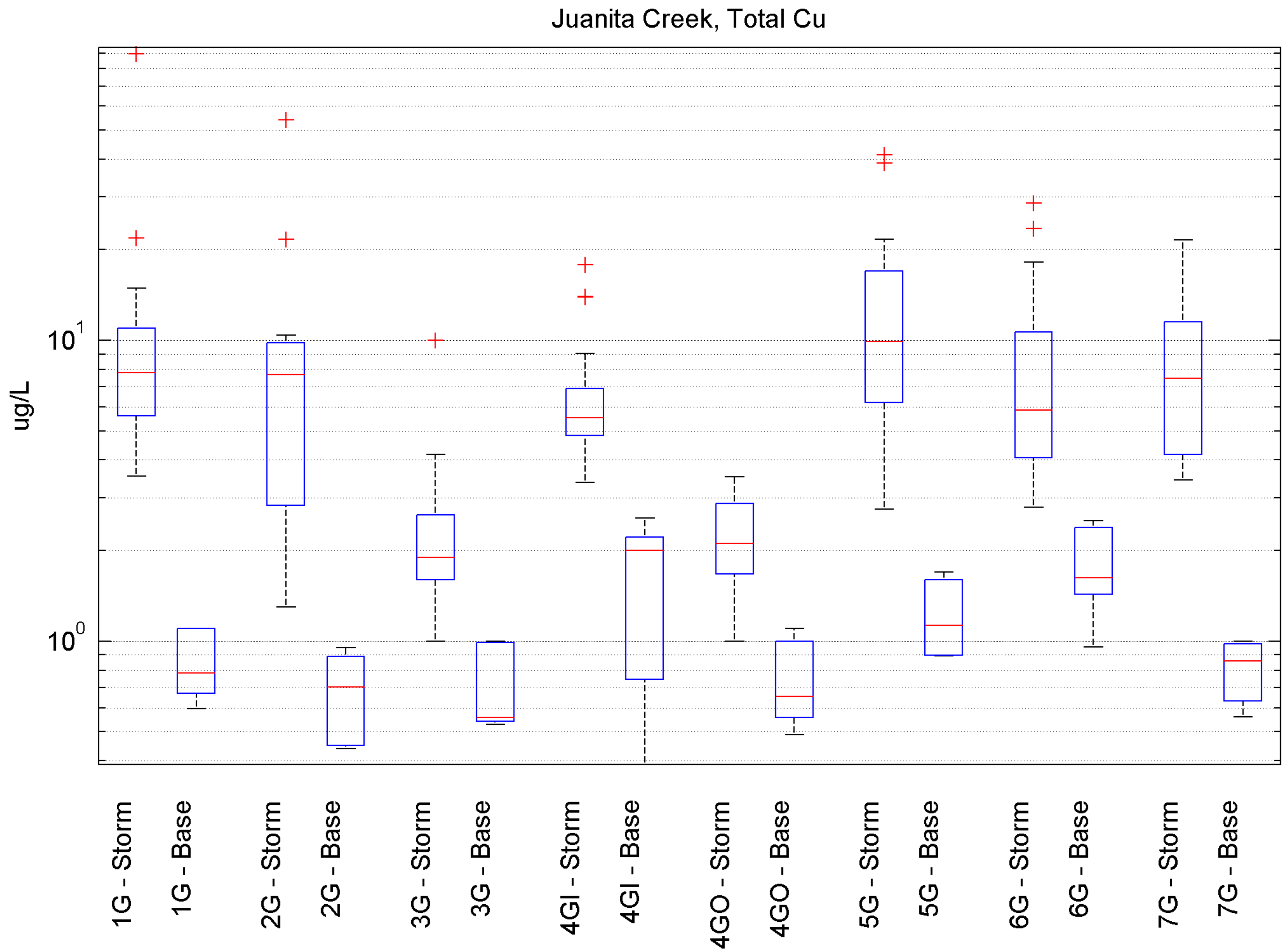
7G - Storm
7G - Base


Juanita Creek, Total Cu
ug/L
10^1
10^0
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
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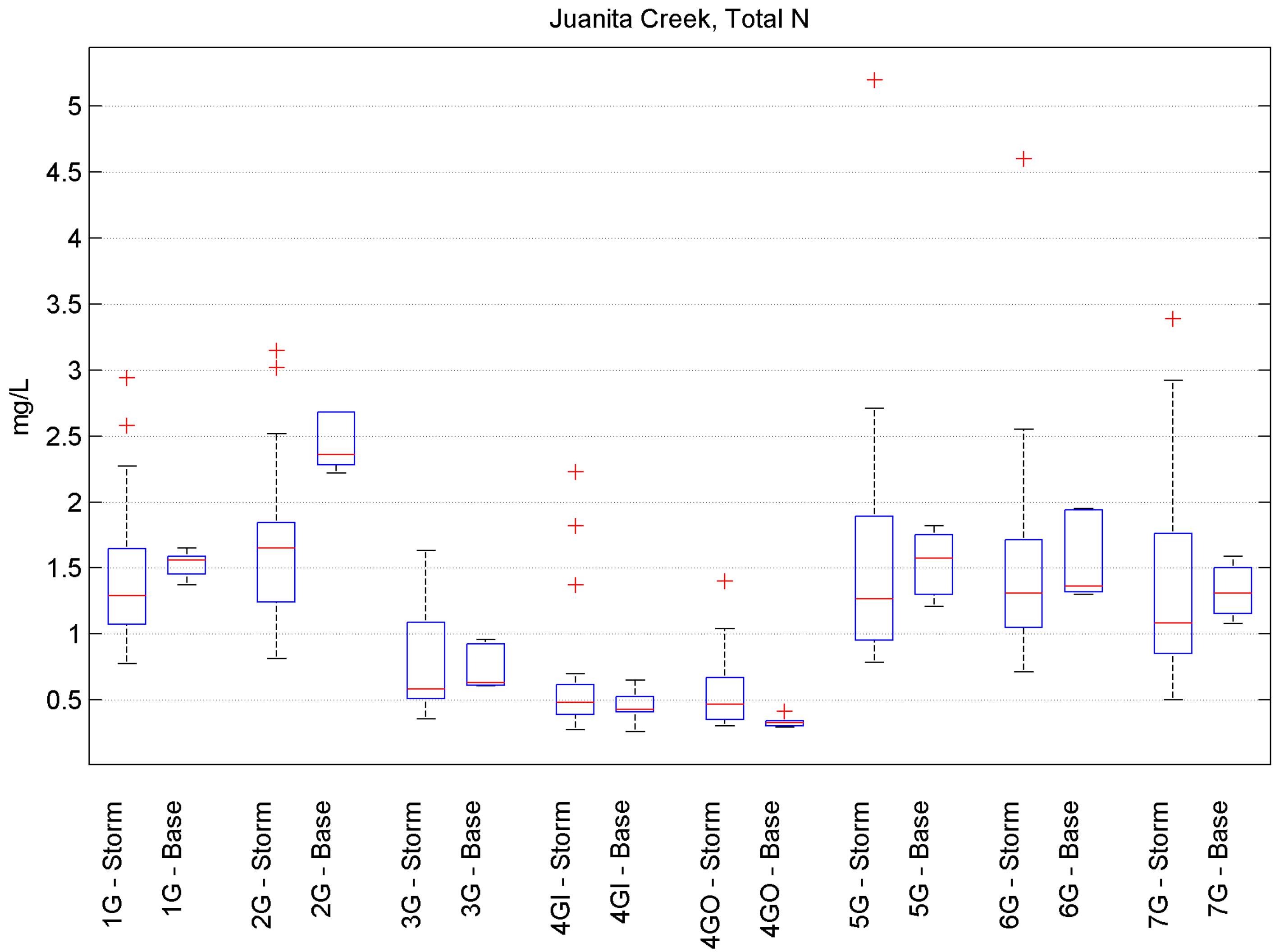
7G - Storm
7G - Base


Juanita Creek, Total N
mg/L
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
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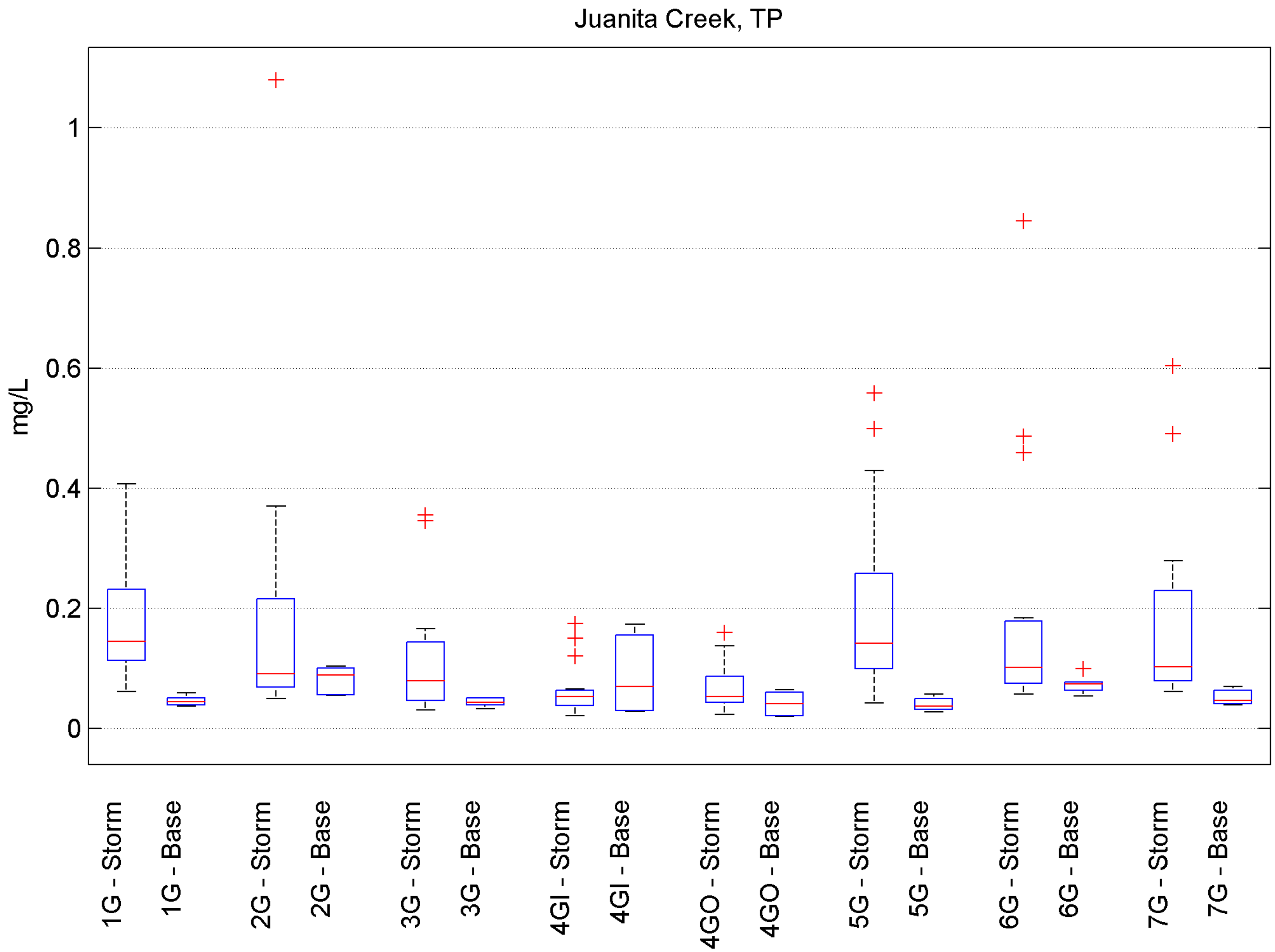
7G - Storm
7G - Base


Juanita Creek, TP
mg/L
0
0.2
0.4
0.6
0.8
1
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
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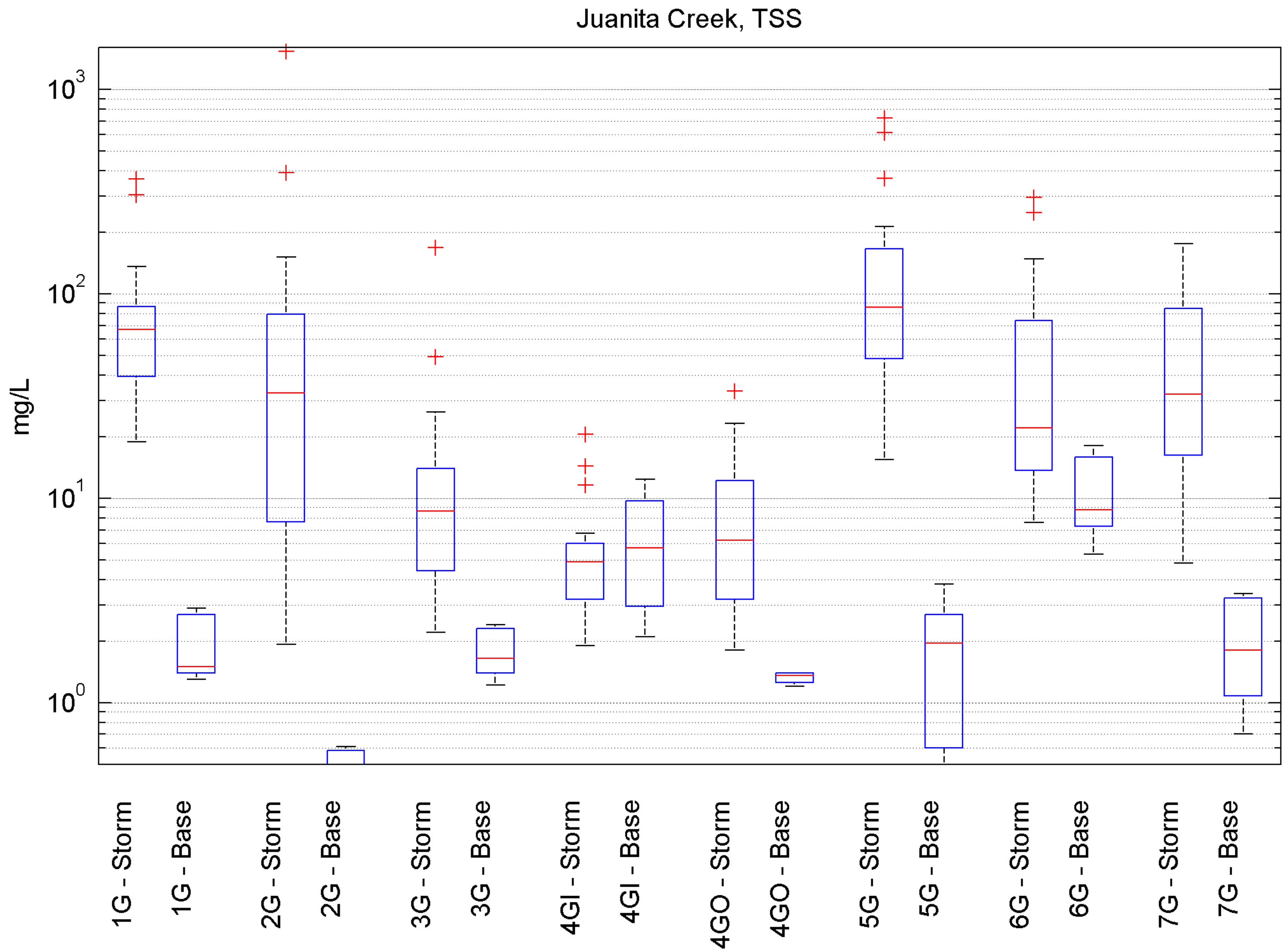
6G - Base
7G - Storm
7G - Base



Juanita Creek, TSS
mg/L
10^3
10^2
10^1
10^0
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
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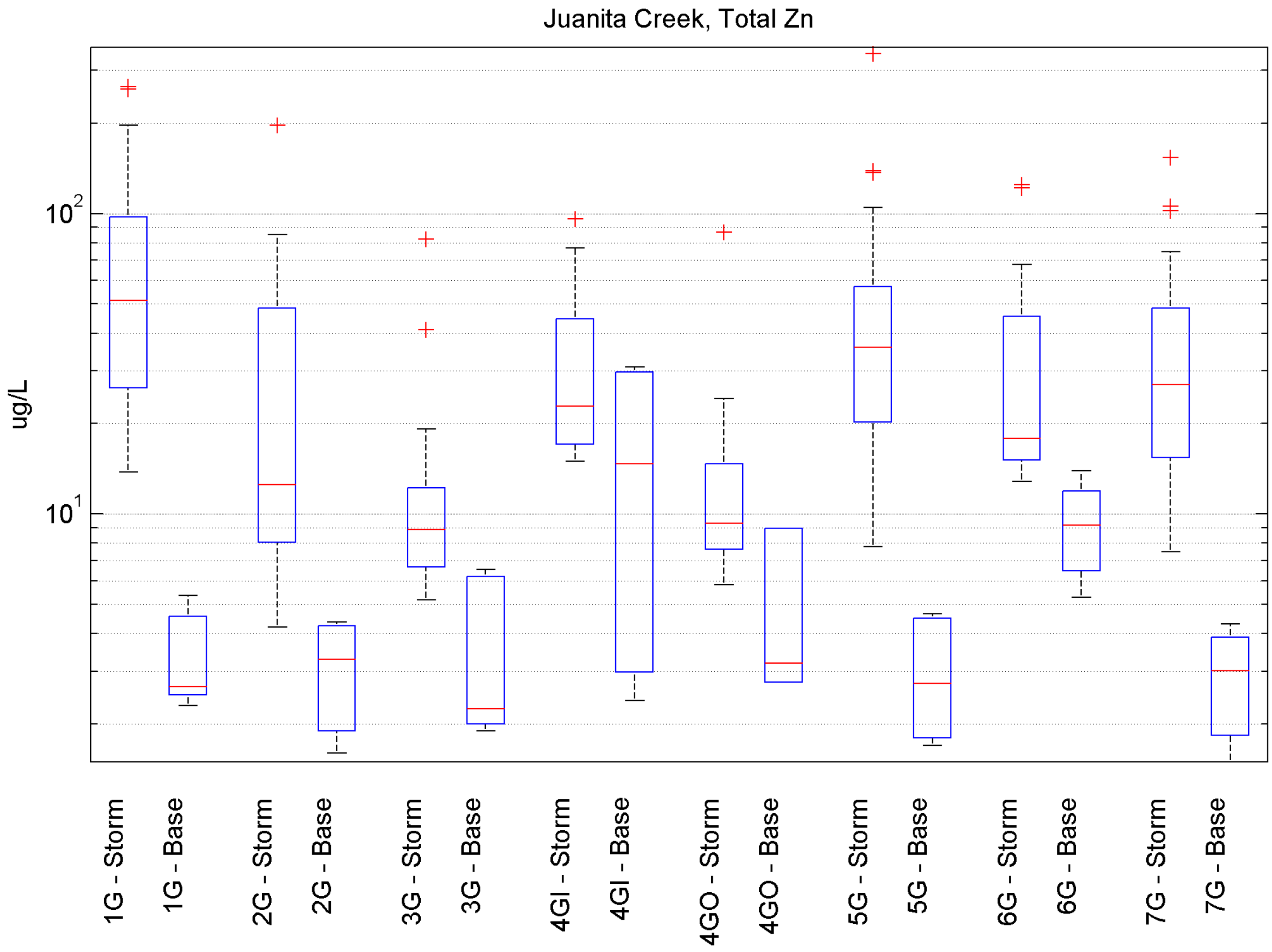
7G - Storm
7G - Base



Juanita Creek, Total Zn
ug/L
10^2
10^1
1G - Storm
1G - Base
2G - Storm
2G - Base
3G - Storm
3G - Base
4GI - Storm
4GI - Base
4GO - Storm
4GO - Base
5G - Storm
5G - Base
6G - Storm
6G - Base
7G - Storm
7G - Base

APPENDIX B. GEOMORPHOLOGIC REPORTS

Report: Juanita Creek Basin Geomorphic Analysis – DRAFT. February 2010.

Memorandum: Juanita Creek—Relationship of Roughness, Gravel Size, and Disturbance Flows. September 2011 (Revised December 2011)

Memorandum: Addendum to Draft Incipient Motion Analyses for Priority Reaches in Juanita Creek. June 2010

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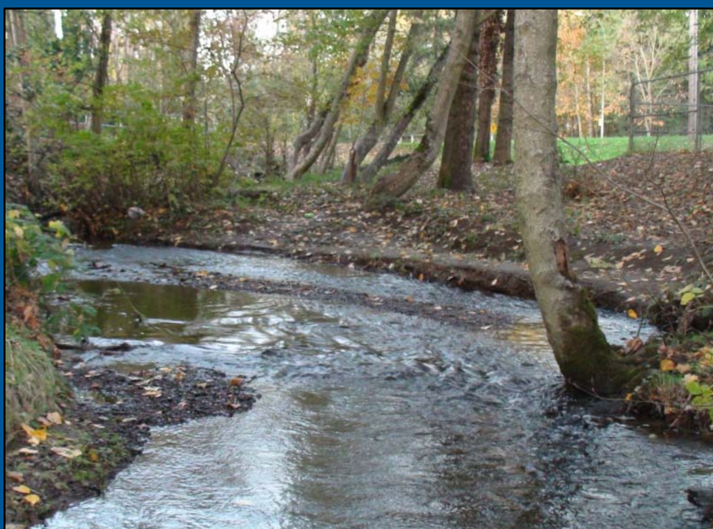
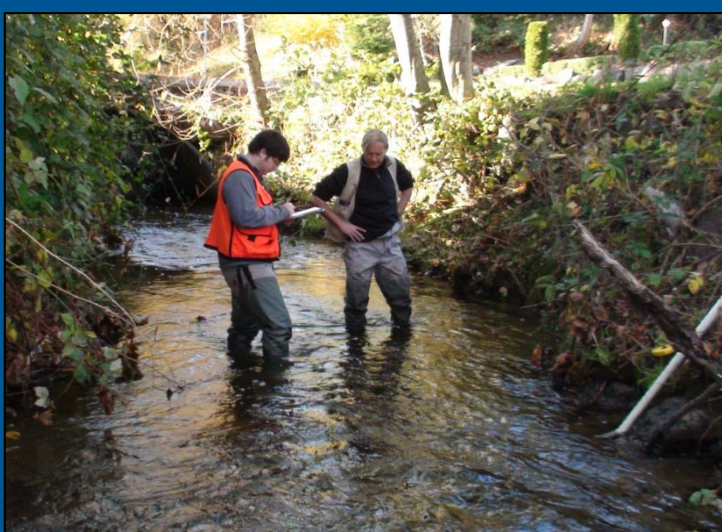
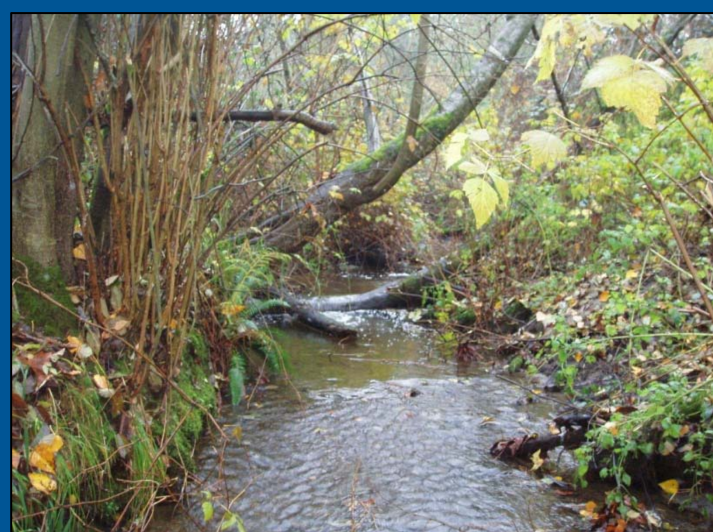
Juanita Creek Basin Geomorphic Analysis - DRAFT

Prepared for:
King County
Department of Natural Resources



Prepared by:

northwest hydraulic consultants

Stillwater Sciences

February 2010

Table of Contents

1. Introduction

As part of a larger effort to characterize conditions in the Juanita Creek watershed and work toward restoration of beneficial uses, Northwest Hydraulic Consultants, Inc. (NHC) was contracted by King County (County) to perform hydraulic and geomorphic analysis on Juanita Creek and several of its major tributaries. The scope for this work included field survey and development of a HEC-RAS hydraulic model for the Juanita Creek mainstem and portions of three tributaries, evaluation of incipient motion thresholds for five "priority" reaches near King County flow gage sites, and basinwide geomorphic data collection and analysis. Stillwater Sciences (Stillwater) served as a subconsultant to NHC for geomorphology and biological issues. This report focuses on analysis and interpretation of the data collected in our geomorphic survey (documented previously in NHC, 2009; NHC, 2010a).

Under the State Water Quality Standards, designated beneficial uses of Juanita Creek include "core rearing" for native aquatic biota. Those stream reaches identified as core rearing are for the protection of spawning, rearing, and migration of salmon and trout, and other associated aquatic life. Assessment of geomorphic conditions critical to these ecological attributes was thus a focus of this characterization.

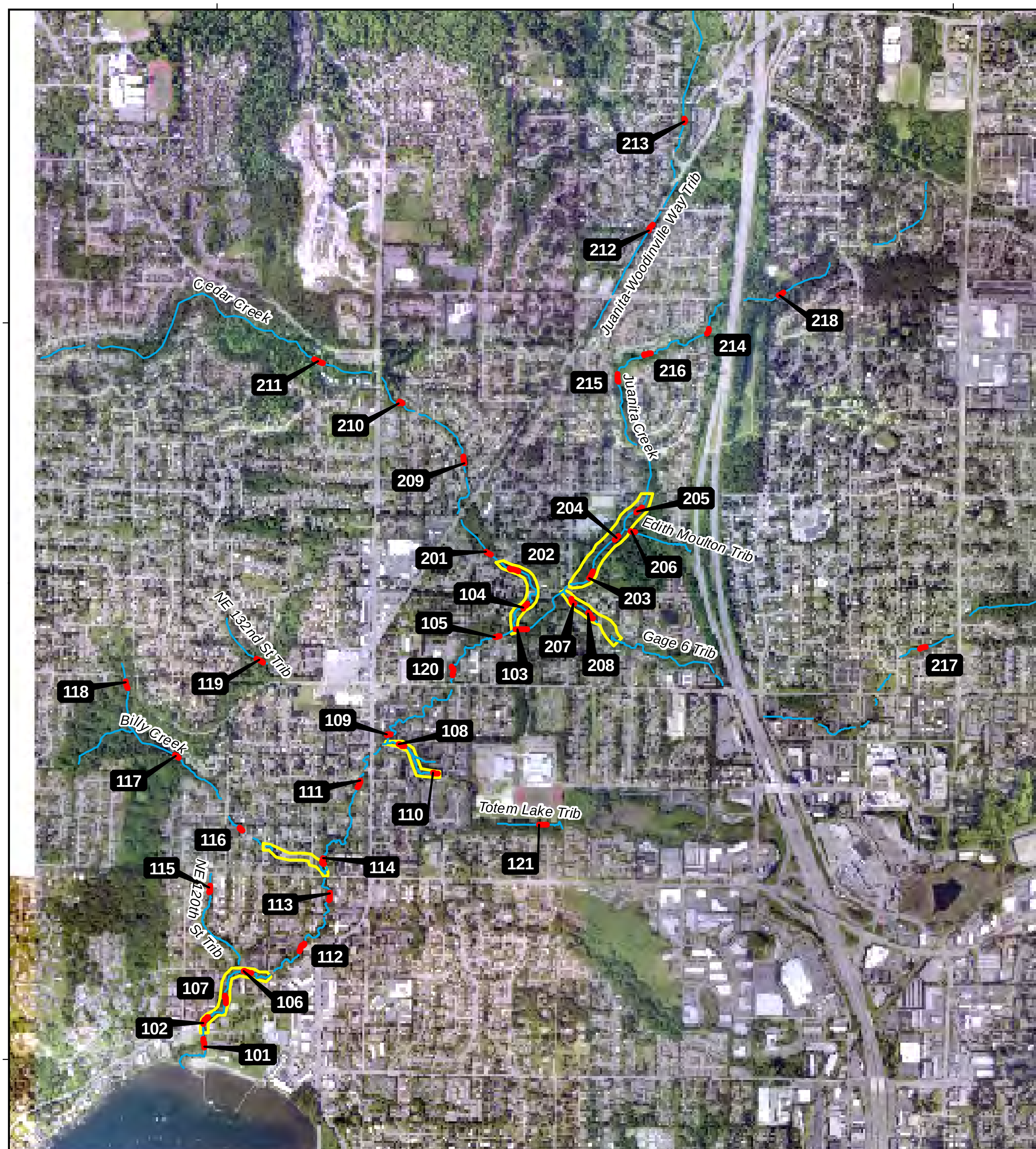
As part of this work, the NHC team observed geomorphic parameters for 39 assessment reaches on the Juanita Creek mainstem and nine tributaries (Figure 1). These parameters included:

- Bank stability
- Substrate size and distribution
- Local slope
- Bankfull channel dimensions (priority reaches only), and
- Large woody debris (LWD) and large pool frequency.

In addition to the nearly 3,800 feet of channel included in the assessment reaches, The NHC team observed LWD and large pool frequency for an additional 4,800 feet of channel length. Substrate sandiness was assessed over a total of 14,600 feet of channel length—30 percent of the open channel network in the entire basin—and included nearly half of the Juanita Creek mainstem.

Though this effort is independent of previous work in the basin, the geomorphic assessment complements and expands upon two previous studies, King County's *Habitat Inventory and Assessment of Juanita Creek in 2000* (Rush et al., 2002) and Otak's *Juanita Creek Basin Stabilization Study* (2000). Both earlier studies were limited to the mainstem of Juanita Creek. The King County assessment included a broad suite of habitat-related attributes: riparian condition, bank stability, adjacent land use, bankfull width and depth, aquatic habitat, pool quality, and LWD. King County sampled selected segments amounting to approximately 35 percent of the mainstem channel between the mouth and 100 meters upstream of I-405. Otak performed a qualitative, observation-based geomorphic assessment, as well as an approximate quantitative analysis based on regime theory, between NE 124th Street and I-405.

This report summarizes the results of the NHC team's field geomorphic data collection, presents an analysis and discussion of trends, and makes recommendations for future work. Section 2 provides data summaries and analyses of trends in and correlations between the observed parameters. Section 3 provides discussion and conclusions regarding what we can infer from the geomorphic data collected, including how they relate to the support of beneficial uses. Section 4 provides recommendations for future work, both as part of the NHC team's remaining involvement in this project (Task 400) and for future outside investigation.

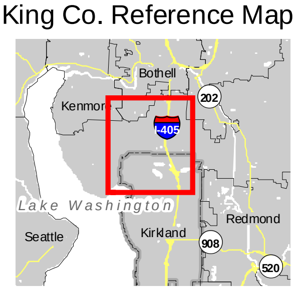


Legend

102 Assessment Reaches

Priority Reaches

King Co. Reference Map



Juanita Creek Geomorphic Assessment - King County, WA

Juanita Creek Geomorphic Assessment Reaches

Scale -1:24,000

1,000 500 0 1,000 2,000 Feet

coord. syst.: WA State Plane N	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 21726	10-Feb-2010

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Figure 1

2. Data Analysis and Trends

Field data collected for the geomorphic assessment, including photographs of each assessment reach, were collected and submitted to the County in the form of an ESRI geodatabase (NHC, 2010a). The spatial-database format facilitates mapping of results and assessment of spatial trends. The following sections provide review and analysis of the various data collected, including spatial trends (or lack thereof), potential correlations between parameters, and comparisons with past studies where applicable. The analysis is primarily qualitative, as the volume and nature of the data generally did not lend themselves to quantitative interpretation.

2.1 Stream Gradient

Channel slopes in the Juanita Creek network (as calculated from a DEM) are overwhelmingly low to moderate, with two-thirds of the channel network (including the entire mainstem) having slopes less than three percent and more than one-quarter having slopes less than one percent. Slopes greater than five percent are rare and occur almost exclusively on the western tributaries coming off of the Finn Hill plateau.

Local slope was evaluated for each of the assessment reaches, either measured directly in the field or (for sites in priority reaches) computed from adjacent cross sections surveyed for the hydraulic model. DEM-computed slopes for corresponding reaches tended to be higher than observed values, though generally within measurement uncertainties. The largest errors occurred mostly on the reaches with the highest slopes. This is not surprising, as in those areas, the channel is typically narrower and thus the GIS channel network (which was used to compute slopes from the DEM) less accurate, leading to greater inaccuracies in the DEM slope computation process. Overall, DEM-derived slopes are probably reasonable for comparison and trend assessment but we do not consider them reliable for local hydraulic or sediment transport analyses.

Plate 1 shows DEM-computed slopes for the entire open channel network, with locally observed slopes also included for the assessment reaches.

2.2 Channel Substrate/Fine Sediment

The NHC team used two metrics to characterize channel substrate. First, a median gravel size (gravel D_{50}) was estimated for each assessment reach *representing only the gravel patches in the reach*; note that this is not the median bed particle size overall because it excluded sand-bedded areas. The gravel D_{50} was determined either from a 100-particle pebble count (for Tier 1 sites in priority reaches) or by visual estimate (Tier 2 sites). Complementing the gravel-size metric, the percent of the bed covered by sand was estimated and classified into one of three bins (0-33% sand, 33-67% sand, or 67-100% sand). This latter metric was evaluated both as a reach average for each of the 39 assessment reaches and by tracking spatial variability in bed sandiness over an extended portion of the channel network. The extended sandiness observations involved continuous assessment of the sandiness categories along the stream channel, delineating segments with consistent bed sandiness and locating changes in substrate composition. Plate 2 illustrates the sandiness characterizations for the assessment reaches and for the extended substrate analysis.

Bed Sediment Trends

The general trend shows fine sediment decreasing up the mainstem of Juanita Creek, though there is some variability in substrate composition throughout. Notable exceptions to this trend

are higher concentrations of fine sediment upstream of NE124th Street, likely due to backwater effects from the culvert and possibly recent activity in the creek (Jenny Gaus, personal communication), and the reach of Juanita Creek through Edith Moulton Park. The latter reach corresponds with an area of locally flatter slope and occurs downstream of a small tributary draining off of I-405 that is very unstable and appears to be a source of significant fine sediment, as evidenced by a visible sandbar deposit at the confluence (Photo 1).

Photo 1. Juanita Creek confluence with left bank tributary in Edith Moulton Park showing extensive sand deposit.

The tributary reaches generally showed a wide range of bed sand percentages (0-33% sand or 33-67% sand). Even higher amounts were observed in the aforementioned Edith Moulton Park tributary, an ephemeral channel fragment off NE 132nd Street, and the northwest tributary along Juanita-Woodinville Way. Fine sediments from this latter tributary may not be reaching the mainstem, insofar as flow at the downstream end of this segment enters a 700-foot pipe that discharges to the creek, and there was no evidence of fine sediment deposition downstream of the pipe outfall.

Gravel was present in all but three of the assessment reaches—Juanita Creek upstream of NE 124th Street (site 114), the ditch along Juanita-Woodinville Way (site 212), and the isolated tributary segment near NE 132nd Street (site 119). Median gravel sizes typically ranged from approximately 11 mm to 45 mm and tended to increase in size with upstream distance from the mouth, though this trend was by no means uniform nor consistent from site to site. These gravels are useable but on the small side of the range typically used by spawning coho salmon and steelhead; they are likely more beneficial for smaller fish such as cutthroat trout (see Table 6 in Section 3.5).

We note that Juanita Creek, like many other lowland streams, may have strong seasonal trends in bed substrate. During a field reconnaissance visit in October 2009, our impression was of much higher bed sand concentrations than were determined during the field data collection in November and December 2009, which followed some substantial storm flows. While it is possible that the preliminary (unmeasured and non-systematic) impressions were not consistent with broader stream conditions, it seems reasonable that greater amounts of fine sediment may collect during summer low flows and then be swept away or redistributed through the system by early winter storms to expose more gravel. If true, this process may have consequences for the spawning success of specific species.

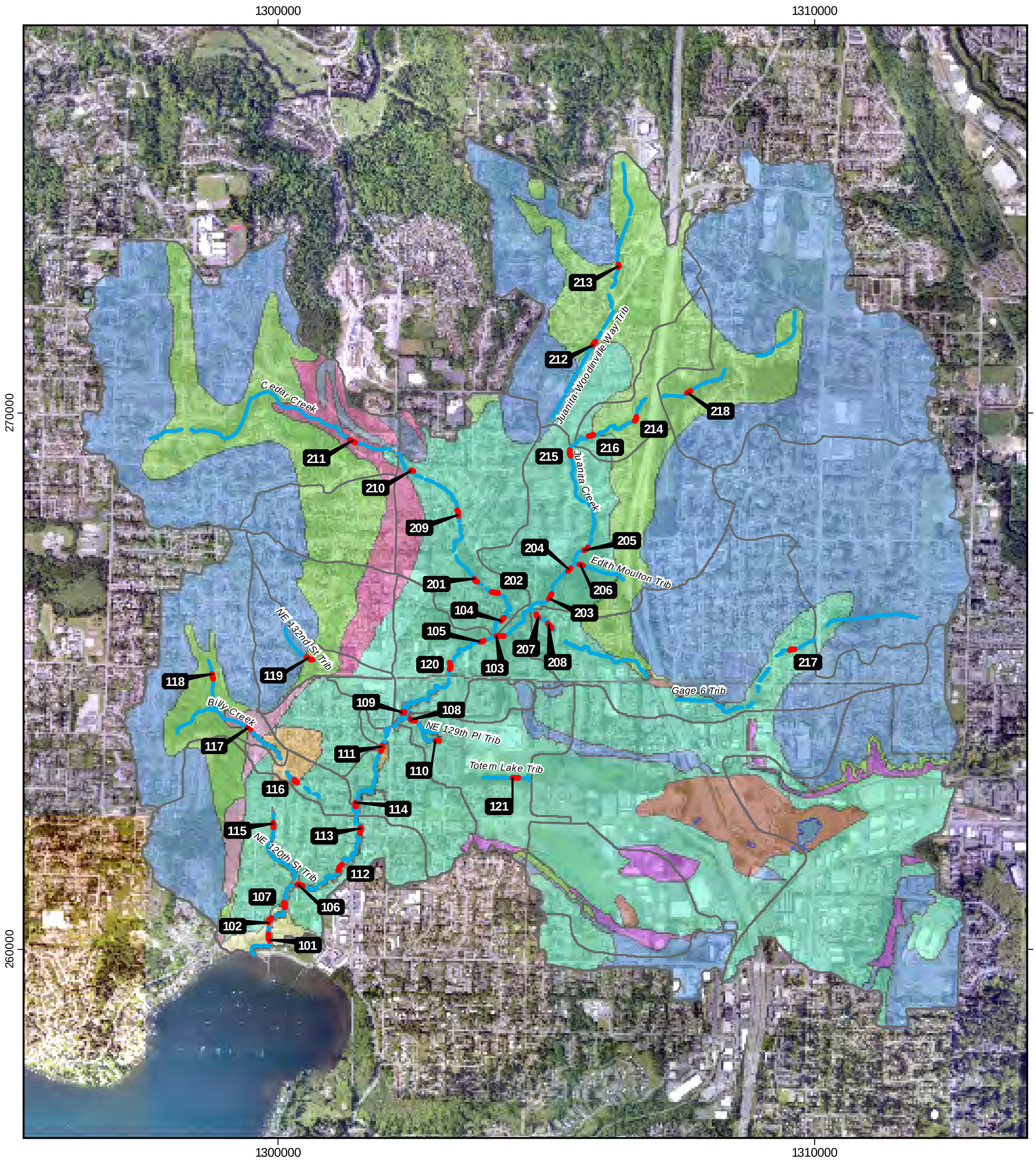
Bed Sediment and Geologic Setting

The surface geology of the Juanita Creek basin (King County, 2003; City of Kirkland areas updated by Troost and Wisher, 2010) is fairly typical of the central Puget Sound lowlands. Most

of the Juanita Creek stream network, and the majority of the assessment reaches, traverses sand and gravel associated with recessional outwash deposits (Qvr) from the Vashon ice-sheet advance about 16,000 years ago. Progressing upslope, the Vashon recessional outwash is flanked by much sandier Vashon advance outwash deposits (Qva). To the west of Juanita Creek, a band of slightly older silt and clay, named transitional beds (Qtb, Qpo), is exposed below the advance outwash and topographically above the (lower, but younger) recessional outwash. As is common in this area, uplands are dominated by Vashon glacial till. Portions of the creek network itself, particularly the lower mainstem, have created narrow bands of alluvial deposits atop the underlying sediments. Figure 2 shows the surface geology for the Juanita Creek basin.

Both outwashes are good sources of relatively coarse sediment to streams, though recessional outwash tends to be exposed on flatter terrain, as in Juanita Creek, where erosive power and consequent sediment production potential is somewhat less. Advance outwash, which is more often exposed on hillslopes as here, is the predominant natural source of sand to stream channels in the region. The silt/clay transitional beds exposed on the western slopes are typically fairly erosion resistant, though can be prone to seepage and landsliding because they impede the downward percolation of groundwater from the plateaus above.

Likely as a result of vigorous rates of sediment transport throughout the watershed, the bed sediment observations generally do not appear to strongly correlate to the adjacent geologic substrate. It could, however, be a contributing factor to higher fine sediment levels observed in the Juanita-Woodinville Way and Edith Moulton Park tributaries. The NHC team found low levels of fine sediment along Billy Creek, which flows through a fairly steep ravine incised into advance outwash and has been anecdotally reported as a significant fine sediment source (Jenny Gaus, personal communication). Given the high levels of instability, however, it is clear that erosion is occurring in the Billy Creek system; the sediment is likely being transported farther downstream along the mainstem rather than being deposited locally. Geologic mapping (Troost and Wisher, 2010) shows an alluvial fan deposit on Billy Creek in the vicinity of 94th Avenue NE, where the creek first enters a pipe system. This suggests that, at least historically, large volumes of sediment were generated in the upper reaches of the tributary but not all reached Juanita Creek itself.



Geology Unit

- Qal - Alluvium
- Qf - Fan deposits
- Ql - Lake Deposits
- Qp - Peat
- Qpf - Pre-Fraser deposits
- Qpo - Pre-Olympia deposits
- Qtb - Transitional beds
- Qtu - Undivided till
- Qva - Vashon advance outwash
- Qvr - Vashon recessional outwash
- Qvt - Vashon till
- Water
- Subbasins

Juanita Creek Geomorphic Assessment - King County, WA

Surface Geology of Juanita Creek Basin

Scale -1:32,000

2,000 1,000 0 2,000 4,000 Feet

coord. syst.: WA State Plane N	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 21726	10-Feb-2010

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Figure 2

Comparison with Previous Studies

Table 1 compares sediment characterizations for the mainstem of Juanita Creek from the earlier Otak (2000) and King County (Rush et al., 2002) studies with our recent observations. In the basin planning study, Otak assessed sediment sizes by taking four bulk samples of sediment along the mainstem at points between NE 145th St upstream of I-405 and NE 124th Street. Otak's study did not include samples downstream of NE 124th Street. King County's habitat assessment study provided only qualitative descriptions of the bed sediment.

The King County study broke the Juanita Creek mainstem into five segments, shown in Figure 3 of their report. Segment 1 spans from the mouth of Juanita Creek upstream to NE 120th Street, Segment 2 from NE 120th Street to NE 126th Street, Segment 3 from NE 126th Street to Cedar Creek, Segment 4 from Cedar Creek to NE 141st Street, and Segment 5 from NE 141st Street to approximately 100 meters east of I-405. To facilitate the limited comparisons that might be justified, the table breaks down the mainstem according to these segments. Both the King County and Otak field observations were made during the summer of 2000.

Table 1. Bed Sediment Comparison with Previous Studies

Segment	**Length (ft)**[†]	**NHC Bed Sediment Distribution**				**King County Notes**	**Median Gravel Size (mm)**	
		Observed Length (ft)	0-33% sand	33-67% sand	67-100% sand		NHC Study	Otak Study[‡]
1	2470	1960	43%	33%	24%	large unstable sediment load in segment, possibly suitable spawning gravel	11-19	n/a
2	2960	1020	45%	25%	30%	many fines on stream bottom	<4-32	9
3	4240	1490	52%	36%	12%	lots of fines	22-64	4.4-9[a]
4	3250	2320	39%	43%	19%			
d/s 108th Ave	1030	1030	67%	15%	18%	gravels possibly suitable for spawning	32-45	n/a
u/s 108th Ave	2220	1290	16%	65%	19%	lots of fines	15-24	11
5	3690	1390	68%	30%	2%	lots of fines		
d/s I-405	3140	1040	67%	30%	2%		11-32	n/a
u/s I-405	550	350	72%	28%	--		32-45	9

[†] Length measured from GIS stream network; not entirely consistent with lengths from King County report.
[‡] D_{50} for bulk sediment sample. Includes fine sediments and subsurface gravels in addition to surface gravels.
[a] Interpolated between two sample points.

The NHC team found sediment composition to be highly variable in the lower three segments, with predominantly gravel stretches alternating with sand blankets and mixed-bed conditions. Median gravel sizes at those locations were also quite variable, though consistently larger than Otak's sampling results, as expected based on the sampling techniques. Similarly, in the upstream segment 5, this study again found substantially lower levels of fine sediment and larger gravel sizes than are suggested by the previous study results. Again, these differences are primarily a result of the different sampling techniques (surface pebble count versus bulk sample), though seasonal factors may play a small role.

Results for segment 4, including the change in bed character from gravelly to sandy upstream of 108th Avenue NE through Edith Moulton Park, were most consistent with the previous studies. This consistency, with nearly a decade between sampling periods, suggests that the fine sediment accumulation in this reach is not a transitory phenomenon. Potential sediment sources and reasons for this distinct change in bed character will be explored further in Section 3 of this report.

In general, the current study found the Juanita Creek mainstem less dominated by fine sediment than suggested by the King County study (their Table 7). In the NHC study, sands were observed to predominate in the lower segments (1 and 2 per the King County study) of the mainstem, though some gravel-dominated segments were also present. Upstream in segments 3, 4, and 5, gravels tended to dominate the streambed, with fines covering less than a third of the bed area (with the notable exception of the reach between the Edith Moulton Park and Gage 6 tributaries). No significant sand-dominated segments were observed on the mainstem upstream of Edith Moulton Park.

We emphasize that these datasets are not strictly comparable, and some of the difference in assessment of fine sediment between the King County and NHC studies may be due to the different seasons in which field work was conducted. The King County study was conducted during the low-flow summer months, while the current study observed conditions during the higher flow period in November and December.

2.3 Bankfull Channel Dimensions

Bankfull channel dimensions were measured only for assessment reaches within the County-defined priority reaches. The five priority reaches (shown on Figure 1), correspond to stream reaches in the vicinity of County flow gages for which the County was interested in more detailed hydraulic and sediment transport information. Bankfull elevations were noted in the field during NHC's hydraulic survey for all cross sections for which they could be readily defined. These were linked to the geomorphic assessment reaches by identifying the hydraulic cross section closest to (or otherwise most representative of) the assessment reach. For representative cross sections for which bankfull elevations were not identified in the field, the bankfull channel was estimated in the office from the surveyed section profile by an experienced engineer/geomorphologist.

The indicators used to identify the bankfull channel were those commonly employed in humid regions and first articulated by Williams (1978): they include the height of the valley flat—or prominent surface on the valley floor—and the observed elevation of the active floodplain, which is the surface of frequent inundation by floods and is typically the lowest level of perennial vegetation. We note that where the valley is confined by adjacent hillslopes (such as a ravine setting) or the floodplain is otherwise narrow or only poorly developed, the results at any one site can be ambiguous.

As is typical for such analyses, the results were plotted against drainage area on a log-log graph. Nearly a century of geomorphic data analysis suggest that a linear relationship (in log-log space) between bankfull channel dimensions and drainage area is commonly observed (e.g. Dunne and Leopold 1978), and that variations in such a pattern can illuminate a legacy of past watershed disturbance. In particular, relationships for streams in western Washington (Booth and Jackson 1997) show that channels with systematically increased discharges resulting from high watershed imperviousness display statistically significant larger bankfull dimensions.

Both overall patterns and some systematic diversions are expressed in the data from Juanita Creek (Figure 3).

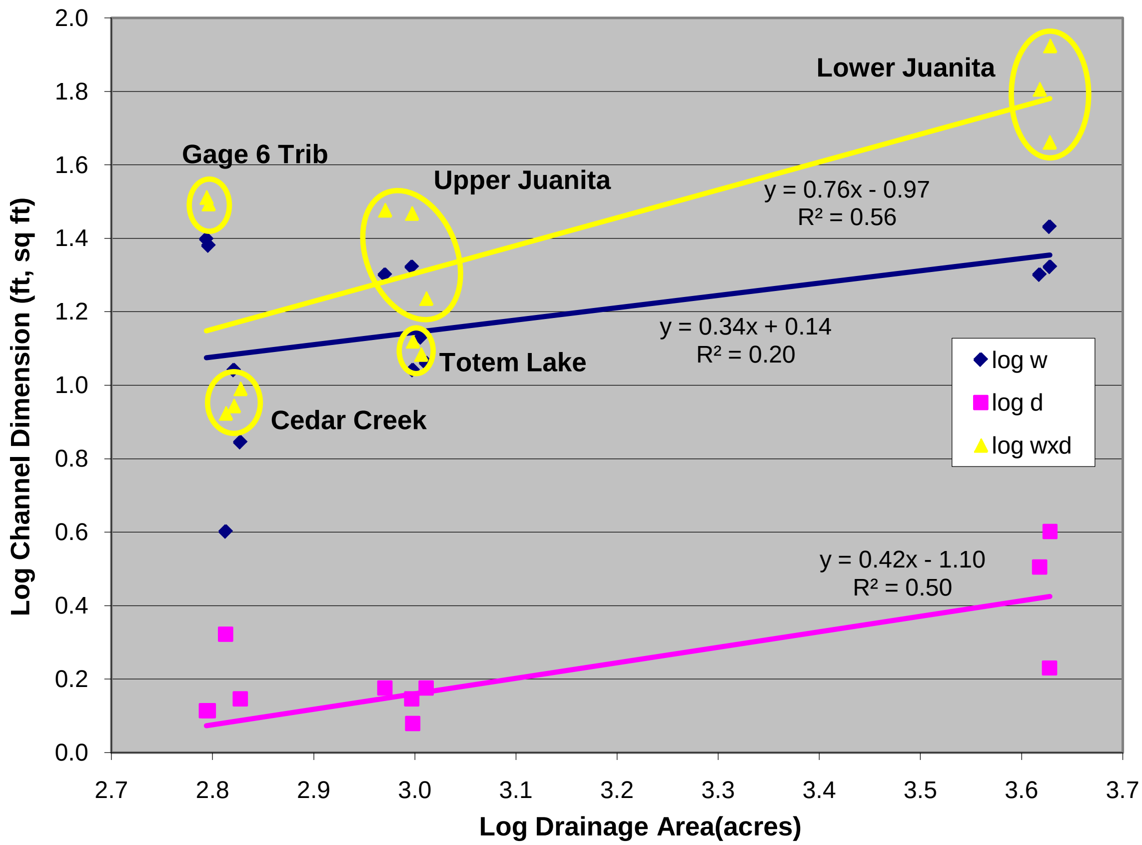

Figure 3. Plot of bankfull dimensions (width, depth, and bankfull area [= w × d]) for the five priority reaches. Labels highlight the grouping of the bankfull area points by priority reach.

As we have observed in other such studies, the variability in both width and depth is greater than that of their product (i.e. bankfull area), but even those data show only a fair correlation with drainage area (r^2 = 0.56). This is likely explained, however, by the presence of two different "populations" of channels amongst the five priority reaches with respect to peak flows: namely, Gage 6 tributary relative to Cedar Creek (50 to 100 percent greater discharges) and Upper Juanita relative to Totem Lake tributary (15 to 50 percent greater discharges). Removing the two "higher discharge" reaches results in a very coherent trend with respect to the channel area-drainage area relationship—an r^2 very close to 1.0 (0.96) and regression equations with exponents (i.e. the multiplicative factor of x) that closely match published ranges. It is thus reasonable to infer that the Gage 6 Tributary and Upper Juanita Creek have seen a disproportionately high magnitude of channel expansion within the Juanita Creek stream network as a result of historic watershed changes.

2.4 Bank Stability

Bank stability was evaluated for each geomorphic assessment reach using the method of Henshaw and Booth (2000). This method, which applies to straight alluvial reaches, uses qualitative indicators of frequency and severity of bank erosion to classify the reach into one of four stability categories (Table 2). For this assessment, a fifth category (Armored) was added for reaches where an appropriately alluvial site could not be identified. The need for bank armoring is typically an indicator of some level of instability in the past, although it generally precludes active bank erosion.

Table 2. Bank Stability Classification Criteria (from Henshaw and Booth, 2000)

Class	Category	Indicators
4	Stable	• Perennial vegetation to waterline • No raw or undercut banks • No recently exposed roots • No tree falls
3	Slightly Unstable	• Perennial vegetation to waterline in most places • Some scalloping of banks • Minor erosion and/or bank undercutting • Recently exposed tree roots rare but present
2	Moderately Unstable	• Perennial vegetation to waterline sparse (mainly scoured or stripped by lateral erosion) • Bank held by hard points (trees, boulders) and eroded back elsewhere) • Extensive erosion and bank undercutting • Recently exposed tree roots and fine root hairs common
1	Completely Unstable	• No perennial vegetation at waterline • Banks held only by hard points • Severe erosion of both banks • Recently exposed tree roots common • Tree falls and/or severely undercut trees common
0[†]	Armored	• Banks held by constructed features (rip-rap, retaining wall, etc.)

[†] Not included in original classification scheme.

Banks of the assessment reaches evaluated by the NHC team fell predominantly into the moderately unstable category: 16 reaches were classified moderately unstable, 9 slightly unstable, 6 completely unstable, 3 stable, and 5 were noted as armored. Even allowing for subjectivity in the classifications, the Juanita Creek stream network is clearly an unstable system. Plate 3 shows stability classifications for the 39 assessment reaches.

Stability Trends and Correlations

There are no apparent spatial trends in bank stability along the mainstem of Juanita Creek, where reaches were largely classified as moderately unstable. The major western (right bank) tributaries—Billy Creek and Cedar Creek—both showed more instability in their upper reaches. The Edith Moulton Park tributary, receiving runoff primarily from I-405, was also classified as completely unstable. In contrast, all assessment reaches on the Totem Lake and Juanita-Woodinville Way tributaries were classified as either stable (accounting for two of the three stable reaches) or slightly unstable. Mainstem assessment reaches immediately downstream of those two tributary confluences were also more stable than surrounding reaches.

The magnitude of bank erosion shows no correlation with local channel slope, but the patterns do suggest the influence of both hydrologic alteration and geologic substrate. Areas of high unit-area discharge, either modeled or inferred by land-cover patterns, are known or likely for Upper Juanita immediately below I-405, the Gage 6 Tributary, and the Edith Moulton Park tributary. These correspond to sites of moderate or high observed bank instability (214 and 216, 208 and 207, and 206).

The geologic material in which the channels are incised is also a likely contributing factor for several of the moderately and/or highly unstable sites that cross the Vashon advance outwash, a very sandy deposit that has been long-recognized for its susceptibility to channel incision (Booth, 1990). These include the upper mainstem (sites 214 and 216) and upper Billy Creek

(site 118). Notable exceptions to this association of high instability with this geologic deposit are Juanita Creek upstream of I-405 (site 218) and the Juanita-Woodinville Way tributary (sites 212 and 213). Other parts of the channel network, generally lying in the headwater reaches of the individual tributaries, also cross this deposit but were not surveyed as part of this study. Where draining highly impervious areas, however, they may also be contributing presently unrecognized high loads of sandy sediment.

Comparison with Previous Studies

Both the Otak (2000) and King County (Rush et al., 2002) reports included some assessment of stability for their assessment sites on the Juanita Creek mainstem, though the methods were different in all three studies. Otak's report included geomorphic field notes as well an approximate quantitative analysis based on regime theory, in which stable channel widths were calculated (assuming existing slopes, dominant discharges, and sediment sizes) and compared with measured widths to determine whether a reach was eroding or aggrading (unstable), or in approximate equilibrium (stable). King County made an assessment of bank stability at regular intervals and computed an overall percent stability for each segment. This approach obscures variability within the 2,500- to 4,500-foot segments (of which 25 to 50 percent was actually assessed) and does not attempt to discern relative degrees of instability.

Table 3 compares the NHC team's stability classifications with the Otak and King County assessments, from downstream to upstream along the mainstem. Our assessments are fairly consistent with the field observations from the Otak report for most reaches, but we note that Otak's field and quantitative assessments were not internally consistent. (The latter analysis concluded that the mainstem of the creek was either stable or aggrading upstream of 108th Avenue NE and generally tending to widen or erode between the 108th Avenue NE crossing and NE 124 Street.) The King County results suggest an overall more stable channel than either of the other two studies, but direct comparisons are difficult given the different assessment scales and techniques.

Table 3. Bank Stability Comparison with Previous Studies

NHC Site	NHC Stability Class	Otak Field Notes	Otak Bed Stability	KC % Stable
101	Armored	Outside study area	Outside study area	43
102	Moderately Unstable	Outside study area	Outside study area	43
107	Slightly Unstable	Outside study area	Outside study area	43
106	Moderately Unstable	Outside study area	Outside study area	43
112	Completely Unstable	Outside study area	Outside study area	68
113	Armored	Outside study area	Outside study area	68
114	Moderately Unstable	Bend with small point bar. Severe erosion on outside of bend. Sand and gravel in channel	Eroding, Unstable	68
111	Slightly Unstable	Severe bank erosion, scalloped bank, sand and gravel in channel	Eroding, Unstable (interpolated)	90
109	Moderately Unstable	Thick sand and gravel in channel. Severe right bank erosion and back stepping. Large gravel bars and thick sand in channel	Eroding, Unstable (interpolated)	90
120	Armored	n/a	n/a	90
105	Moderately Unstable	n/a	Depositional at all flows	90

NHC Site	NHC Stability Class	Otak Field Notes	Otak Bed Stability	KC % Stable
103	Moderately Unstable	n/a		75
203	Moderately Unstable	Ravine-like through school park with banks up to 20' high	n/a	75
204	Moderately Unstable	Channel widening, sand and gravel bars, braided secondary channels through and around bars, shallow banks	n/a	75
205	Moderately Unstable	Channel widening, sand and gravel bars, braided secondary channels through and around bars, shallow banks	Depositional, stable at 2-yr flow	75
215	Slightly Unstable	Thick sand and gravel deposits/bars in channel	Stable	73
216	Completely Unstable	Thick sand deposits in channel	n/a	73
214	Moderately Unstable	Sand deposits associated with wood debris jams, upstream bank erosion, bank failure, and sloughing	n/a	73
218	Slightly Unstable	Banks scalloped and undercut	Depositional or stable	73

2.5 Incipient Motion Analysis

Channel stability and sediment transport capacity along the five priority reaches in the Juanita Creek system (see Section 2.3) were assessed by performing an incipient motion analysis, utilizing Shields criterion. As with the bankfull channel dimensions, this analysis applies only to the priority reach (Tier 1) geomorphic assessment sites. This section provides a brief summary the incipient motion analysis approach and results—more detailed information is available in a separate memorandum (NHC, 2010b).

Approach

The Shields criterion is the standard method for evaluation of incipient motion and is based on laboratory and field observation of sediment mobilization and measured flow characteristics (USACE, 1994). Required input for the analysis includes computed bed shear stress and measured surface bed-sediment grain size.

Bed shear stress along each assessment site within the priority reaches was computed using the HEC-RAS hydraulic model developed for the system. Cross-sections for the HEC-RAS model were surveyed at relatively wide intervals, so channel structure such as pools and riffles was not well-captured. This situation can lead to over-estimates of computed flow velocities and bed shear stress within HEC-RAS, especially at lower discharges. To compensate, NHC performed two tasks. First, additional cross-sections were interpolated in the vicinity of the Tier 1 assessment reaches and the resulting computed shear stresses were averaged. Second, we partitioned the bed shear stress into two components, total shear stress and grain shear stress. Grain shear stress is the portion of shear exerted on the bed grains resulting in sediment mobilization, while total shear stress, which is computed by HEC-RAS, is associated with the entire cross-section.

To estimate the threshold at which sediment is mobilized, the Shields criterion was utilized. As part of this analysis a dimensionless Shields number (τ^*) is computed using the following relation:

$$\tau^* = \frac{\tau_0}{(\rho_s - \rho)gD}$$

Here, τ^* is the critical Shields stress (constant value determined from the literature), τ_0 is the grain shear stress at incipient motion, ρ_s and ρ are the densities of sediment and water, respectively, g is gravitational acceleration, and D is a representative grain size diameter.

The equation above was then used to compute the shear stress (τ_0) responsible for mobilization of four grain size classes: coarse sand (D = 5 mm), small gravel (D = 10 mm), the gravel D_{50} at the site (median gravel size), and the gravel D_{90} at the site (diameter that exceeds 90 percent of those sampled). The shear stress for incipient motion computed using the Shields criterion was then compared with the grain shear stress (derived from total shear stress computed by HEC-RAS) for discharges ranging from 10 percent of the 2-year to the 500-year flow.

Flow Ranges for Sediment Transport

The range of flows for which the four sediment size classes are mobilized is illustrated in Table 4, and summaries of the findings for each of the priority reaches, from upstream to downstream, are provided below. The reader is directed to NHC's technical memorandum (2010b) for more detailed information on incipient motion thresholds, including critical shear stresses and associated flows. Our results indicate that the gravel D_{90} is never mobilized up to the 500-year flow, nor is even the gravel D_{50} in half of the Tier 1 assessment reaches. Note that, as elsewhere in this report, the gravel D_{50} (and D_{90}) refer to the size distribution of surface gravels in each assessment reach—excluding fine sediments—and not to the reach's overall bed particle size distribution. Thus, in sandier reaches, much more than 50 percent of the bed may be mobile at (and even below) the threshold of motion for the gravel D_{50}. For reference, Table 4 includes the level of sandiness (see Section 2.2) of each assessment reach.

Table 4. Bed Characteristics and Transport Flow Ranges

Priority Reach	NHC Site	Sand %	D_{50} (mm)	D_{90} (mm)	Largest Size Class Mobilized at Flow Quantile[†]							
					½ of 1.01	1.01	1.5	2	5	10	25	100
Upper Juanita	205	33-67%	19	69								
Upper Juanita	204	0-33%	24	44								
Upper Juanita	203	33-67%	15	33								
Gage 6 Trib	208	0-33%	43	101								
Gage 6 Trib	207	0-33%	33	79								
Cedar Creek	202	0-33%	21	39								
Cedar Creek	104	33-67%	19	32								
Totem Lk Trib	110	0-33%	25	50								
Totem Lk Trib	108	0-33%	15	36								
Lower Juanita	106	33-67%	15	128								
Lower Juanita	107	33-67%	12	35								
Lower Juanita	102	0-33%	19	38								

[†] Return interval (years), annual series.
Yellow = All sand (D=5 mm); Olive = Small gravels (D=10 mm); Gray = D_{50}; Purple = D_{90}

Upper Mainstem

All sands were computed as mobilized at relatively low discharges in this priority reach. The median gravel size is transported from the upper portion of the reach (Site 205) at the 2-year flow, but may be trapped in the middle portion of the reach (Site 204). Site 203, at the downstream end of the priority reach, is characterized by finer gravels and correspondingly lower bed slope and energy gradient than the middle of the reach. Gravels greater than 10 mm in size are constrained from moving out of this reach by backwater from the 108th Avenue NE culvert. Sand and fine gravel, however, were computed as mobilized above the 1.25-year flow and thus may contribute to downstream deposition.

Gage 6 Tributary

The Gage 6 Tributary is a steep reach with a bed slope of approximately 2.7 percent. Gravels up to 30 mm are readily mobilized in the upper portion of the priority reach (Site 208) by discharges greater than the 1.25-year flow. The downstream end of the reach (Site 207) is backwatered by the 108th Avenue NE culvert at the 1.01-year flow, causing a reduction in bed shear stress up to the 50-year flow and limiting transport to sands between the 5- and 50-year flows. At most flows, however, sand and gravel up to 20 mm are readily mobilized throughout the reach and likely transported downstream to the mainstem, though the gravel D_{50}s (highest among Tier 1 assessment reaches) were computed as immobile.

Cedar Creek

The Cedar Creek priority reach lies between two culverts; however, the downstream culvert causes no backwater even at very large, infrequent discharges. Sand and fine gravels were computed as mobilized at the 1.01-year event and higher, but the gravel D_{50} is not mobilized at discharges less than the 100-year flow at the upstream end of the reach (Site 202). At the downstream end of the reach (Site 104), the gravel D_{50} is mobilized at much lower flows. Hydraulic conditions do not constrain this tributary from delivering sand and gravel to the mainstem.

Totem Lake Tributary

Sand is mobilized in this priority reach at relatively low discharges, and discharges greater than the 2-year flow are competent to move small gravels. Sediment was observed to be coarser at the upstream end of the priority reach (Site 110), where the gravel D_{50} is not mobilized because the energy grade line tends to be low due to adverse bed slope coming out of the culvert. At the downstream end of the reach (Site 108), the gravel D_{50} is mobilized at the 10-year flow, and sediment delivery downstream is not impeded by backwater. This reach was observed to have consistently low sand substrate, which may be caused by upstream sand deposition in wetlands and a detention pond.

Lower Mainstem

Gravels in this priority reach are smaller than the other priority reaches, and at the upper end of the reach (Site 106), the gravel D_{50} is mobilized at much lower flows than computed elsewhere in the network. Beginning in the middle portion of the reach, backwater from the Juanita Drive culvert begins to limit transport at the 1.5-year flow. At the mid-reach site 107, the gravel D_{50} is mobilized only at discharges between the 1.01- and 1.5-year flows, and only sands and very fine gravels are expected to be transported downstream to Lake Washington.

Comparison with Previous Studies

It is difficult to compare this incipient motion analysis with the Otak data and regime calculations (see Section 2.4), because Otak focused solely on the mainstem above NE 124th Street, collected sediment data during the low-flow season instead of the high-flow season, and

collected only four point bulk sediment samples in contrast to our approach of conducting pebble counts of surface gravels at each site. Furthermore, Otak used simplified hydraulics and hydrology compared to the present study.

2.6 Pools and Large Woody Debris

The scope of the current analysis is generally limited to physical geomorphology. However, recognizing that the County's overall goal for Juanita Creek is to restore (or improve) habitat and ecological function, the field assessment was taken as an opportunity to collect additional data that may be useful for later biological assessment and analysis. In this study, the NHC team counted LWD and large pools within the assessment reaches and in the "access reach" segments traversed to reach those assessment reaches. LWD, defined as wood greater than 12 inches in diameter and 10 feet in length, was classified into bins based on pieces per approximately 100 feet (after McBride and Booth, 2005). Large pools, defined as those having a residual depth greater than or equal to the bankfull depth, were counted and their locations recorded. For both LWD and large pool dimensions, measurements were approximate visual assessments only. Plate 4 illustrates LWD and large pool distribution in the areas of the stream network evaluated for this study.

The total channel length over which LWD and large pools were evaluated was approximately 8,600 feet. As expected based on previous work on Juanita Creek, LWD was scarce to non-existent throughout the stream network. Only one reach, just upstream of NE 124th Street near the Billy Creek confluence, had more than five pieces per 100 feet. More than 40 percent of the assessed channel length had no pieces of LWD at all.

Our geomorphic survey identified 29 large pools, which appears to be fairly consistent with King County's findings in the 2002 report. (That report counted all pools; comparison is based on the number of pools identified in the study greater than 0.5 meters deep.) This study did not attempt to evaluate habitat quality of the pools identified.

King County's 2002 Habitat Assessment report provides a much more thorough and comprehensive evaluation of habitat parameters for the Juanita Creek mainstem than was attempted in this study. The reader is directed to that study for more detailed analysis and interpretation, including assessment relative to "properly functioning conditions." King County also recently conducted a B-IBI analysis for seven sites in the Juanita Creek basin, finding that, with the exception of one site on lower Cedar Creek, conditions were generally poor (Berge and Burkey, 2009).

3. Discussion and Conclusions

Restoration of beneficial uses or properly functioning conditions in a profoundly disturbed system such as Juanita Creek requires recognition and rehabilitation of an array of inter-related physical, chemical, and biological conditions. Going into this study, previous work on Juanita Creek suggested that the most significant physical problems in the system are an altered hydrologic regime (specifically increased magnitude and duration of high flows), pervasive channel instability, excessive fine sediment in the bed, and lack of habitat structure and complexity. This section will focus on what the current study tells us about the extent and possible causes of those problems, as well as other issues recognized in the course of our work.

3.1 Hydrology

Increases in magnitude and duration of high flows due to urbanization of the watershed are undoubtedly the primary contributing factor to, if not the root cause of, the widespread channel instability and fine sediment issues in the Juanita Creek watershed. Our work on this study to

date has not included extensive hydrologic analysis; thus, this discussion is somewhat limited by a lack of full understanding of the basin hydrology. Where possible, we have incorporated the hydrologic data that were available and our knowledge of typical hydrologic response to watershed soils and imperviousness.

Of the six major tributaries to Juanita Creek (Upper Juanita, Cedar Creek, Totem Lake Tributary, Gage 6 Tributary, Billy Creek, and Juanita-Woodinville Way Tributary), we currently have peak flow information for four (Table 5).

Table 5. Juanita Creek Tributary Peak Flows

Tributary	Drainage Area (ac)	Peak Flow (cfs) by Return Interval					
		1.01-yr	2-yr	5-yr	10-yr	25-yr	100-yr
Upper Juanita	1040	21	48	64	74	86	103
Totem Lake	1025	15	33	46	57	71	95
Cedar Creek	675	16	31	41	49	58	74
Gage 6	625	39	56	67	76	86	103

Despite having the smallest drainage area, the Gage 6 Tributary shows the highest peak flows. On a unit-area basis, the Gage 6 Tributary flows are two to three times those for Upper Juanita and Cedar Creek for flows up to the 10-year discharge, which are generally more important in determining geomorphic response than larger but more infrequent events. Cedar Creek and Upper Juanita produce virtually the same flows per unit area. Totem Lake Tributary flows are notably lower, most likely due to flow attenuation by upstream wetlands and detention.

Billy Creek has similar soils and land use to Cedar Creek, so its watershed would likely produce similar flows. The Juanita-Woodinville Way tributary has similar land use but a higher percentage of outwash soils than the other upland areas, so unit-area flows would probably be similar to or slightly lower than Upper Juanita. A significant portion of the Edith Moulton Park tributary drainage is occupied by I-405, suggesting that peak flows would be quite high, possibly closer to the Gage 6 Tributary on a unit-area basis.

Based on the available information, the Gage 6 Tributary stands out as a problem area, though the entire Juanita Creek basin has undoubtedly been subject to significant flow increases. King County's HSPF model of the basin provides a key tool for further investigation of basin hydrology to extend our understanding of how the hydrologic regime has been affected and where the greatest needs—and best opportunities—for flow control may exist.

3.2 Channel Stability

The NHC team's observations confirm that bank instability is a pervasive problem through almost the entire watershed, perhaps even more so than suggested by earlier studies. As discussed in Section 2.4, the most unstable reaches observed tend to be associated with areas of high discharges and (in many cases) more erosive geologic substrates.

For the mainstem, the non-geologic factors that contribute to ongoing instability include the (presumed) historic and ongoing increases in watershed imperviousness (and the flow increases that accompany them); channel confinement throughout much of the mainstem corridor, forcing discharges into abnormally narrow channels; and the high volumes of water that provide ample force to erode channel banks that are not at an equilibrium width or slope for the prevailing flow regime.

The Juanita-Woodinville Way and Totem Lake tributaries are the only significant exceptions to the generally basinwide instability. Despite draining a large (roughly 25 percent of the entire Juanita

Creek basin) and highly impervious watershed, all three sites on the Totem Lake tributary were relatively stable. As mentioned above, the Totem Lake basin contains several wetlands and a regional detention pond that attenuate high flows, and peak discharges per unit area are significantly lower than for the other major tributaries.

3.3 Fine Sediment

In contrast to channel stability, results of this study suggest that fine sediment is somewhat less pervasive throughout the channel network than previously reported, though the observed amounts almost certainly reflect substantial degradation of aquatic habitat (see Section 3.5). Surface gravel patches were present throughout most of the stream network and dominant over more than half of the channel length observed. We hypothesize that there is a seasonal trend in the surface expression of fine sediment, with sediments accumulating during the low flow season, then transported downstream as flow levels increase in the fall and winter. This would be consistent with the differences in fine sediments observed in previous studies (during low-flow season) and the current study (where field work was conducted in late fall). As will be discussed further in Section 3.4, Juanita Creek regularly sees flows capable of moving even coarse sands, but due to hydraulic limitations, gravel movement may be very limited in many parts of the system.

Given the widespread instability in the system, a primary source of fine sediment is likely bank erosion. Photo 2 shows examples of bank erosion from three different parts of the basin.

a) b) c)

Photo 2. Examples of bank erosion throughout the Juanita Creek stream network: a) Juanita Creek just above Cedar Creek (site 103), b) Upper Billy Creek (site 118), c) Gage 6 Tributary (site 208)

Currently, bank erosion is occurring almost everywhere in the basin, so there is no “smoking gun” in terms of fine sediment source locations. It would be reasonable to expect, however, that reaches in erosive geologic substrate would have the highest potential for continued erosion, particularly if subject to very high flows or continued increases in flows. This would point to Billy Creek, the upper reaches of Cedar Creek, the upper mainstem, and potentially the Juanita-Woodinville Way tributary as areas of particular concern.

Three locations stand out as hot spots for fine sediment deposition: Juanita Creek upstream of NE 124th Street, Juanita Creek through Edith Moulton Park, and the Edith Moulton Park tributary. The two mainstem reaches are in relatively flat stretches upstream of culverts and near confluences with steeper tributaries with high sediment-production potential. As previously mentioned, the reach upstream of NE 124th Street was subject to recent activity in the creek, so it is unclear whether the deposition in that reach is more transitory. Comparisons with the King County and Otak studies, however, show that the Edith Moulton Park reach upstream of 108th Avenue NE has shown similar depositional character for at least the past decade.

Given significant basinwide fine sediment loads, high sand content in the lower mainstem is expected and cannot really be addressed locally; rather, reduction in upstream sediment would be necessary to limit deposition in these areas. The mixed pattern of bed sediment (i.e. patches of sand, gravel, and mixed bed sediments) in the lower mainstem downstream of NE 124th Street (and possibly even 108th Avenue NE) affirms that large amounts of sand are in transport, and local hydraulics control specific locations of deposition versus continued downstream transport.

Again, the Totem Lake Tributary appears to be the exception to the otherwise basinwide trend. With low levels of fine bed sediment, relatively stable banks, and upstream wetlands capable of capturing sediment from farther up in the watershed, Totem Lake Tributary is unlikely to be a significant source of fine sediment to Juanita Creek.

3.4 Sediment Transport and Hydraulic Regime

With the HEC-RAS model developed in this study, NHC was able to perform more extensive and continuous hydraulic analysis than has previously been reported. Results of the hydraulic modeling were instrumental in the incipient motion analysis (Section 2.5) and are also useful for identifying the influence of hydraulic constraints that may not be readily apparent.

One of the more interesting, and perhaps unexpected, findings is the extent to which backwater effects from various culverts, mainly along the mainstem, limit downstream sediment transport, especially of gravels. By acting as grade controls and slowing flows such that gravel movement is restricted, the culverts are likely limiting incision and helping to maintain local accumulation of bed gravels. Thus, it could be argued that the presence of these restrictive culverts is actually helping to maintain the physical structure of the system. On the other hand, limited sediment transport capacity through these culverts may create a “sediment-hungry” flow downstream of the culverts and lead to additional downstream bank erosion. Scour pools and widened channel sections were observed at the downstream face of several culverts, as is typical of hydraulic flow restrictions, but stability observations do not obviously support exaggerated downstream erosion beyond the immediate vicinity of the culverts.

The incipient motion analysis pointed to the Juanita Drive and 108th Avenue NE (on both Juanita Creek and the Gage 6 Tributary) culverts as the most significant barriers to sediment transport in the priority reaches. Outside the priority reaches, model profiles suggest that the consecutive crossings of NE 128th Street, 100th Avenue NE, and NE 129th Place and (to a lesser extent) the NE 124th Street crossing may also act as sediment barriers.

Given the fact that some of these culverts experience significant backwater even at relatively low flows, it is possible they would be targeted for replacement, either as flood control or fish passage improvements. This bears a caution that replacement of these culverts may have unintended consequences in terms of increased downstream sediment loads, loss of gravels, and potentially bed incision in addition to the channel widening that is already occurring. It appears that some of the culverts are acting as hydraulic brakes, and thus may be serving at least part of the hydraulic function of the pools and large wood that are almost entirely lacking throughout the Juanita Creek system.

3.5 Habitat Structure and Complexity

The LWD and large pool data collected in this study corroborate earlier working assumptions that existing conditions in Juanita Creek are far from ideal in terms of providing instream habitat of sufficient quantity and quality to support native fish species and other biota. Native salmon and trout require cool, well-oxygenated water and complex physical habitat conditions to successfully complete their life cycle. The King County habitat assessment (Rush et al., 2002) completed in

2000 reached similar conclusions. It broadly characterized existing conditions and made recommendations for additional studies, including the current one.

While area estimates of spawning habitat have not been quantified, visual estimates of the spatial extent of sand and fines that cover the stream bed (based on data from this study) strongly suggest that such habitat is limited and is a significant factor in limiting the productivity of native fish populations. Gravel patches of sufficient size and porosity vary as a function of the size of the spawning adults. When these areas are compromised by the chronic intrusion of fine materials, it significantly reduces their quality. Fine sediments (less than 0.86 mm in diameter) that exceed just 10 percent (by weight) of the bed substrate significantly reduce survival of incubating eggs deposited by native fishes. At about 15- to 20-percent fine sediment, egg-to-fry emergence survival drops off precipitously through limitation of intergravel dissolved oxygen and entombment of fry before they emerge (Quinn, 2005). The species-specific preference for spawning gravel sizes and spatial area are provided in Table 6 below (Saldi-Caromile et al., 2004). Note that suitable spawning habitat is determined by a mix of water depth, velocity, and substrate size and presumed porosity.

Table 6. Channel Conditions Required for Spawning Criteria for some Salmonids

Species	Minimum Depth (m)	Velocity (m/s)	Substrate Mix Size Range (mm)	Mean Redd Area (m^2)	Req'd Area per Spawning Pair (m^2)
Fall chinook salmon	0.24	0.30 - 0.91	13 - 102	5.1	20.1
Spring chinook salmon	0.24	0.30 - 0.91	13 - 102	3.3	13.4
Summer chinook salmon	0.30	0.32 - 1.09	13 - 102	5.1	20.1
Chum salmon	0.18	0.46 - 1.01	13 - 102	2.3	9.2
Coho salmon	0.18	0.30 - 0.91	13 - 102	2.8	11.7
Pink salmon	0.15	0.21 - 1.01	13 - 102	0.6	0.6
Sockeye salmon	0.15	0.21 - 1.07	13 - 102	1.8	6.7
Kokanee	0.06	0.15 - 0.91	13 - 102	0.3	0.15
Steelhead	0.24	0.40 - 0.91	6 - 102	4.4 - 5.4	n/a
Rainbow trout	0.18	0.48 - 0.91	6 - 52	0.2	n/a
Cutthroat trout	0.06	0.11 - 0.72	6 - 102	0.09 - 0.9	n/a

While the results of this study suggest that sand covers less bed surface area than reported in previous studies, they nonetheless suggest strongly that spawning habitats that would allow expected survival of incubating eggs are limited within the watershed. Independent of hydrologic alterations, this condition alone will limit survival and rebuilding of native fish populations within the basin. Seasonal fluxes of fine sediment out of gravel-bedded reaches may provide transient spawning habitats, but subsequent deposition of new fines transported from upstream and deposited overtop of established redds will still reduce survival of eggs and alevins.

As was previously reported in the King County survey (Rush et al., 2002), the data from the present study affirm that overall habitat complexity is poor within Juanita Creek and limits the degree to which native aquatic biota might recover to a reasonable approximation of their inherent potential. Deep pools are rare and the distance between pools is very long, challenging both adult and juvenile fish to take advantage of such holding and rearing habitat during critical low-flow months.

The frequency of large wood in the channel is also critically low, which limits available holding and rearing habitats, especially for juvenile salmonids. Wood provides hydraulic roughness and

increases micro-habitat scale upwelling and down-welling areas that can help increase nutrient availability, moderate elevated temperatures, and increase intergravel dissolved oxygen. Although an abundance of undersized culverts, particularly along the mainstem, has provided some degree of structural stability to the channel that might otherwise have been more severely compromised as well, the absence of critical habitat elements is clearly documented in the present data set.

3.6 Critical Reaches

Critical reaches are those that our data suggest may be the biggest problem areas or those that may have the most potential for improvement. The latter group would also include sites that appear to present opportunities for cost-effective solutions, for example, where undeveloped area remains in the watershed that may be preserved to prevent additional future flow increases. Again, additional hydrologic data will provide a wealth of information to further refine these preliminary lists.

Problem Areas

Table 7 summarizes the locations that our data suggest are among the most significant problem areas in the Juanita Creek basin. Specific problems and potential causes have been discussed in the previous sections. To the extent feasible, these areas should be prioritized as targets for mitigation projects.

Table 7. Significant Problem Areas

Location	NHC Site(s)	Category: Hydrology	Bank Stability	Fine Sediment	Sediment Transport	Habitat (LWD/Pools)†
Juanita Creek u/s of 108th NE (Edith Moulton Park)	203, 204		✓	✓	✓	✓
Edith Moulton Park trib	206	✓‡	✓	✓		
Gage 6 Trib	207, 208	✓	✓		✓	✓
Juanita Creek u/s of NE 124th	114		✓	✓		
Juanita Creek d/s I-405	214, 216		✓			
Billy Creek	116, 117, 118		✓			✓
Upper Cedar Creek	210, 211		✓	✓		✓

† Based on limited data collected in this study; not intended as a complete list.
‡ Assumed; data not available.

Opportunity Areas

Locations listed in Table 8 are not necessarily the biggest problem areas in the basin, but rather areas where we perceive potential opportunities for mitigation projects or preventive actions that could limit future degradation to the system. Additional information, such as publicly-owned property, could point to other opportunity areas.

Table 8. Opportunity Areas

Location	NHC Site(s)	Category	Description
Juanita Creek u/s of I-405	218	Flow control, habitat	Preserve undeveloped forested areas in watershed, forested riparian corridor
Upper Cedar Creek	211	Habitat	Preserve forested riparian corridor
Cedar Creek d/s of Juanita-Woodinville Way	201	Habitat	Preserve forested riparian corridor
Upper Billy Creek	117, 118	Habitat	Preserve forested riparian corridor
NE 120th Street trib	n/a	Flow control, habitat	Preserve undeveloped forested areas in watershed
Edith Moulton Park trib	206	Flow control, sediment	Potential restoration, undeveloped floodplain/riparian corridor in park
Juanita Creek through Edith Moulton Park	203, 204	Flow control, habitat	Potential restoration, undeveloped floodplain/riparian corridor in park
Gage 6 trib d/s of I-405	n/a	Flow control	Potential for increased wetland storage

Limited Return Areas

An important corollary to the identification of critical reaches is the identification of areas where mitigation efforts are likely to provide limited improvement to Juanita Creek. The Totem Lake Tributary stands out from this analysis. While the Totem Lake basin almost certainly has internal problems and may be a primary culprit in water quality issues on Juanita Creek, from a hydrologic and geomorphic standpoint, it does not appear to have a significant negative impact on Juanita Creek. Investment in improvements would thus be expected to provide little or no return in terms of measurable improvement on Juanita Creek.

Also, as discussed previously, fine sediment loads in lower Juanita Creek are dominated by sediment generated elsewhere in the basin and transported down the stream network. Thus, until upstream sediment sources can be addressed, any downstream sediment control measures would be only stop-gap solutions likely to be quickly overwhelmed.

4. Recommendations for Future Work

The results and conclusions of this study generally corroborate earlier work on the mainstem and expand our view of the system into some of the tributary channels. This study clearly identifies some critical areas for which specific solutions can begin to be developed but also points to needs for further investigation, both of physical parameters addressed in this study and other areas (e.g. biological and water quality assessments) well beyond our scope.

4.1 Phase II Analyses

The scope for this work includes an allowance for a second phase (Task 400) to begin to identify and evaluate management actions to address the identified problems in the Juanita Creek basin. This section suggests a proposed approach for that work, based on the results of the hydraulic and geomorphic analyses.

Potential Management Actions

Management actions, which potentially encompass physical projects, regulatory changes, and public education and outreach, should be considered in the context of 1) addressing existing problems in the Juanita Creek basin and 2) moving toward the goal of restoration of beneficial

uses in the stream. The primary problems identified in this and previous studies are high flow peaks and durations, channel instability, fine sediment accumulation on the streambed, and lack of habitat structure and complexity. Management actions targeting flow control, instream energy dissipation, bank stabilization, and/or establishment of off-channel habitat and refugia are among the more promising avenues to addressing one or more of the major issues.

Given that the basin is already highly developed and undeveloped space is scarce, the opportunity for more traditional approaches to flow control such as regional detention may be limited. Instead, so-called “alternative stormwater management techniques” such as distributed storage and infiltration, possibly utilizing portions of the public street right-of-way, have the potential to reduce runoff near its source while improving water quality. Infiltration, in particular, may have significant potential in the Juanita Creek basin given the high percentage of outwash soils. It is highly unlikely that even a large investment in traditional and alternative flow control could restore forested flow regimes in Juanita Creek or its tributaries. Therefore, instead of aspiring to this unrealistic goal, efforts should be made to identify alternative flow regime targets that will result in reasonably stable channels and significantly improved aquatic habitat. This identification would provide the basis for estimating the locations and sizes of required flow control facilities.

There are a number of approaches to energy dissipation and bank stabilization that are also consistent with establishment of physical habitat structure. Provided they are designed correctly for an appropriate flow regime, projects employing large woody debris, boulders, and various types of bank plantings can successfully stabilize banks and redirect flows, as well as encourage development of pools and create shelter and lower velocity refugia. In steeper reaches, use of wood and rock to create step-pool channel geometries can similarly reduce flow energy, create habitat structure, and provide some attenuation of peak flows.

Opportunities for floodplain restoration are limited in the Juanita Creek basin because development in many areas extends nearly to the creek. Where buffer areas still exist, however, re-establishing connection between the creek and floodplain enhances storage, promotes creation of off-channel habitat and recruitment of LWD, and provides an area for fine sediment deposition.

Otak’s 2000 study provided a series of potential stabilization measures, many similar to the types of projects outlined above. The Otak report recommends application of a set of stream stabilization and enhancement techniques to both specific and generic locations along the Juanita Creek mainstem. Site-specific recommendations were targeted at existing instream facilities and included: 1) removal of sediment from the existing Highland Woods pond located just upstream of I-405, 2) abandonment of a gravel-filled sediment pond on the upstream side of NE 124th Street, and 3) construction of new floodplain storage and habitat enhancements in the ravine upstream of NE 140 Street. Generic techniques fell into four categories:

- Stream channel restoration, including establishment of meanders, stable cross sections, and pool-riffle sequences;
- Stabilization of debris flows from steep banks by protecting toes of slopes, tightlining stormwater outfalls, and vegetating earth slips and slumps;
- Streambank protection using various "soft" engineering techniques; and
- Flow modification using weirs and deflectors to modify and redirect flow energy.

We are not aware at this time the extent to which Otak’s proposed measures may have been implemented or attempted by the City of Kirkland in the past decade. Some evidence of restoration projects was observed in our field assessment (e.g. channel/meander restoration in the park north of Juanita Drive), though not at the scale and frequency recommended by Otak’s report.

Evaluation Metrics

In order to evaluate the effectiveness of potential solutions and compare them to existing conditions and other alternatives, it is important to develop a set of metrics that: effectively represents the (not always readily quantifiable) variables of interest; is capable of distinguishing between desirable and undesirable conditions; and, ideally, can be linked to habitat function or ecological health. Where models can be used to reasonably represent alternative actions, metrics that can be computed or derived directly from model data have the advantage of being useful in a predictive capacity; i.e. they can be used to assess an unobservable "with project" condition.

Quantitative metrics can be readily identified for evaluation of hydrologic, sediment-transport, and stability impacts. In past work, NHC has used metrics quantifying the frequency and duration of bed-entraining flows and the effective energy of the stream to characterize geomorphology. Several of these metrics were based on those developed by Booth et al. (2001) on Puget Sound lowland streams and by Doyle et al. (2000) to distinguish channel stability on Midwestern streams. For Juanita Creek, we would suggest several hydro-geomorphic metrics:

- **F_{Qe}**: Disturbance frequency of spawning gravels, i.e. frequency of flows capable of mobilizing spawning gravel, as an average number of events per year.
- **T_{Qe}**: Cumulative duration of spawning gravel entraining flows, after Booth et al.'s T_{Qmean} or $T_{0.5yr}$
- **Effective energy**: Average annual effective work (to entrain bed sediments) per unit width, calculated from the integral of unit effective stream power
- **$T_{0.5yr}$**: Cumulative duration of flows exceeding the "half-year" discharge (i.e. flow that is exceeded on average twice a year). Metric proposed by Booth et al. (2001) that tracks well with B-IBI and is not sensitive to basin area (as is the analogous T_{Qmean})

These metrics have been found to be robust and capable of providing relative assessments of hydrologic and/or geomorphic conditions compared to some threshold or reference condition. Except as it can be manifested in changes to flows, velocities, or channel dimensions, physical habitat is more difficult to quantify with such metrics.

For evaluation of actions aimed at physical habitat creation, use of something like the National Marine Fisheries Service (NMFS) Matrix of Pathways and Indicators (NOAA, 1996) to set targets or estimate improvement toward defined "properly functioning conditions" may be a more useful evaluation tool. This was the approach used to characterize 2000 conditions in Juanita Creek in King County's habitat assessment (Rush et al., 2002).

Evaluation Approaches

To kick off Phase II, we recommend a workshop with King County (and City of Kirkland) staff to share and transfer knowledge regarding specific hydrology, geomorphology, and habitat issues and concerns in Juanita Creek, as well as solutions that may have already been proposed, considered, or attempted in the basin. The product of that meeting would be a list of locations and potential actions to be evaluated and a list of metrics by which they would be evaluated.

Using this list as a starting point, the NHC team will evaluate the effectiveness of proposed actions or combinations of actions in improving hydrologic, stability, sediment, and habitat conditions both locally and downstream. To the extent that the proposed actions or their effects on the channel can be represented in the HSPF model, that will be a primary evaluation tool. The specific metrics proposed above can all be computed from HSPF-simulated time-series and measured channel properties and then compared with existing conditions, potentially a desirable reference condition, and other solutions.

Proposed actions or parts of actions that cannot be effectively modeled, such as habitat quality, would be qualitatively evaluated by appropriate experts (at the County, Stillwater, or NHC) and some sort of ranking system devised to assess relative benefits.

4.2 Additional Field Investigation

The data collected in this study are much more spatially extensive than had been previously collected on Juanita Creek. This has provided opportunity for broader assessment of the basin but also raised additional questions.

Substrate comparisons with previous studies suggested the possibility of a seasonal trend in fine sediment accumulation. A similar sediment observation program to what was undertaken for this study in the spring and/or summer of 2010 would provide a set of comparable data from which the presence and magnitude of that trend could be better characterized.

The incipient motion analyses completed for this study required assumptions regarding the relationship of forces acting to mobilize the bed particles (grain shear) with total shear in the cross section. To the extent that the County anticipates using these results further, our assumptions may be best verified by field studies of incipient motion of different gravel sizes in priority reaches.

We also believe that a more continuous bank stability assessment, along the lines of our extended substrate analysis would be especially enlightening. The assessment reach characterization performed in this study was more of a snapshot approach, with the thought that we might observe broader spatial trends in bank stability. Absent these, a more extensive coverage emphasizing variability along a reach may reveal trends that could not be discerned from the current dataset. An extended stability dataset could also be correlated with the extended substrate data and hydraulic conditions to evaluate some of the relationships suggested in this report.

4.3 Other Areas

As stated in the Introduction, this study is only a part of a larger investigation and effort to restore beneficial uses in Juanita Creek. While we believe it contributes to the understanding of physical factors affecting the creek's ability to support native biota—hydrology, geomorphology, and (at a limited level) physical habitat—chemical and biological factors are well beyond our scope. Similar efforts to this study in those areas, if not already underway, will be integral to developing a comprehensive basin plan and management approach. Observations of juvenile fish within the system indicate that limited populations of native fish are able to successfully spawn and rear in the system. Eventually understanding specific locations of these habitats and other factors that currently limit expansion of these populations will allow for a more focused restoration effort.

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Memorandum

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DATE: September 16, 2011 *(Revised December 20, 2011)* PROJECT: 21726
TO: Mark Wilgus, Jeff Burkey, Curt Crawford, Dale Nelson
COMPANY/AGENCY: King County Water and Land Resources Division
FROM: Peter Brooks and David Hartley
SUBJECT: Juanita Creek- Relationship of Roughness, Gravel Size, and Disturbance Flows

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Introduction

A combination of data obtained from field measurements, hydraulic model computations, and the literature were utilized to develop gravel disturbance metrics on Juanita Creek. These metrics include selection of an appropriate Shield's parameter to predict incipient motion of gravel material, estimation of the threshold discharge (Q_c) at which existing and proposed median grain size (D_{50}) will mobilize, and the development of an approximate relationship between Q_c and Manning's n values. These metrics were evaluated at Sites 107, 105, 203 that represent lower, mid, and upper Juanita Creek, respectively.

Shield's Parameter

Incipient particle motion was evaluated using a standard Shield's analysis that relies on the selection of Shield's parameter (τ^*_c). Shield's parameters are commonly cited as ranging from 0.045 to as low as 0.03 for engineering applications (USACE, 1995), but wider variations have been observed in natural settings (Buffington and Montgomery, 1997). Wilcock et al (2009) suggest the influence of sand on the surface can reduce standard values by a factor of four, resulting in enhanced gravel transport for a given discharge.

Findings obtained from field measurements, including pebble counts and bedload sampling conducted on Juanita Creek, are inconclusive but do suggest that the results of Wilcock et al (2009) are valid to some degree.

As such NHC employed a methodology for selecting the Shield's parameter based on conditions observed in the reach of interest. If the sand component observed in the bed was less than approximately 10%, a standard Shield's parameter of 0.045 was implemented. If the surface sand content was observed to be greater than 10% a "modified Wilcock" value of 0.0225, or half the standard value, was selected as the Shield's parameter. Here, a factor

four reduction was deemed too drastic considering the inherent uncertainties associated with computations as well as measurements.

Shear Partioning

NHC partitioned the bed shear stress into two components, total shear stress (τ_t) and grain shear stress (τ_g), and then derived the following relation:

$$\frac{\tau_g}{\tau_t} \propto \frac{f_g}{f_t} \qquad (1)$$

Here, f_g and f_t are the Darcy friction factors associated with grain and total roughness, respectively. Grain roughness, fg, can be computed using the following empirical relation (Henderson, 1966):

$$f_g = 0.113\left(\frac{D_{84}}{R}\right)^{\frac{1}{3}} \qquad (2)$$

In this relation R is the hydraulic radius of the channel (in feet) and D_{84} is the particle diameter (in feet) that exceeds 84 percent of the particles sampled. Combining the Manning's and Darcy equations, total roughness, f_t, can be computed using the following equation:

$$f_t = 8g\left(\frac{n_t}{1.49R^{\frac{1}{6}}}\right)^2 \qquad (3)$$

Here, g is gravitational acceleration and n_t is the total Manning's roughness value, i.e. the value used to represent channel roughness in the HEC-RAS model. Relations 1-3 allow for the estimation of bed shear stress associated with incipient motion, τ_g, using results computed by HEC-RAS, i.e. τ_t, and sediment characteristics of the respective study reach.

Gravel Disturbance Metrics

From a simplified hydraulic standpoint, there are effectively two possible ways to reduce the disturbance frequency of a given gravel size in a stream channel: 1) increase the overall roughness of the channel, i.e. the *total* Manning's roughness (n_T); or 2) coarsening of the gravel bed, i.e. increase the effective D_{50} particle size.

Figures 1-3 illustrate the impacts varying the total Manning's roughness coefficient, along each reach, on computed grains shear stress (τ_g) over a range of discharges. Threshold shear stress (τ_c) computed for several grain sizes of interest, including existing condition D_{50} values at each reach, as well as those observed as suitable spawning gravels for Coho salmon, are plotted for reference. Kondolf and Wolman (1993) report Coho spawning grain size data from 11 different sites. A median spawning gravel size of 16.5 mm was measured, with lower and upper quartiles of 11 and 31 mm, respectively.

Lower Juanita Creek (Site 107)

- Existing total Manning's roughness values, n_T, on lower Juanita Creek likely range between 0.035 and 0.040.
- Existing D_{50} particle sizes are relatively small (11.7 mm)
- Based on observed surface sand content incipient motion was assumed to be reduced, i.e. a 'modified Wilcock' values for the Shield's stress (τ^*_c) for this reach was set to 0.023.
- Backwater influences created by the Juanita Creek Drive culvert reduced the computed bed shear stress at discharges exceeding the 1.01 to 1.25-year frequencies, thus the range discharges exceeding this level are not plotted on Figure 1.

The two upper τ_g profiles in Figure 1, computed for likely existing n_T values of 0.035 and 0.040, lie well above the existing particle τ_c threshold (0.09 psf), thus illustrating the likely mobility of these particles during low-frequency events (less than 50% of the 1.01-year flow, 56 cfs). Note: At the lowest discharge evaluated (10% of the 2-year flow, 21) HEC-RAS defaulted to critical depth for the 0.040 τ_g profile at one of the cross-sections evaluated and resulted in computation of an excessively high, and likely unrepresentative, τ_g value. As such, the data point on Figure 1 was omitted.

Incremental increases to the total channel roughness (n_T) results in a decrease of the τ_g profiles, but the existing D_{50} particle is still computed as being mobile up to an n_T of approximately 0.048.

When values of total channel roughness (n_T) begin to exceed 0.048, the threshold discharge to initiate motion begins to increase. For example, the existing D_{50} particle is mobilized at the 1.01-year flow (112 cfs) for an n_T value of approximately 0.054.

Increasing the total channel roughness also increases the influence of the backwater at lower discharges, thus resulting in a decrease of particle mobility. For an n_T value of 0.060, at no discharge will the existing D_{50} be computed as mobile. In other words, backwater conditions are compounded by friction losses within the channel and essentially 'choke' the channel. Under such conditions sediment aggradation could be expected, as well as potentially negative flood impacts.

An increase in threshold discharge (Q_c) can also be achieved by coarsening the channel bed. Were the median particle diameter of the bed material be increased from the existing D_{50} value of 11.7 mm to approximately 27 mm, the Qc could increase to just less than the 1.01-year flow (112 cfs).

Manipulation of both channel roughness and surface grain size distribution may be a suitable option to reduce disturbance frequency. Figure 1 indicates a modest increase in total channel roughness from the existing value of approximately 0.035-0.040 to 0.045, combined with coarsening of the surface grain size distribution to achieve a D_{50} of 14-18 mm would increase the threshold discharge from a negligibly small value to approximately the 1.01-year flow. This may be a suitable option if excessive roughening of the channel becomes a concern with regards to flooding. From a spawning gravel standpoint the current D_{50} value of 11.7 mm does fall within the range observed for Coho salmon, but on the lower end. Increasing this value slightly through selective gravel augmentation may yield benefits to in-stream habitat.

Middle Juanita Creek (Site 105)

- Existing total Manning's roughness values, n_T, on middle Juanita Creek likely range between 0.036 and 0.040.
- Existing D_{50} particle sizes are relatively large (39.5 mm)
- Based on limited observed surface sand content, the Shields parameter for incipient motion was assumed to be standard value of 0.045.
- Backwater influences do not exist at this site (at least in HEC-RAS model)

The grain shear stress (τ_g) profiles in Figure 2 resemble those expected in a reach without backwater influences, i.e. the computed value of τ_g increases with discharge.

The two upper τ_g profiles in Figure 2, computed for likely existing n_T values of 0.036 and 0.040, indicated the threshold discharge for existing conditions at Site 105 is approximately a 100-year flow (290 cfs).

The surface substrate at Site 105 is armored with generally coarse gravel and cobbles and the channel is relatively confined between 3-4 ft high banks. The armored condition at this site implies that this site is supply-limited, but it may be more accurate to characterize it as a transport-reach where material from upstream is transported though with little deposition occurring.

The measured existing D_{50} grain size of 39.5 mm actually exceeds the maximum D_{50} value for Coho spawning of 33 mm, reported by Kondolf and Wolman (1993).

Due to the coarseness of the existing bed material, manipulation of the channel roughness and surface grain size distribution would not result in the benefit of reducing disturbance frequency at this site.

Increasing the channel roughness may actually help capture smaller gravel transported from upstream, and effectively reduce the surface grain size distribution, possibly into the range preferred by Coho.

Upper Juanita Creek (Site 203)

- Existing total Manning's roughness values, n_T, on upper Juanita Creek likely range between 0.035 and 0.040.
- Existing D_{50} particle sizes are relatively small (15.3 mm)
- Based on observed surface sand content, the Shields parameter for incipient motion was assumed to be reduced, i.e. a ' modified Wilcock'. The selected Shield's stress (τ^*_c) for reach was set to 0.023.
- Backwater influences created by the 108th Ave NE culvert reduced the computed bed shear stress at discharges exceeding the 2- to 50-year frequencies, thus the range discharges exceeding this level are not plotted on Figure 3.

The two upper τ_g profiles in Figure 3, computed for likely existing n_T values of 0.035 and 0.040, indicated the threshold discharge for existing conditions at Site 203 is between the 1.01-yr flow (21 cfs) and the 1.25-year flow (36 cfs).

Site hydraulic and sediment transport measurements were collected on March 9, 2011 at Site 203. Measured discharge ranged from approximately 12 to 40 cfs (~ 1.5-yr flow at peak) during the visit. A Manning's n value of 0.036 was calculated from velocity and water surface slope measurements. Gravel particles were collected by the sediment trap with

largest being in the 22 mm size class. These measured values agree well with computed values shown in Figure 3.

When the value of total channel roughness (n_T) is 0.050, the threshold discharge to initiate motion increases to approximately the 2-year flow (~48 cfs). At n_T values greater than 0.050 backwater influences, similar to those previously discussed on lower Juanita Creek (Site 107), begin to occur and result in decreased particle mobility.

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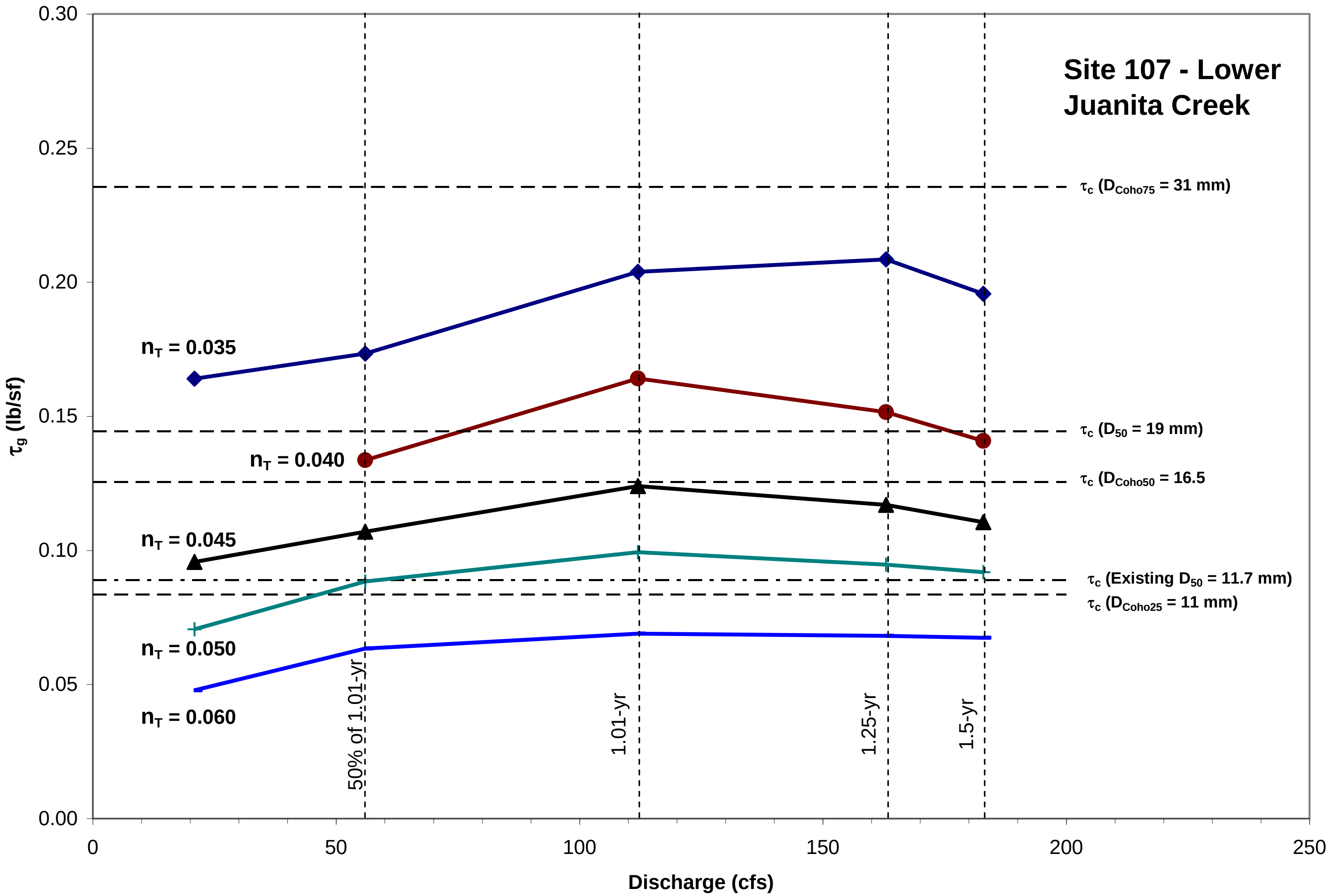

Figure 1. Grain shear stress (τ_g) as a function of discharge for varying values of total channel roughness (n_T) at Site 107, Lower Juanita Creek.

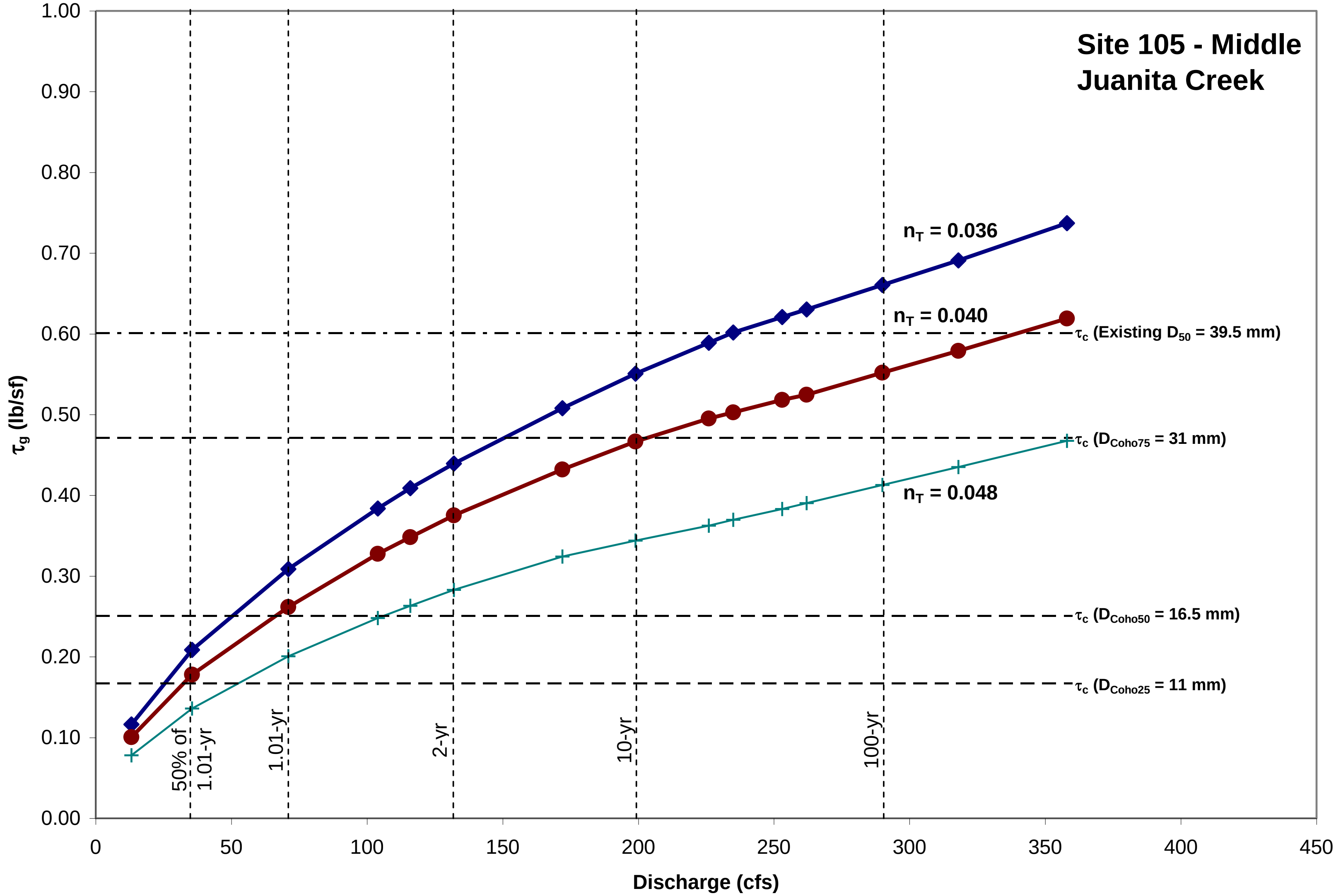

Figure 2. Grain shear stress (τ_g) as a function of discharge for varying values of total channel roughness (n_T) at Site 105, Middle Juanita Creek.

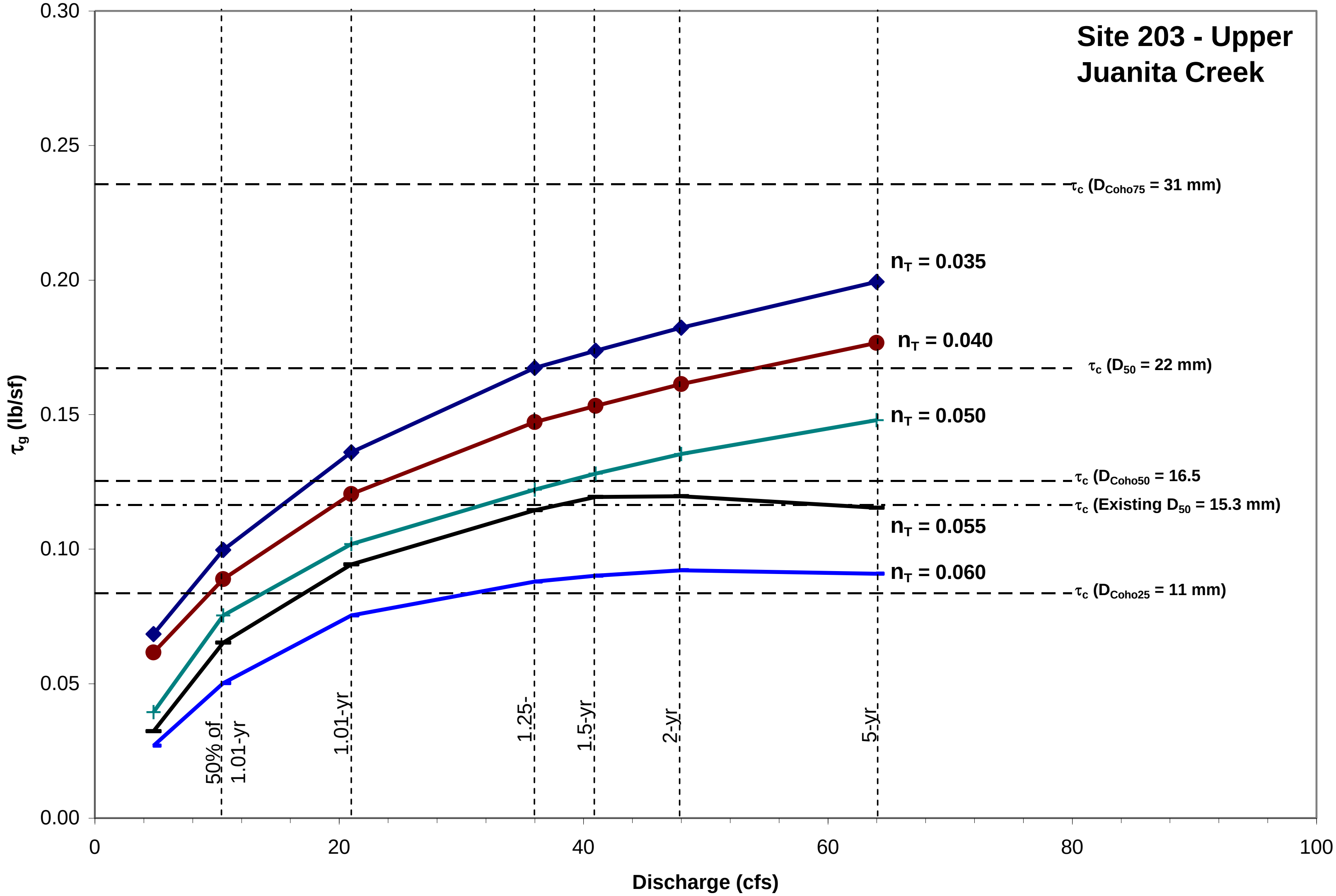

Figure 3. Grain shear stress (τ_g) as a function of discharge for varying values of total channel roughness (n_T) at Site 203, Upper Juanita Creek.

Memorandum

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Date: June 21, 2010

To: Steve Foley and Jeff Burkey, King County Water and Land Resources Division

From: David Hartley, Peter Brooks, Derek Booth (Stillwater Sciences)

Subject: Addendum to Draft Incipient Motion Analysis for Priority Reaches in Juanita Creek

Pages: 5

Introduction

Since submission of the original incipient motion analysis memorandum (NHC, January 11, 2010) followed by the project report (NHC and Stillwater Sciences, February, 2010) which included a summary of the results of the incipient motion analysis, it has come to NHC's attention that recent research (Wilcock et al, 2009) on gravel mobility suggests that a far more mobile condition for bedload sediment in Juanita Creek might exist than previously described by project documents. A key conclusion of this recent research is that the presence of relatively small percentages of sand (i.e., 10% to no more than 30%) on the bed of a flume or stream mixed with coarser gravel material causes the gravel particles to mobilize at shear stresses that are up to 4 times smaller than the critical shear stress calculated using the typical value of the Shields parameter for mixed gravel-sized sediment (0.045). Given that fine sediment at such levels are pervasive within the Juanita Creek stream system, this theory of mixed sediment mobility would suggest that gravels throughout Juanita begin to move at discharges that are considerably lower than previously reported in the aforementioned report and memorandum. This addendum provides guidance and recommendations to the County for re-interpretation of previously presented results as well as for further analysis of sediment transport in Juanita Creek.

Bed Material Movement of Gravel in the Presence of Sand

Initiation of motion, sediment transport, and the evolution of the grain mixture and form of the stream bed in a hydrologically and geomorphically dynamic system like Juanita Creek involves extremely complex, three-dimensional, temporally and spatially variable processes that defy precise numerical analysis, simulation or quantification. Given these complexities, earlier project work related to sediment mobility took a well-established, simplified approach of using a standard and constant value for the dimensionless critical shear stress (Shields Parameter) equal to 0.045. This parameter is central to the determination of sediment mobility, because it is directly proportional to the shear stress needed to mobilize the bed but is the only parameter that cannot be directly measured.

Calculation of the critical shear stress required to move each size class (sands, D_{50}, and D_{90}) was made separately for Tier-1 sites where a pebble count was taken, using a single value for Shields Parameter. While this simplified approach has the virtue of being understandable and unpretentious in its objectives, it should also be the best feasible

approach possible and convey reasonably conservative estimates. Our reconsideration of the Juanita Creek incipient motion situation suggests that these caveats have not been sufficiently met in our earlier work products.

The reasoning behind this change is as follows: the method we applied to identify the critical shear stress, corresponding discharges, and flow quantiles at which the different sized particles would move at the Tier-1 sites would have been approximately valid for plain stream beds with mixed gravel sediment having the measured median particle sizes. As mentioned in the introduction, an increasing body of new data and research indicates that the presence of sand in a gravel-dominated bed significantly reduces the shear stress necessary to move that gravel. As shown in Figure 1a (after Wilcock et al., 2009), the dimensionless critical shear for gravel movement decreases from above 0.040 down to less the 0.010 as the sand percentage on the surface increases from 10% to 30%. This empirical result may be viewed as a kind of "reverse hiding factor". The logic behind the hiding factor is that finer particles "hide" behind or are shielded from flow forces by larger particles and therefore do not move as readily as they would if only their size made up the bed. This phenomenon has been postulated and observed by sedimentologists since the early 1950s. The effect of the hiding factor is illustrated by the curves shown in Figure 1b for bimodal sediment mixtures of sand and gravel in which the gravel-to-sand size ratio varies from 10 to 50, going from the lowest curve in the figure to the highest.

One way to view the *reverse-hiding factor* and *hiding factor* data summarized by these figures is in terms of an *effective grain size*—or, alternatively, an *equivalent critical shear stress* since these are linearly related for sediment of a given density. The *effective size* is the size that is equivalent to what has been observed for stream beds with uniform sediment. Figure 1a indicates that sand fractions on the bed that are greater than 30% reduce the effective size of the co-existing gravels by a factor of four compared to a bed composed solely of that gravel size. Conversely, as indicated by Figure 1b, when sand makes up only 10% of a bed, the effective size of that sand increases by as much as a factor of eight (based on the lowest curve corresponding to gravel size to sand size ratio of 10) compared to a plain sand bed composed of that size sand.

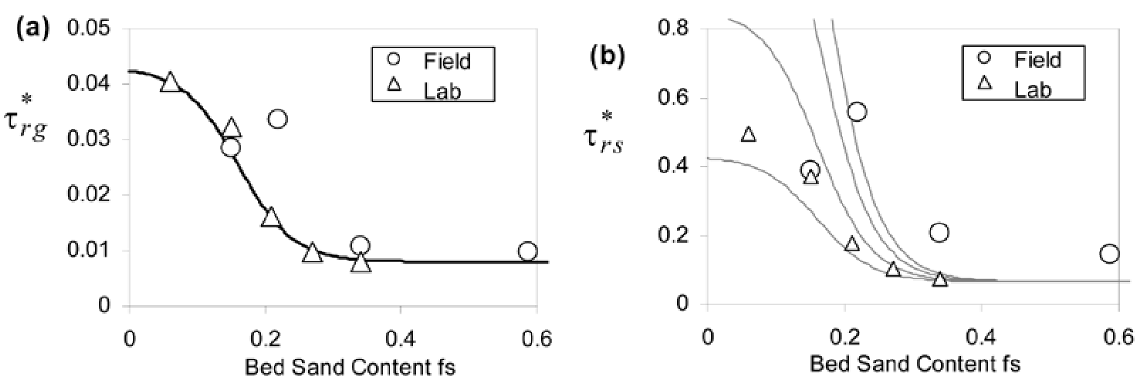

Figure 1. Change in the critical shear stress (τ^*) as a consequence of changing sand fractions (from Wilcock et al., 2009).

In effect, based on these data and concepts, approximately ***the same shear stress and corresponding stream discharge*** would be expected to initiate both gravel and sand motion for a mixed bed surface composed of 85% gravel of 20-mm size and 15% sand of 2-mm size. This critical shear stress would be only about half of the critical shear stress needed to mobilize a bed composed 100% of 20-mm gravel bed—in other words, adding sand makes the bed much easier to mobilize. It is also approximately five times the critical shear stress required to mobilize a uniform bed composed of 2 mm sand.

Re-Interpretation of Incipient Motion Calculations

During field data collection for this project, varying amounts of sand were observed on the surface of the bed of Juanita Creek at all sites; however, the sand percentage was characterized only semi-quantitatively in bins of 0-33%, 34-66% and 67%-100%. At all sites, including the Tier-1 sites where pebble counts were taken to establish gravel size distribution, stream bed patches reflecting all three sand percentage bins were typically observed. Given the high critical shear sensitivity to relatively low percentages of sand in the bed noted above, it is recommended that the critical shear values previously calculated for gravels be *reduced by a factor of two* corresponding to a bed sandiness of approximately 15%. The resultant critical shears and discharges should be regarded as reflective of conditions under which sediments of the specified size and smaller would be set in motion throughout the length of the Tier-1 site in question. A comparative summary of results of this recommendation is shown in Figure 2a and 2b below. Figure 2a is the original figure from the February, 2010 Geomorphic Assessment report based on the constant dimensionless critical shear stress value of 0.045, while Figure 2b provides the same information based on the revised approach for mixtures of gravels and fines discussed above.

In contrast to the original analysis, the revised analysis indicates that with a couple of exceptions, the D_{50} gravel mobility is expected to occur at discharge values as low as 50% of the 1.01-year peak which is typically exceeded over 20 times per year in the Juanita system. Exceptions include the lower portions of the Totem Lake tributary and the lower mainstem just above NE Juanita Dr. Additionally, in the original analysis there was virtually no D_{90} mobility predicted (an unusual outcome, as discussed at the time), while in the revised analysis the D_{90} is expected to be mobile at approximately bankfull discharges on lower Cedar Creek (site 104) and also on the upper mainstem of Juanita Creek (site 205). As originally noted, culverts within Juanita Creek cause backwater at extreme flood discharges and suppress upstream sediment mobility and bed scour at several locations; however, this is not as constraining to overall gravel transport as originally reported.

In our judgment, these analytical results correspond much more closely to both observed conditions along the Juanita Creek channel network and our expectations from other measurements on alluvial stream systems in humid regions. The benefit to the County of

Figure 2a. Original Figure 4 from February, 2010 Report

Priority Reach	**NHC Site**	**Sand %**	**D_{50} (mm)**	**D_{90} (mm)**	**Largest Size Class Mobilized at Flow Quantile†**							
					½ of 1.01	1.01	1.5	2	5	10	25	100
Upper Juanita	205	33-67%	19	69								
Upper Juanita	204	0-33%	24	44								
Upper Juanita	203	33-67%	15	33								
Gage 6 Trib	208	0-33%	43	101								
Gage 6 Trib	207	0-33%	33	79								
Cedar Creek	202	0-33%	21	39								
Cedar Creek	104	33-67%	19	32								
Totem Lk Trib	110	0-33%	25	50								
Totem Lk Trib	108	0-33%	15	36								
Lower Juanita	106	33-67%	15	128								
Lower Juanita	107	33-67%	12	35								
Lower Juanita	102	0-33%	19	38								

† Return interval (years), annual series.
Yellow = All sand (D=5 mm); Olive = Small gravels (D=10 mm); Gray = D_{50}; Purple = D_{90}

Figure 2b. Revised Mobility Summary per this Addendum.

Priority Reach	**NHC Site**	**Sand %**	**D_{50} (mm)**	**D_{90} (mm)**	**Largest Size Class Mobilized at Flow Quantile†**							
					½ of 1.01	1.01	1.5	2	5	10	25	100
Upper Juanita	205	33-67%	19	69								
Upper Juanita	204	0-33%	24	44								
Upper Juanita	203	33-67%	15	33								
Gage 6 Trib	208	0-33%	43	101								
Gage 6 Trib	207	0-33%	33	79								
Cedar Creek	202	0-33%	21	39								
Cedar Creek	104	33-67%	19	32								
Totem Lk Trib	110	0-33%	25	50								
Totem Lk Trib	108	0-33%	15	36								
Lower Juanita	106	33-67%	15	128								
Lower Juanita	107	33-67%	12	35								
Lower Juanita	102	0-33%	19	38								

† Return interval (years), annual series. Olive = Small gravels (D=10 mm); Gray = D_{50}; Purple = $D_{90,}$ Sands will generally mobilize when D50 of gravel is mobilized.

having a more reliable and credible analytical framework is that alternative future scenarios can now be evaluated with respect to their sediment-transport consequences with greater confidence. Although the predictions of sediment-transport modeling must always be treated judiciously, particularly when executed under significant constraints of time and budget, they can provide broadly useful guidance notwithstanding the high imprecision involved. This modification to our original results does not alter the fundamental patterns of sediment mobility previously recognized, but the quantitative results should allow a more functional evaluation of alternatives.

Summary and Recommendations

The County should consider the impact of fine sediment (sand) on gravel mobility in evaluating and refining restoration alternatives for Juanita Creek. As demonstrated by Wilcock et al (2009), the introduction of fine sediment to the stream bed increases gravel mobility with consequences for stream biota (e.g., salmon redd scour and fine sediment intrusion), downstream sedimentation, and channel stability. The pervasive instability of channel banks throughout the Juanita system is surely a significant source of fine sediment to the stream bed and likely causes further instability associated with enhanced bed material mobility. If the County intends to calculate sediment transport rates as part of its restoration design, it may be prudent to consider the application of the two fraction model developed by Wilcock and Crowe (2003) and further documented in the BAGS report (Pitlick et al, 2009). Application of this method would be significantly enhanced by the collection of additional pebble counts that track both gravel sizes and the percentage of the bed occupied by sand for reaches of particular concern.

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APPENDIX C. JUANITA CREEK RECONNAISSANCE AND BIBI REPORT

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2008 Juanita Creek Reconnaissance and B-IBI Report

Hans B. Berge, Senior Ecologist
and
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March 2009

King County

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Introduction

King County is working with the City of Kirkland on a hydrological assessment to better understand the relationship between stormwater, hydrology, and natural conditions in Juanita Creek. In June 2008, we visited many sites throughout the Juanita Creek Basin to investigate habitat conditions for salmonids, channel shape and form, hydromodifications, and to get a sense of how habitat conditions have changed in Juanita Creek since 2001 when King County conducted habitat surveys in Juanita Creek.

Background

The Juanita Creek basin covers approximately 6.6 mi^2 (17.14 km^2) and extends north to Simonds Road, South to 116th Street, east to 132nd NE, and west to 84th Avenue NE (Figure 1). The mainstem of Juanita Creek originates east of Interstate 405, and flows west and south entering the northeast portion of Lake Washington immediately west of Juanita Beach Park into Juanita Bay. Repeated logging and residential and commercial development (especially since the 1960's) has reduced the forest coverage to approximately 12% (Rush et al. 2002). Despite this lack of forest cover, the watershed is still home to many native fish species, including coho (*Oncorhynchus kisutch*), cutthroat (*O. clarki*), and sockeye (*O. nerka*) salmon (Ajwani 1956; Williams et al. 1975; Vanderhoof 2002).

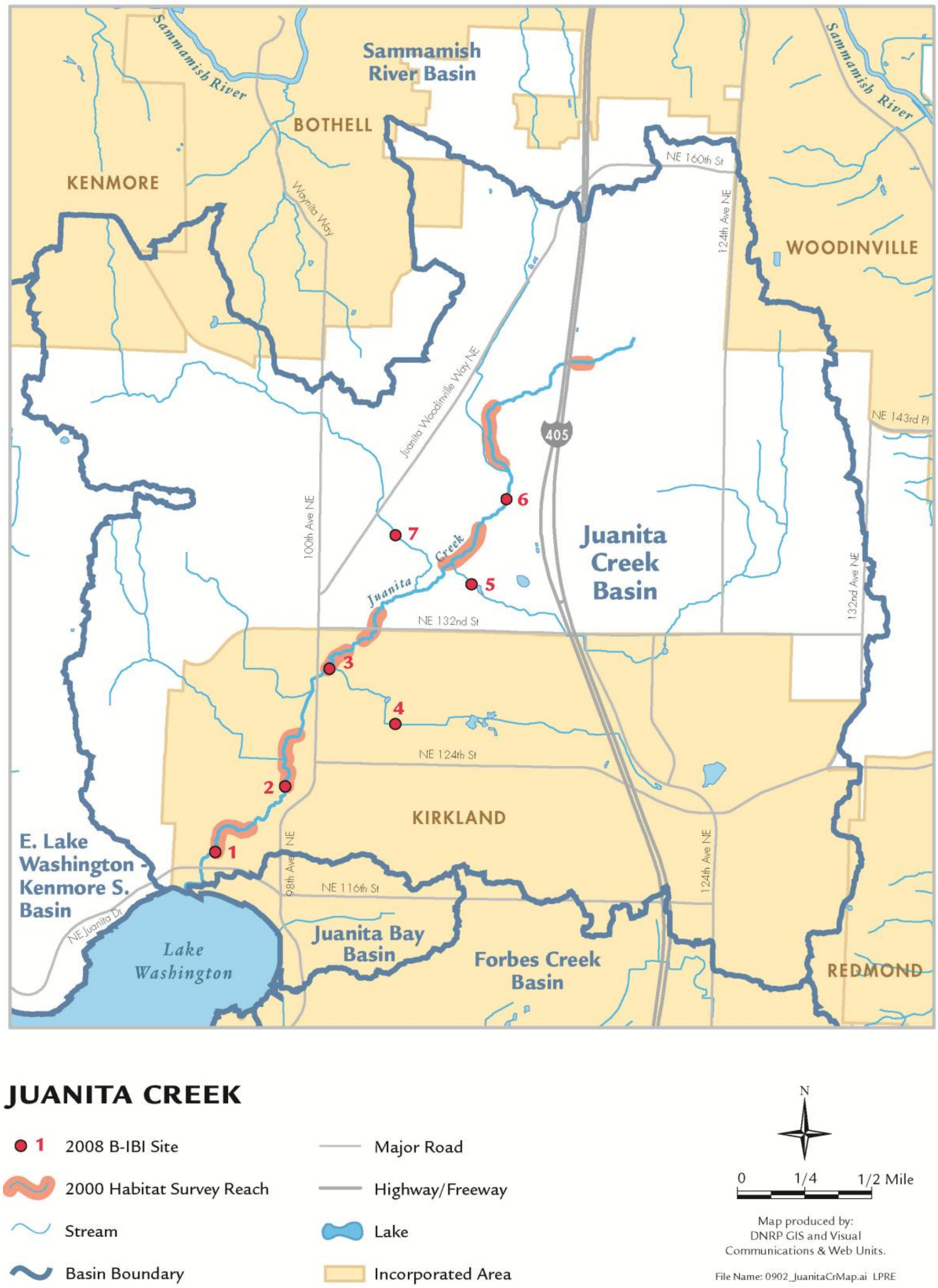

Figure 1. Map of Juanita Creek including King County 2008 B-IBI sampling sites.

Methods
We conducted a rapid assessment of the basin based on publicly accessible points (road crossings, and publicly held lands) to investigate relative habitat quality throughout the watershed and any opportunities for habitat restoration. We observed channel dimensions, wetted width and depth, riparian vegetation, and presence or absence of woody debris.

In September 2008, we collected macroinvertebrate samples from seven sites (Figure 1) in Juanita Creek following the protocols of Karr and Chu (1999). The purpose of macroinvertebrate sampling was to assess the relative benthic index of biotic integrity (B-IBI) of Juanita Creek in comparison to other Puget lowland streams. We compared these data to those collected by the City of Kirkland using the same methods (in 2001-2007) at 3 sites in the same vicinity as our collections in 2008.

Results
Results of the reconnaissance were consistent with the data collected in 2001 (Rush et al. 2002). Specifically, riparian vegetation was poor in general, although a trend of increasing riparian vegetation with distance upstream was evident. Blackberries (*Rubus discolor* and *R. laciniatus*) were common throughout the creek. Reed canary grass (*Phalaris arundinacea*) was common in smaller tributaries, and adjacent to wetlands and stormwater facilities. Coniferous vegetation was largely absent; although at site 7 (Figure 1) there was a relatively healthy second growth riparian zone around the stream. In-channel structure (e.g. large woody debris) and associated pools were severely lacking throughout the stream. In addition, fines (sand and silt) were pervasive in almost all stream reaches as was no obvious point sources.

Results from macroinvertebrate samples collected from Juanita Creek in 2008, include B-IBI ratings of very poor (10-16), poor (18-26), fair (28-36), and good (38-44). Replicates collected in 2008 at sites one through three and site 7, suggest that B-IBI scores can be variable within the same reach and on the same date (Figure 2). None of the sites in Juanita Creek have been classified as excellent (46-50).

Nonetheless, cutthroat fry were observed in the lower half of the mainstem while trout fry were present further upstream just below the confluence of the three head-water tributaries. Fingerlings were found 0.5-miles upstream of the confluence in the northwestern head-water tributary. However the two lowest right-bank tributaries identified in Figure 1, plunge to the mainstem with greater than 3-feet elevation drop from protruding corrugated metal pipe culverts.

Channel geometries were predominantly trapezoidal in shape for the mainstem, with tributaries to Juanita creek ranging from trapezoidal, rectangular with heavy bank armoring, to fully piped with intermittent day-lighting.

During the reconnaissance done in late June 2008, summer base flows (3.5-cfs) were near their typical minimums (~ 3-cfs). In all but one of the headwater catchments, flow rates were estimated to be on the order of 0.10-cfs. Negligible flows were found in the

headwaters stream channel connecting Totem Lake to a large wetland area. Coincidently, the majority of commercial land use is also located in this catchment.

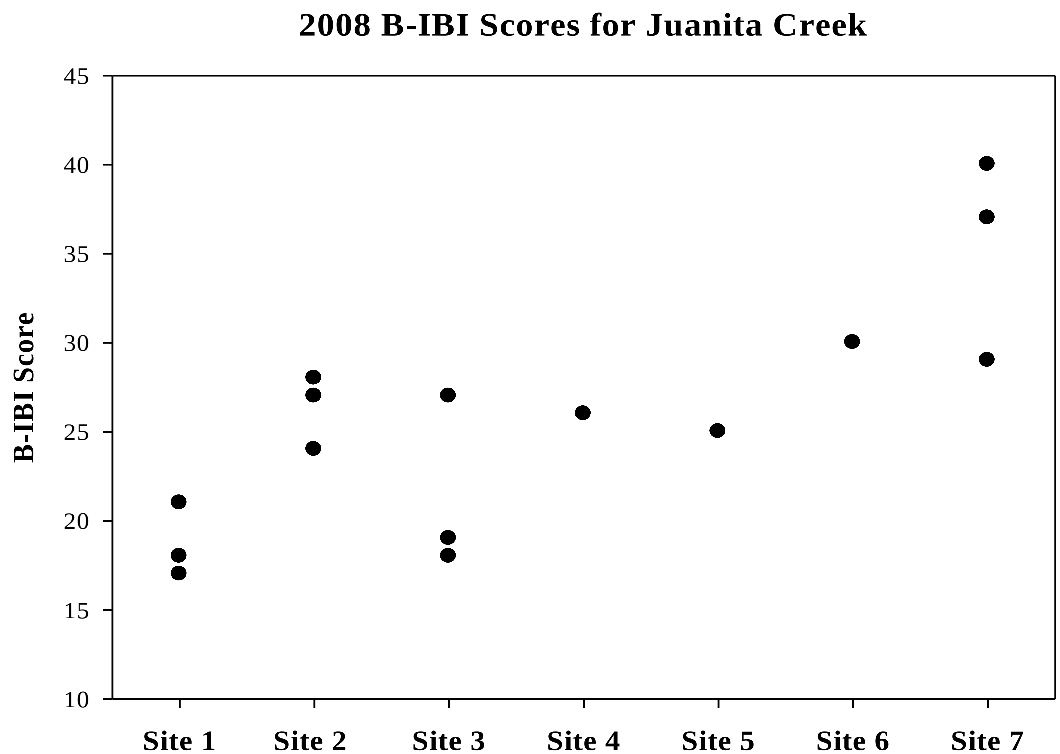

Figure 2. B-IBI scores for seven sites sampled in Juanita Creek in September 2008 by King County.

Indices other than B-IBI may be useful in describing diversity and stress associated with anthropogenic change. A community dominated by one or two species, is less diverse than one with several different species with relatively even proportions. Evenness is a measure of the abundance of different species making up the richness for a particular area, or sample in our case. Diversity indices are a combination of evenness and richness. The Hilsenhoff diversity index generally increases with increasing ecosystem stress (Hilsenhoff 1988), while the Shannon diversity index generally decreases in values with increasing stress. Simpson's D in comparison, simply refers to equitability of families represented, and a score of 0 represents infinite diversity and a score of 1 represents no diversity. In Juanita Creek, the diversity indices show much more variability than the B-IBI scores (Table 1).

Table 1. Diversity metrics for macroinvertebrate samples collected in Juanita Creek in 2008. B-IBI scores represent mean values (three samples) at sites 1-3, and site 7.

Location	Number of Samples	Evenness	Shannon Diversity	Simpson's D	Hilsenhoff Biotic Index	B-IBI Score (10 - 50)
Site 1	3	0.7750	1.9140	0.8064	5.5928	18. 7
Site 2	3	0.7180	2.1237	0.8128	5.4166	26.3
Site 3	3	0.6123	1.5713	0.6722	6.1759	21.3
Site 4	1	0.4560	1.1330	0.5026	7.3955	26
Site 5	1	0.2720	0.5970	0.2506	4.2823	25
Site 6	1	0.6280	1.8160	0.7789	5.9481	30
Site 7	3	0.7600	2.0570	0.7832	5.3671	35.3

B-IBI scores calculated from 2001 to 2008 are useful in understanding the ecological condition of Juanita Creek (Table 2). Despite some interannual variability at sampling sites (Figure 3), the overall trend is that sites in Juanita Creek are generally poor (Table 2). In 2008 samples, site 3 appears to be much higher than in previous years, and may be due to interannual variability. This site is at a park, and is surrounded by established deciduous riparian zone and may be improving over time. In addition, site 4 is above a fish barrier.

Table 2. B-IBI scores from 2001 through 2008 in Juanita Creek. 2001-2007 data are from the City of Kirkland (Betsy Adams, Kirkland, unpublished data) at locations that correspond to (sites 1 and 3), and are nearby (site 2) 2008 King County sampling sites. ***NS*** = not surveyed that year. Each value represents one sample collected at that site for years 2001-2007, and three samples averaged for 2008.

Location	2001	2002	2003	2004	2005	2006	2007	2008
Site 1	16	16	18	***NS***	18	20	19	19
Site 2	14	14	22	***NS***	18	20	19	21
Site 3	16	16	***NS***	10	16	20	15	26

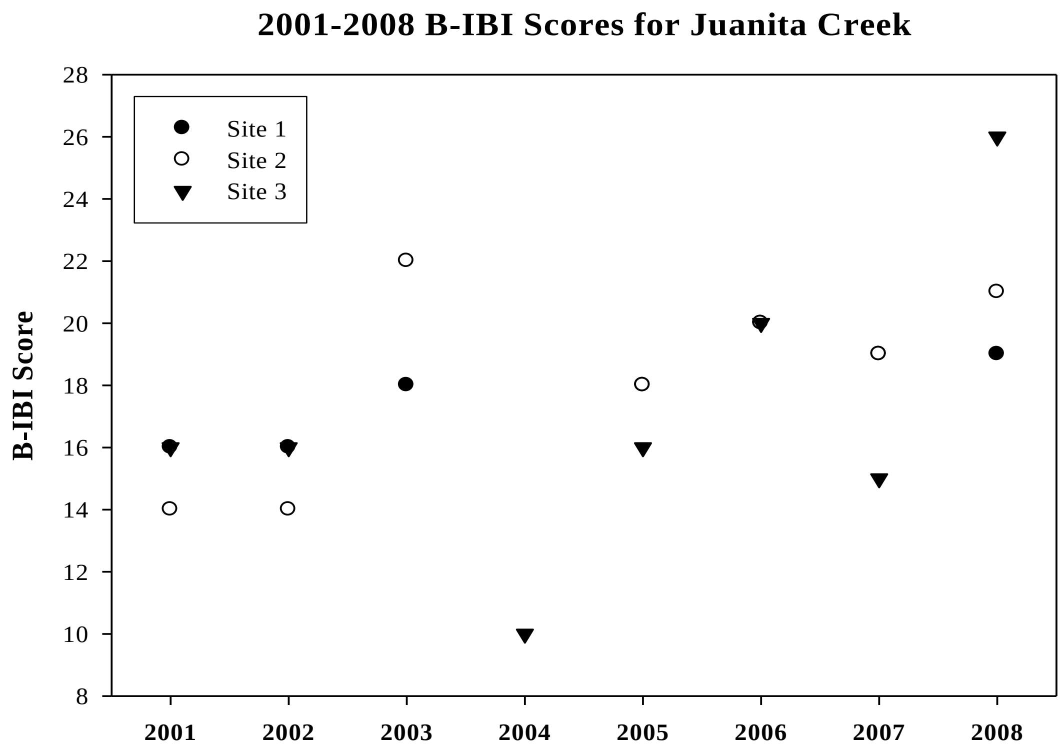

Figure 3. B-IBI scores from 2001 through 2008 in Juanita Creek. 2001-2007 data are from the City of Kirkland (Betsy Adams, Kirkland, unpublished data). Each value represents one sample collected at that site for years 2001-2007, and three samples averaged for 2008.

Discussion

These findings are consistent with the work of Rush et al. (2002) in summarizing the condition of aquatic habitats in Juanita Creek as lacking the complex habitat structure necessary to support and sustain salmon. In addition, supporting work with a FIBI (fish index of biotic integrity) confirmed the same result, that Juanita Creek provides relatively poor diversity of fish species (Matzen and Berge 2008). Despite poor physical habitat (Rush et al. 2002), biological integrity (Matzen and Berge 2008), and water quality (Rush et al. 2002), Juanita Creek still provides fish habitat.

Like other urbanized streams in the Puget Sound lowlands, Juanita Creek is lacking in riparian corridor, channel stability, large woody debris, and riparian vegetation. Fine sediments were located throughout the stream, reducing the amount of productive spawning habitats sites. Efforts focused on rehabilitation in Juanita Creek need to address reestablishing natural processes that create and maintain fish habitat. Modifying the current flow regime to create one that is more supportive of these natural processes would form an important element in any plan to significantly improve habitat in Juanita Creek. The work being done now to measure and model flow and water quality

relationships will better inform management plans to improve Juanita Creek's habitat for native species.

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APPENDIX D. JUANITA CREEK BMP POLLUTANT REMOVAL EFFICIENCIES

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Appendix D
Juanita Creek Stormwater Retrofit
Definition of BMP/Facility Pollutant Removal Efficiencies
Ecology Grant: G0800618

August 2012

King County
Department of Natural Resources and Parks
Water and Land Resources Division
Stormwater Services Section
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201 South Jackson Street, Suite 600
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Appendix D
Juanita Creek Stormwater Retrofit Definition of BMP/Facility Pollutant Removal Efficiencies

Ecology Grant: G0800618

Prepared for:
Stormwater Management Implementation Grant Program
Washington State Department of Ecology
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Bellevue, Washington

Prepared by:
King County Science and Stormwater Services Sections
King County Water and Land Resources Division
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Water and Land Resources Division

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1.0 SUMMARY

The sub-task presented for this section was estimation of removal efficiencies for several pollutants by two BMP*/Facility designs. Removal efficiency is defined here as efficiency in decreasing concentration from facility inlet to outlet, excluding bypass. Efficiency varies depending on a number of factors including but not limited to facility type – which defines which unit processes† are in play – facility design, implementation, and maintenance, flow rate, influent concentrations of individual pollutants, particle size and density‡ distributions, relative concentrations all pollutants, and for some pollutants, redox conditions. Removal efficiencies would be best represented by ranges and uncertainties, but this would be a large effort in and of itself, some uncertainties are unquantifiable, and the Juanita model requires single values.

Discussion of reported pollutant-removal unit processes and sources of uncertainty about reported removal efficiency values constitutes a major portion of this exercise; the latter has evolved to the majority. The main findings of this review are: that there is a great deal of uncertainty in published pollutant concentration values – much of it unquantifiable as a result of undocumented sampling and analysis methods and some suspect bias in same; that improvement is still needed in sample collection methodology; that equations used for predicting pollutant removal may need to be selected according to location-specific parameters; and that it is imperative to get representative local regional stormwater pollutant profiles – even down to the level of specific land uses – especially before basin-wide retrofit programs, but not ignoring considerations for treatment facility design in general, and that profiles should include particle size and density and/or settling rate (with test temperature) distributions. Caveats notwithstanding, for the modeling exercise at hand, 'true' percent removal rates are not critical for this exercise, because we are modeling different scenarios relative to each other (Wilgus 2011b).

1.1 Caveats

1.1.1 General

All removal efficiencies should be viewed with these caveats: The pollutant removal values used in this modeling exercise are applicable solely to this exercise, and should not be considered applicable in any other context. They are based on a necessarily time-limited literature search

* Best management practice. Some agencies refer to engineered treatment facilities as BMPs. King County defines engineered stormwater treatment structures as facilities, and non-engineered treatment and practices as BMPs. WA Ecology has recently made a distinction between "bioretention BMPs" and "rain gardens" as the former being engineered and the latter not. King County does not currently allow rain gardens as water quality treatment facilities, and does not have design criteria for that purpose. As such, these are being referred to as BMPs for this modeling exercise, even though specific design criteria are given here.

† Including but not limited to removal processes, e.g. sorption, chemical speciation and combination, ion exchange, chelation, flocculation, filtration, bacterial die-off (several processes involved in this alone), plant uptake, etc; and pollutant release processes, e.g. re-suspension, ion-exchange (e.g. road salt), change in redox conditions, and in the case of bacteria, growth exceeding die-off). First heard use of this term attributed to Eric Strecker (Geosyntech Consultants, OR).

‡ or specific gravity (SG) – relatively interchangeable with density insofar as SG is the ratio of the density of a substance (in this case particles) to that of another – in this case water at specified pressure and temperature. At standard conditions, the density of water is 1, so the ratio (SG) is numerically the same as the density of the substance, but SG is unitless; e.g., with a density of 2.65 g/cm^3, pure quartz has a SG of 2.65.

and survey, and cannot be claimed to represent a scientific meta-analysis. A major data source is the International Stormwater BMP Database (ISWBMPDB 2011a), which is a compendium of stormwater data, at least some if not much of which has been reported elsewhere. Minton (2009) provides a quite comprehensive and detailed assessment of the database from a user's point of view.

As put by Gossett et al. (2004), "various monitoring programs have differing project goals and objectives, differing mandates from regulatory agencies, differing sampling designs, and differing laboratory analytical methods", all of which clouds accuracy and hinders data comparability. The same can be said for independent research monitoring efforts, whether compiled in the database or found separately. To the best of our knowledge these data have not undergone independent third-party assessment of facility monitoring settings, protocols, and setups, data quality, and verification of as-built design and maintenance operations* as would be required for scientific meta-analysis, or for evaluation as to comparability to King County or WA Ecology design and maintenance standards. Moreover, there is very little data in the database from western Washington.

There is a general lack of and/or inconstancy in reporting details on facility configuration and maintenance, and monitoring design, implementation, and verification – all of which can affect reported pollutant concentrations, and by extension, removal efficiencies. Some data from Washington were voluntarily retracted by the providing agency for these reasons, and because of lack of supporting evidence regarding data representativeness and credibility. For the modeling exercise at hand, for conventional wet ponds, total suspended solids (TSS) removal is based on regulatory design manual assumptions, and total phosphorus (TP) removal is extrapolated from manual assumptions. Neither of these is based on the current literature survey, however, the same issues just raised are as applicable to the presumptive basis for TSS and TP removal as for any other pollutants.

During the course of this investigation, some pollutant removal rate estimates changed as a consequence of information in more recently found literature, and/or reassessment of estimation assumptions or methodology. This leaves an apparent disconnect between 'final' pollutant-removal efficiency estimates in this section, and those used for modeling, which relied on the earlier estimates. However, it seems fairly safe to say that whatever values are apparent today will also differ in the future as more data are collected and as more accurate and consistent sampling and analysis protocols are implemented, and representative regional data subsets become more prevalent.

1.1.2 Percent Removal

It is important to note that the value of percent removal as a measure of facility or BMP utility is a subject ongoing debate in the stormwater community. The arguments against use of percent removal are probably best summarized in a 'Frequently Asked Questions' (FAQ) white paper on percent removal, by Wright Water Engineers & Geosyntec Consultants (2007). The first listed argument notes, "In almost all cases, higher influent pollutant concentrations into functioning BMPs result in reporting of higher pollutant removals than those with cleaner influent".

* In this author's experience and view from two agencies, it is not uncommon for facilities to be installed incorrectly, not according to design, materially defective, and/or not maintained. In addition, this author has seen a number of monitoring setups that could not be justified as capable of delivering representative samples or flow data.

Not mentioned in the FAQ on percent removal are particle size and density distributions, which directly affect efficiency of removal by settling, and result in stratification which can result in unpredictable variable monitoring bias, which can affect apparent removal efficiency.

The last listed argument in the FAQ on percent removal is that reported percent removal often excludes bypass; and compounding this problem there is lack of consistency in how much bypass is allowed if reported at all. Li et al. (2008) support this, noting, "The efficiency calculation is often based only on the treated portion, and bypassed pollutant mass may not be considered, which overestimates pollutant reduction rate". WA Ecology's TAPE[*] program (Ecology 2011) requires "treating at least 91 percent of the total ~~annual~~[‡] runoff volume", "The proponent is not required to measure water quality parameters in the bypass flow", and removal efficiency is calculated on a concentration or loading basis from storm influent and effluent data. Regression analysis is required for pollutant removal as a function of flow rate to assess pollutant removal at the design flow rate (Ecology 2011). Ecology's stormwater manual says, "The goal also applies on an average annual[‡] basis to the entire annual[‡] discharge volume (treated plus bypassed) (Bakeman et al. 2012; O'Brien et al. 2005). In the absence of these kinds of specifics for removal efficiency values from other sources, we are assuming bypass is not generally included in pollutant concentration values, but we do not know how much bypass is allowed, and this uncertainty clouds the meaning and comparability of percent removal values.

[*] Guidance for Evaluating Emerging Stormwater Treatment Technologies, Technology Assessment Protocol - Ecology (TAPE)

[†] deliberate strikeout. *annual* is part of the quote, but is being deleted in this context throughout TAPE (Howie 2012)Howie DC, 2012. Personal Communication, Phone discussion regarding difference between TAPE and Ecology's Stormwater Management Manual for Western Washington, with regard to % removal and volume to be treated ed.

[‡] Unknown whether WA Ecology will change this as it is in TAPE (prior footnote)

2.0 Water Quality BMP/Facility Designs, Scenarios, and Pollutants Modeled

Two water quality BMP/Facility designs have been described for this modeling effort:

- Regional level-2 stacked detention over wet ponds (live storage over dead storage): The dead storage volume is presumed to be a permanent (non-leaky) wet pool. Some at least peripheral macrophyte vegetation is likely in actual implementation design, but is not included in the model.
- Rain gardens as low impact development (LID) best management practices (BMPs): Water leaves by two routes, primarily by infiltration to groundwater. All infiltrate to groundwater is assumed to reach a local stream. If and when there is any excess resulting in surface effluent, it is routed to a regional wet pond. Macrophyte vegetation is certain in actual implementation design, but is not included in the model.

Several scenarios are being modeled, including at least two with varying percent of catchment area served only by rain gardens, and one with 80% of the catchment area draining to rain gardens and 20% draining directly to regional wet ponds, which also get any surface discharge from the rain gardens.

2.1 Design Details

2.1.1 In-Common Design and Modeling Assumptions and Considerations

There is assumed to be no re-suspension of sediments Burkey (2011). Plant uptake of water and transpiration are not factored, but pond evaporation is. Modeling assumes no macrophytes, but includes algae (Burkey 2012). Episodic and/or cyclic low dissolved oxygen (hypoxia to virtually anoxia) may occur; the model assumes full mixing with no stratification when this occurs (Burkey 2011). This affects nutrient speciation and results in the facilities being both sinks and/or sources for particular nutrient species, depending on state of oxygenation.

2.1.2 Regional Wet Ponds

The wet ponds are assumed to be constructed according to the 2009 King County Surface Water Design Manual. Design is level-2 flow-control stacked detention over wet ponds (live storage over dead storage, with wet pool capacity of Vb/Vr[*] = 3). They are assumed to be non-leaky i.e. no infiltration is factored (Burkey 2011), and to contain no macrophytic vegetation, although in reality incidental vegetation is likely. Pollutant-removal efficiency is not adjusted downward for the pre-treated portion of influent routed from rain garden ponds[†] (Burkey 2012).

[*] The ratio of the pond volume Vb to the volume of runoff from the mean annual storm Vr, where Vr = mean annual storm depth x runoff coefficient. King County's methodology for calculating Vr and Vb is given in its 2009 Surface Water Design Manual, pages 6-70 – 6-72.

[†] See Section 1.1.2 (Percent Removal)

2.1.3 Rain Gardens

These facilities have been described as ponding rain gardens, nominally 33 x 11 feet*, with a nominal one foot deep water storage volume yielding 375 cubic feet (363 according to LxWxD), not accounting for side slopes or including any void space in the underlying media; Vb/Vr* = 7 (Wilgus 2011a). These are functionally infiltration ponds with no underdrain or bottom outlet. There is no surface effluent discharge until water reaches the 1 foot elevation level. As long as there is inflow, continuous infiltration is expected both prior to and during surface effluent flow. When there is surface flow all the way through the pond, i.e. for a full pond when inflow exceeds infiltration capacity (during annual peak flow (Wilgus 2012)), travel time from inlet to outlet is calculated to be 7 hours . All overflow, i.e. that which does not infiltrate, is routed to a regional wet pond (Burkey 2011).

Infiltration is limited by underlying soil, and is assumed to be 0.15 in/hr in inherently low-infiltration areas and 3.0 in/hr in high infiltration areas (*ibid*). According to US EPA (1983), Seattle average annual precipitation is 21.5 hours in duration, and time between storm midpoints is 101 hours; this leaves an average antecedent dry period of 79.5 hours. At 0.15 inches infiltration per hour, a filled pond will drain down 11.925 inches – functionally draining its 1 foot depth, certainly after minimal sediment has built up. At an infiltration rate of 3 inches per hour, only four hours are required to drain a full pond, so surface discharge from one of these to a regional pond is not expected at all. Even at an infiltration rate of 0.15 inches per hour, at Vb/Vr = 7, the pond will rarely fill; but when it does, drainage is still primarily through infiltration, and secondarily by surface discharge to a regional wet pond.

After any pollutant removal by the rain garden bed media, 100% of infiltrated water is assumed to reach a surface stream; there is assumed to be no potential for infiltration to an aquifer deeper than that discharging a stream (Burkey 2011). The model incorporates infiltrate dilution by mixing with groundwater, but no pollutant removal is assumed in underground flow; i.e., after mixing, there is a direct hydraulic connection to an adjacent stream (Burkey 2011, 2012).

Rain garden media depth and composition are not required for the model (Burkey 2011). The rain gardens are vegetated (Wilgus 2011a). However the potential effect of vegetation on infiltration rate is not factored; only evaporation is modeled (Burkey 2011).

For basin implementation, for survival and functionality, planting should be a mixture of bushes, rushes, reeds, sedges, and grasses. In practice we should consult with an in-house ecologist or landscape horticulturalist when selecting plants. However, these are not considerations for the model.

A conventional wet pond has at least one foot of sediment storage in addition to pond wet storage. In contrast, the wet pond pools themselves are only one foot deep. Maintenance will need to take this into consideration, as well as the fact that plants will inherently be disturbed or removed as collateral damage during sediment removal.

* This needs to be adjusted and calculated for both the top and bottom of the rain garden basin, depending on side slope, so as to maintain a volume of 375 cubic feet in this case (see following citation in text), or whatever rain garden pond volume is appropriate for site-specific runoff volume.

2.2 Pollutants in the Model

- Total suspended solids (TSS)
- Copper – solid and dissolved (Cu-solid, Cu-diss)
- Phosphorus – total and soluble reactive ((SRP) aka orthophosphate (OP))
- Nitrate (NO_3^-), ammonia/ammonium (NH_3/NH_4^+), total Nitrogen
- Fecal coliform bacteria (FC)

3.0 Pollutant Removal Processes

3.1 In-Common Conditions and Processes

When either a wet pond or a rain garden pond is full, surface water discharge will occur. For a wet pond, discharge is at a controlled design flow rate up to bypass conditions. Surface discharge will be relatively rare in the rain garden pools with Vb/Vr = 7 and infiltration rate = 0.15 inches per hour; and should rarely if ever occur in the rain garden ponds where infiltration rate = 3 inches per hour. During overflow for both the wet pond and the low infiltration rain garden pond, as flow velocity increases, travel time decreases and flow velocity increases beyond design rate, which should result in lower than design percent pollutant removal rates.

With regard to surface water discharge, particle settling is the primary pollutant removal process. Wet ponds will experience periods of both quiescent and dynamic settling, but the shallow infiltrative rain garden ponds are assumed experience only dynamic settling[*]. To the extent that dissolved materials may complex with each other to form precipitates, or sorb to suspended solids, some dissolved materials can be removed by settling.

Hypoxia to anoxia may occur in a wet pool or a rain garden. In a wet pool either can occur in bottom sediment and at the sediment / water column boundary layer or even higher under stagnant conditions or with algal blooms. In a rain garden it may occur in media overlying low infiltration soils, e.g. clay or glacial till, or even in a deep media under-layer, particularly under prolonged saturated conditions. Causes of hypoxia/anoxia include biochemical oxygen demand, sediment oxygen demand, respiration by bacteria, fungi, nematodes, etc., and nighttime respiration by algae (and by macrophytes in the real world). Oxygenating factors include air to water exchange, and oxygen generation by photosynthetic macrophytes and algae during sunlight hours. Fluctuating dissolved oxygen levels would affect water column and pore water chemistry, and likewise boundary layer chemical interactions between the soil mix or bottom sediment and the water column, ultimately affecting some pollutants' speciation, removal, and release.

Stormwater ponds evolve over time, with changes in sediment and vegetation. There is evidence that that can affect some pollutant removal rates over time (Lavieille 2005; Pettersson et al. 2007).

3.2 Regional Wet Pond Processes

The regional wet ponds are being modeled hydraulically as non-infiltrative, each holding a permanent wet pool, and absent vegetation. Therefore, water can only leave by two routes, evaporation and surface outflow (three routes if bypass is considered separately).

Pollutant removal in the ponds is predominantly by settling, sometimes also referred to by others as sedimentation. Precipitation of solutes to solids may contribute. US EPA (1986) factors both quiescent and dynamic settling in their wet pool TSS removal efficiency calculation. According to US EPA (1983), in this region average storm duration is 21.5 hours during which settling is presumed to be dynamic, and average dry period between storms is 79.5 hours during which settling is presumed to be quiescent. Anecdotally, we know that for this region during the wet

[*] The rationale for this is discussed in Section 3.3, Rain Garden Processes.

season storms are often longer and dry periods between storms are often shorter. Conversely, during the dry season storms are often shorter and dry periods between storms longer.

To the extent that some dissolved pollutants may react to form solid precipitates, and/or sorb to mineral and/or organic suspended solids, there will be some dissolved pollutant removal ultimately by settling. However, this route is complex and dependent on presence and concentration of multiple constituents, and therefore not assessed here except as a source of variability and hence uncertainty. We should keep in mind that not all TSS is natural mineral; some is organic solids, e.g. tire wear and plastic debris, vegetative and faunal debris and detritus, fecal matter, etc., none of which will settle as fast as mineral solids; and for any given particle size, anthropogenic-source metals will settle faster than silica-based mineral. While removal of TSS may be modeled (albeit with some complexities and uncertainties there too), ultimately for this exercise design-manual assumptions are used; and they are extrapolated for pond total phosphorus removal. All other pollutant removal efficiencies are based on published empirical rates.

Of the pollutants being tracked in the model, the only pollutant subject to evaporation might be ammonia; but this is very unlikely, since most will probably be dissolved ammonium ion NH_4^+ at stormwater pH (< 7.0)[*]. To the extent that hypoxia or anoxia may occur in sediment pore water and/or at the sediment-water column boundary layer, and to the extent that organic matter is available as an electron donor, some nitrate may be reduced to N_2 (and to a lesser degree N_2O) gas, but this is complex, site- and condition- specific, and not modeled.

3.3 Rain Garden Processes

There are two pollutant-removal routes in the rain gardens – infiltration through media, and particulate settling in the pond above the media. They cannot be discussed separately because there is interaction between the pond and the infiltration bed, and because the pond is ephemeral.

The rain gardens contain vegetation, but this is not being factored in the model with regard to particle settling. Because rain garden ponds differ in design and operation from conventional wet ponds, without analysis or empirical evidence, we can't assume the same pollutant removal rates. Given that Vb/Vr = 7, wet-pool depth = 1 foot, and minimum infiltration rate = 0.15 inches per hour, except for multiple back-to-back and/or large storms, the rain gardens are expected to drain completely between storms, so the pond pollutant-removal process resembles a dry detention pond more than a wet pond, although there is no bottom outlet drain as with a dry pond. According to Minton (2011), "stormwater treatment systems that are dry between storms experience only the dynamic settling process. Minton notes that turbulence decreases settling efficiency in wet ponds[†], but does not address it when discussing dynamic settling efficiency, which he attributes to solely to hydraulic loading rate. Papa et al. (1999) do factor turbulence, adapting the dynamic settling efficiency equation from MOEE (1994), which also factors turbulence. With dynamic flow in this system, turbulence is expected. Turbulence factor[‡] is

[*] At pH 7, $[NH_3] / [NH_4^+] = 0.0056$. At pH 6, the ratio is $5.6234*10^{-4}$

[†] US EPA (1986) comments with regard to wet ponds which experience both quiescent and dynamic settling, "the quiescent process has a lesser effectiveness for the removal of particles with the higher settling velocities compared with dynamic removals", but add that "The efficiency and importance of the quiescent process is reflected by its significantly higher effectiveness in removing the slower settling fractions".

[‡] The name is counterintuitive, as is its other commonly given name, *short circuiting factor*. It is alternatively defined as the *number of hypothetical basins in series* by Pitt et al. (2005).

inversely proportional to turbulence (MOEE 1994); i.e., the higher the number the better, although beyond a factor of 3 it has little effect on outcome, as indicated in Figure 1.

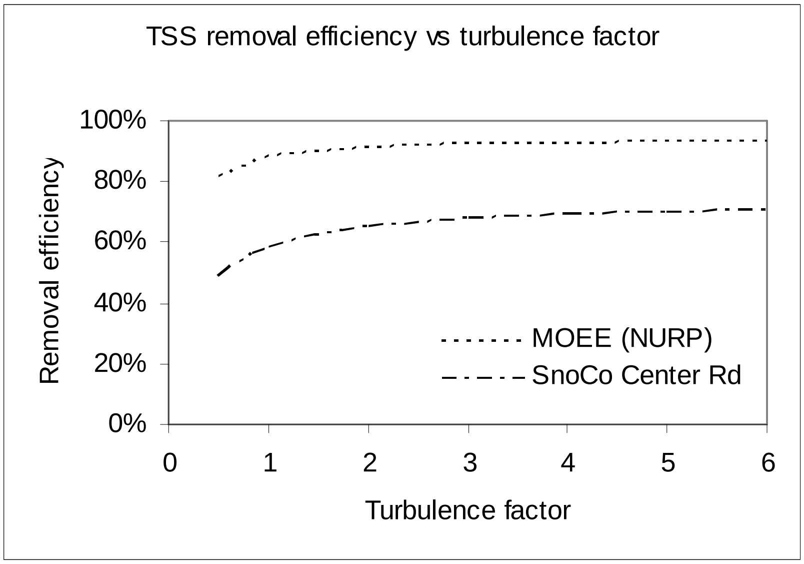

Figure 1. TSS removal efficiency as a function of turbulence factor

This graph was generated by calculating percent TSS removal efficiency according to the Papa et al. (1999) equation for removal efficiency for dry ponds, with a series of turbulence factors, using the MOEE (1994) particle settling rates and Snohomish County* (Herrmann 2012) particle size distributions, 1 ft pond depth, 0.15 in/hr infiltration, and maximum drawdown time = 12 inches / 0.15 inches/hr = 80 hrs. TSS removal efficiency is covered in detail in Section 2.1.1. MOEE and Papa et al. (1999) use a turbulence factor of 3 representing average or good dynamic settling conditions; MOEE assigns n = 5 near quiescent settling conditions.

Water not lost to evaporation is infiltrated until inflow exceeds the infiltration rate, the rain garden pond fills, and discharge to surface conveyance occurs; but infiltration will continue concurrently with surface discharge, to a degree depending on soil type and saturation. There is no surface effluent discharge from the rain garden ponds until water reaches the 1 ft elevation level. When there is flow all the way through the pond, i.e. for an already full pond, travel time from inlet to outlet is calculated to be 7 hours during annual peak flow (Wilgus 2012).

Assumptions regarding rain garden pollutant removal other than TSS should be tempered with the knowledge that compared to a simple retention pond, rain gardens are much more complex systems with respect to water, soil, plant, and microbial interactions; they are subject to wet/dry cycles, which require selection of plants that can tolerate both prolonged inundation and dry periods; that depending on design and climatic conditions, the rain garden media may experience conditions ranging from oxic to hypoxic to anoxic to varying degrees and for varying amounts of time – which will affect plant selection and will affect chemistry and hence some of the pollutant-removal interactions and efficiencies; that pollutant removal mechanism matrixes and efficiencies differ between the infiltrative route and the pond to surface flow route; that surface scouring could occur during high runoff events; and that maintenance in the way of sediment removal may be needed frequently because of the shallow depth of the pond component of a rain garden, and that will disturb established vegetation.

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* Provisional data, subject to change

Metals removal for the pond component is functionally the same as that for a wet pond, except particle settling is expected to be exclusively dynamic. Metal solids are part of TSS, and will settle according to the same factors, i.e. size, density, shape, roughness, and porosity. Some dissolved metals may sorb to mineral and/organic particulate solids, or may combine with some anions in solution, forming settleable precipitates. Remaining dissolved metals may go the infiltration route, or will leave in effluent, and move on to a regional wet pond.

Minton (2011) says that dense planting, as with grass in a biofiltration swale or filter strip, will minimize contact between stormwater and soil; and this will minimize the ability of the facility to remove nutrients from stormwater, since plants take up nutrients only through their roots. It will also affect the apportioning of infiltration to ground vs. discharge to surface water. On the other hand, Minton does not define 'dense' quantitatively, and neither he nor others indicate numeric nutrient removal rates correlated to specific plantings. In any event, the rain garden design under consideration will not have dense planting in comparison to e.g. to a grass swale.

Minton (*ibid*) says that experience with wetlands indicates that even with harvest of foliage, plants account for little in the way of nutrient or metals removal, and that most metals removal is by sorption to soils and plant roots. This is contrasted by (Davis et al. 2006), who say, "Analysis of the fate of nutrients in bioretention suggests that accumulation of phosphorus and nitrogen may be controlled by carefully managing growing and harvesting vegetation". Minton says of bioretention vegetation, "the dominant role of the plants may be indirect. They provide an ecosystem conducive to microbial growth"; and then points the limitation in that eventually microbial die-off and growth will equilibrate, suggesting that some initial pollutant removal rates will not be sustained over time. However, under the right conditions, steady-state microbial activity, not dependent on population growth, could e.g. continue to meditate denitrification (nitrate reduction to $N_{2\ (gas)}$). Under other conditions, other nitrogen species may be transformed to nitrate and exported as such (Davis et al. 2006).

In general in our region, nitrate is of concern for discharge to groundwater and phosphorus is of concern for discharge to surface water. Nitrate is highly soluble and has very high mobility in soil, although the relationships between nitrate and other nitrogenous compound speciation (ammonia/ammonium, nitrite, nitrate, and $N_{2\ (gas)}$) are complex, and are affected by redox conditions, microbial activity, and plant uptake and recycling.

To the extent that nitrate may be of concern, i.e. that the concentration might be in a range approaching the state ground water quality standard of 10 mg/L[*], vegetation should be selected in concert with soil characteristics to minimize infiltration risk to groundwater. This would be a balancing act. We want enough infiltration to get nutrients to plant roots for uptake by plants, and we want infiltration for flow control, but we don't want high nitrate concentrations infiltrating to groundwater. In practice, high nitrate to groundwater from stormwater may not be a problem. From the International Stormwater BMP Database, for all BMP facilities combined, the highest 95% confidence limit effluent values were 1.34 and 1.23 mg (NO_3^-)-N / L respectively for 2008 and 2011 (ISWBMPDB 2008, 2011c), compared to the 10 mg/L groundwater quality standard. Very limited site characterization monitoring[†] for King County's

[*] Chapter 173-200 WAC (Washington Administrative Code)

[†] For commercial, high density residential, and low density residential runoff, respective sample sizes were 7, 7, and 11 runoff events sampled (October 2009 – September 2010).

NPDES current stormwater permit yielded a maximum value of 1.8 mg $(NO_2^-)+(NO_3^-)$* from low density residential land use, with a median of 0.7 mg/L.

Soluble phosphate may combine with some cations to form relatively insoluble complexes, e.g. hydroxyapatite† (which is not very mobile in soil as it becomes part of total solids) and other precipitates with differing solubilities‡. However, some species of precipitated phosphorus are subject to re-release into the water column, and plant uptake and nutrient recycling are also factors.

In short, nutrient speciation is complex, and has implications for removal rates. Concerning uptake by plants, we need to be mindful that any nutrient removal by this route is only effective to the extent that the nutrients are sequestered in organic molecules in plant tissues, and to the extent that vegetation is harvested. Otherwise, e.g. mowed or dead plants and fallen leaves may recycle nutrients with possible release of nutrients into water passing through the facility. As noted above, there the degree of nutrient removal feasible by plant harvesting is not settled.

If native soil infiltrates too rapidly and does not function as a treatment layer per KC SWDM requirements, then a treatment layer is required. The infiltration rate limit is 2.4 inches per hour over “Critical Aquifer Recharge Areas” (CARAs), and 9 inches per hour over non- CARAs. As noted previously, the modeling exercise at hand assumes a native soil infiltration rate of 0.15 or 3.0 inches per hour, depending on soil type. In practice then, planning for a basin over a CARA should model limiting infiltration to 2.4 inches per hour.

While infiltration safeguards for groundwater, infiltration media, and groundwater discharge to streams is on our radar for review, current infiltration consideration in the King County Surface Water Design Manual (SWDM) is simply a presumptive approach that considers groundwater protected if facility design and/or soil treatment layer criteria are met. While it is reasonable to assume *basic*§ treatment for effluent from e.g. a wet pond discharged to an infiltration facility, whether that level of treatment is actually protective of groundwater or not is another question. We consider a very limited suite of pollutants in stormwater as indicators of overall treatment, but with little to no real idea of remaining risk from a large universe of potential pollutants of concern.

This is true for surface water discharges as well as groundwater. In addition, groundwater quality standards differ from surface water quality standards. That said, infiltrated pollutants that get to groundwater may still wind up in surface water discharge to streams (Minton 2011). For purposes of this model, 100% of infiltrate is assumed to reach the surface receiving water, albeit with dilution by mixing.

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* NO_2^- is generally a small fraction relative to NO_3^-

† Synonymous with hydroxylapatite; a complex of phosphate, calcium, and hydroxide ions.

‡ e.g., $K_{sp\ (25\ deg\ C)}$ for one form of hydroxyapatite $Ca_5(PO_4)_3OH = 1.0 \times 10^{-36}$; fluorapatite $Ca_5(PO_4)_3F$ 1.0×10^{-60} (note effect on solubility from simple substitution of F for OH compared to hydroxyapatite); $AlPO_4 = 6.3 \times 10^{-19}$; $Ca_3(PO_4)_2 = 1 \times 10^{-26}$; $FePO_4 = 1.3 \times 10^{-22}$; $Zn_3(PO_4)_2 = 9.0 \times 10^{-33}$. From Selected Solubility Products and Formation Constants at 25°C. http://www.csudh.edu/oliver/chemdata/data-ksp.htm. Although not stated, these are likely to be dissociation products for pure water, and will vary under differing ionic strength conditions. Solubility decreases as temperature decreases, so these values should be viewed more or less as relative, with the following additional caveat: "Unfortunately, there is no simple way to predict the relative solubilities of salts from their Ksp's if the salts produce different numbers of positive and negative ions when they dissolve in water." from: http://chemed.chem.purdue.edu/genchem/topicreview/bp/ch18/ksp.php#use

§ SWMMWW and SWDM definition of basic; i.e., 80% TSS removal for design flow.

Water column (pond) pollutant removal for surface water discharge depends on aqueous chemistry, sorption to solids, and particle settling rates. In the case of dry ponds and shallow or undersized wet ponds, re-suspension may impair net pollutant removal, although re-suspension is not factored into the model. Pollutant removal via infiltration involves some of these same processes, but includes others as well, and the reaction environment is different. Anionic pollutants – chloride is a classic example – are highly mobile in soils, which tend to be dominated by negatively charged surfaces themselves. Nitrite (NO_2^-) and nitrate (NO_3^-) are also highly mobile; NO_2^- is usually converted rapidly to NO_3^-, which is itself subject to additional speciation as previously noted. Phosphate (HPO_4^{-2}) under weakly acidic to alkaline conditions readily complexes, e.g. with calcium and hydroxide to form hydroxyapatite, which is highly insoluble; or e.g. under acidic conditions may complex with aluminum and/or iron (Minton 2011). In the latter case sorption and release of soluble phosphorus are affected by changing redox conditions. And although attenuation is expected both by physical filtration and die-off, even fecal bacteria may travel through macropores in loam over silt loam or sandy loam, into and through the vadose zone, capable of causing groundwater contamination (Unc and Goss 2003). According to Keswick and Gerba (1980), pathogenic viruses and bacteria can both penetrate soils to groundwater at depths greater than those presumed by stormwater manuals to be protective. Balousek (2002), notes that "viruses at very low concentrations pose a high risk of contamination".

Assuming relatively sparse rather than dense vegetation[*], these facilities should not be modeled as vegetated swales or filter strips. Assuming infiltration as modeled, these facilities should not be viewed as treatment wetlands as the both the flood and saturated ground conditions will be ephemeral, although at different time scales. If this latter assumption is not true, and a rain garden was to hold water constantly, then modeling as a treatment wetland might be more appropriate. Hydraulically these are being modeled as shallow wet ponds with potential infiltration rates limited by underlying soils. For water quality they are viewed as dry ponds with dynamic particulate settling prior to any surface discharge, and otherwise as bioinfiltration facilities.

[*] Dense and sparse are relativistic terms. Without attempting a quantitative delineation; intent here is to think of dense as a 'lawn' of grass, as in a grass filter strip or regular bioswale (as opposed to a wet(land) bioswale), and sparse as being more likely a somewhat diverse collection of more widely spaced plants that are in this case both drought and water-inundation tolerant.

4.0 Sources of Uncertainty

4.1 Literature review

There is certain to be some to considerable overlap and/or repetitiveness in underlying data behind summary statistics in synthesis papers. For one thing, some recent synthesis papers use data from the International Stormwater Database (ISWBMPDB 2011a). For another, in this author's experience, stormwater data in general are widely copied from one report to another. Some are summaries of summaries, e.g. Table 7-2 in Heaney et al. (1999), and stormwater characteristics reported in Geosyntec Consultants and Wright Water Engineers (2011). The end result is that absent time for a comprehensive re-evaluation of all found reports to ferret out independently collected data, this current assessment is no exception, falls prey to the same weaknesses. It does not claim to represent the full state of values or variability in the underlying raw data or the real world, but is offered as a current review relevant to the question of pollutant removal efficiencies for the modeled and similar facilities.

4.2 Data quality and representativeness

In this author's experience, in the world of stormwater monitoring and reporting, data are often not supported by Sampling and Analysis Plans or Quality Assurance Project Plans, or Standard Operating Procedures. These may be absent, or if they exist, are often difficult to obtain, or if obtainable are often inadequate. Reports frequently do not indicate degree of adherence to sampling and analysis plans or method and data quality objectives. This isn't to say unequivocally that all the data are no good – it's to say for the most part, the representativeness and quality of the data cannot be known. Reports that do contain their own caveats – as opposed to reports containing none – may highlight one or more reasons to be skeptical about the results. e.g. Kantrowitz and Woodham (1995), note that their results are not entirely empirical:

> "Because all the stormwater entering the detention pond was not measured at the inflow site, computed stormwater inflow loads were adjusted to account for loads from the unmonitored areas. The ratio of stormwater volume measured at the outflow site to stormwater volume measured at the inflow site was used to adjust inflow loads for individual storms. Pond efficiencies for selected water-quality constituents for each of the storms were estimated by dividing the difference in outflow and adjusted inflow loads by the adjusted inflow load".

In this example, loads from unmonitored areas cannot be known and cannot be assumed to be the same as loads from monitored areas; so this is an area of potential error of unknown magnitude, casting doubt on the veracity final reported calculated values.

This is a case in point with respect to sample representativeness, but at least the authors are forthcoming and the reader is advised regarding reliability of the data. This author has observed similar examples where wet ponds and vaults had more than one inlet, but only one was being sampled, with the presumption that this was representative of the other inlet(s), but without even as much as pilot paired-sampling to assess whether that was a reasonable assumption. In one case the inlets fed from different sides of a divided highway, where there might well be different traffic loads during each runoff event, and therefore a reasonable expectation that pollutant concentrations might differ between the two inlets. Another case had to do with highway runoff

having been treated by a vegetated filter strip mixing with and therefore diluted by runoff from an adjacent vegetated embankment before being collected as representing 'treated' runoff.

The most current International Stormwater BMP Database (ISWBMPDB) composite BMP facility table, as of August, 2012 is dated November, 2011 (ISWBMPDB 2011b). Only three retention ponds, two detention basins, and one bioretention facility are represented from the state of Washington. Whether those facilities in other states use the same design criteria as are applied in Western Washington or King Couny is an open question with inadequate resources to answer here.

With regard to representativeness, consistency is lacking in nomenclature among reports. What is called a wet pond in one report may be called a retention pond in another. While most would consider these terms synonymous, in the absence of comparable detailed information about all facilities of one kind or another (e.g. wet ponds), we cannot assume they are all the same; therefore, a portion of variability in pollutant removal efficiency is likely to be a consequence of differences in design. More to the point, in the absence of thorough vetting, we cannot assume that pollutant-removal rates from facilities in other states represent the same efficiencies we might expect from facilities built according to state of Washington design standards.

By the same token, some of the reported values are themselves based on assumptions, e.g. Claytor and Schuler (1996) show no data for bioretention, but say, "Presumed to be comparable to Dry Swale". While the presumption may have some merit, it does not yield empirical data for bioretention. This same citation is also an example of a common ambiguity. Reports often do not state, e.g. whether pollutant percent removal rates for bioretention is with respect to surface flow or filtered under-drainage leaving the facility. In this cited case (*ibid*), bioretention performance is said to be presumed to be comparable to a dry swale. Since dry swale performance is assessed by surface flow pollutant concentration change, in this case that should also apply to bioretention, and underdrain filtrate would not be part of the equation. Where there is un-resolvable ambiguity as in this case, reported or alluded-to values are not used in our assessment.

In some cases, where reported, sample sizes are simply too small to be considered by those authors to be representative; e.g. in Winer (2000), median pollutant removal rates are flagged when based on < 5 data points. Historically little to no effort has been made to determine sample size required for statistical significance. One known current protocol is WA Ecology's TAPE guidance (Ecology 2011; Hoppin 2008) which is required for that agency's approval of new 'emerging' technologies, and which was applied to required monitoring of water quality treatment BMPs under the 2007-2012 NPDES Phase 1 Stormwater Permit. TAPE contains sample size criteria for stated statistical goals. However, the vast majority of historic stormwater monitoring has not met this level of rigor; and in this author's opinion, the number of known ponds tested using TAPE protocols is too small and of too limited geographic range to be considered representative of anything more than the locations where tested; the results are constrained by low TSS influent values and particle size distributions skewed toward very fine particles.

Another area of concern is infrequent reporting on how non-detects ('equal to or less than' the lower reporting limit) and 'equal to or greater than' data are dealt with; and that when reported, rarely if ever is appropriate methodology used. Non-detects are also referred to as left-censored data. At the other end, right censored data may occur e.g. when fecal coliform (FC) are enumerated by 'most probable number (MPN), and there is a method upper limit of e.g. 1600 or 2000 MPN (or a multiple if pre-dilution is applied). When membrane filtration (MF) is used, upper limits are imposed by plates designated 'too numerous to count' (TNC), generally where

colonies reach 150 – 200 per plate, multiplied by dilution factor. Right censored data may also occur when values reach the upper limits of instrumentation readouts or calibration ranges. Helsel (2005) has documented that deletion and substitution methods[*] for left censored data can cause serious errors in both summary statistics and statistical tests. There are appropriate statistical approaches to deal with these situations to yield good approximations of data distributions, but whether or not these methods are applied is rarely if ever reported. At least some of Helsel's methods for dealing with non-detects are derived from earlier established statistical methods for dealing with right censored data. In this author's reading of stormwater literature to date, any reported handling of censored data has involved substitution for non-detects and use of the upper limit values at the high end. This author has never seen correction for right-censored data applied, even though at least one of these methods pre-dates and is the basis for one of the non-detect methods.

Another problem with summary reports is that they are frequently summaries of summary data. Yet whether the source data are weighted or not in the compiled summaries is rarely if ever reported. e.g., the median of medians from three separate studies might be reported, with original sample sizes of n = 5, 12, and 20 runoff events, yet the medians may or may not all be treated equally; there is no way of knowing. Some reports give "average" values, some specify arithmetic mean, and some specify median, and some use geometric mean, which is generally close to median.

Clary et al. (2010) are careful to point out many sampling limitations including some of the issues noted above; and in addition, grab sampling limitation on representativeness, holding time and sample splits as sources of error, and uniquely for bacteria, issues raised by culture, dilution, and count methods.

Last, as a case in point example, Heaney et al. (1999) provide this caution in their report:

> "Note: The above-reported removal rates represent a variety of site conditions and influent-effluent concentration ranges. Use of the averages of these rates for any of the reported constituents as design objectives for expected BMP performance or for its permit effluent conditions is not appropriate. Influent concentrations, local climate, geology, meteorology and site-specific design details and storm event specific runoff conditions affect the performance of all BMPs."

4.3 Modeling assumption: Re-suspension of solids

For the rain garden pools dynamic settling is assumed, not quiescent, because given the shallow depth of the ponds and the assumed infiltration rates, they are not expected to hold a 'permanent' wet pool, and are expected on average to drain completely between runoff events. While this may be some debate over whether this is overly conservative or reasonable at low infiltration, we are also assuming no re-suspension from wind, which is arguably a non-conservative assumption, since shallow ponds are more susceptible to wind re-suspension than are deep ponds. Bentzen et al. (2009) report that with a pond with an average and maximum depths of 1.44 to 2.1 feet respectively, "mean outlet concentration of suspended solids is well correlated with wind speed."

[*] e.g. reporting limit (RL), ½ RL, 1/SQRT(2RL), etc.

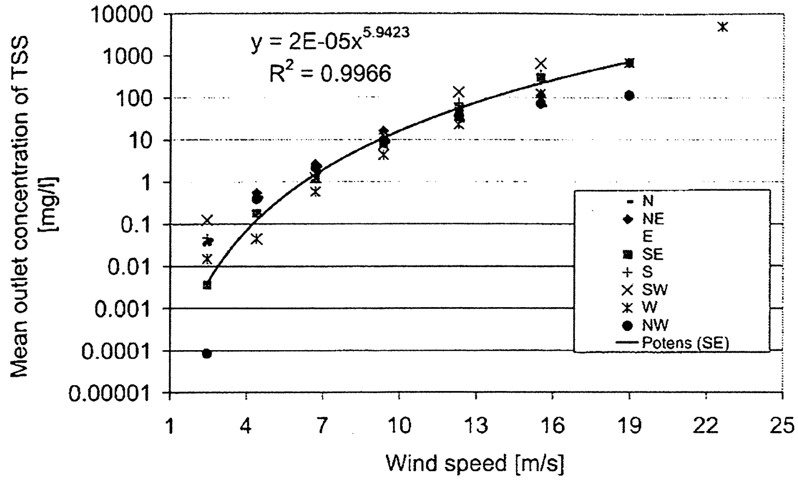

Figure 2. Mean effluent concentration as a function of the wind force from eight directions (Bentzen et al. 2009)

On the other hand, that pond had a clear surface, but plants in the rain garden ponds may dampen wind effects to some extent. Filled rain gardens in low permeability soils (0.15 in/hr infiltration) will hold water for 79.5 hours (3 1/3 days) between rain events, those modeled in high permeability soils (3.0 in/hr) will drain completely during a 4-hour lull in precipitation.

The rain gardens in pond mode are vegetated but do not hold a 'permanent' wet pool between storms. TSS pollutant removal is estimated here as if these were dry ponds subject only to dynamic settling. The vegetation is not dense (i.e., not a grass mat; more along the lines of reeds, sedges, and/or bushes, perhaps with some low density grass); while this will not present much if any enhanced filtration, it is likely to interfere with laminar plug flow, on which Stokes' calculations are based. On the other hand, the presence of vegetation should largely mitigate potential wind-driven sediment re-suspension, so we can avoid factoring that in for the time being). The 7 hour travel time is based on purely hydraulics calculations, not considering the plants.

4.4 Reported pollutant removal rates

As noted previously, there is considerable uncertainty of pollutant removal efficiency. As stated in the Summary:

> *All removal efficiencies should be viewed with these overriding caveats: The pollutant removal values used in this modeling exercise are applicable solely to this exercise, and should not be considered applicable in any other context. They are based on a necessarily time-limited literature search and survey, and cannot be claimed to represent a scientific meta-analysis. A major data source is the International Stormwater BMP Database (ISWBMPDB 2011a), which is a compendium of stormwater data, at least some if not much of which has been reported elsewhere.*
>
> *As put by Gossett et al. (2004), "various monitoring programs have differing project goals and objectives, differing mandates from regulatory agencies, differing sampling designs, and differing laboratory analytical methods", all of which hinders data comparability. The same can be said for independent research monitoring efforts, whether compiled in the database or found separately. To the best of our knowledge these data have not undergone independent third-party assessment of facility monitoring settings, protocols, and setups, data quality, and verification of as-built design*

and maintenance operations[*] as would be required for scientific meta-analysis; or for evaluation as to comparability to King County or WA Ecology design and maintenance standards. Moreover, there is very little data in the database from western Washington.

The name "International" notwithstanding, the ISWBMPDB contains only one study from outside the US – a study from Sweden. Coincidentally, an analysis of stormwater treatment facilities in Sweden (Persson and Pettersson 2009) reported that, " . . . all data on ponds in Sweden that have been monitored were collected and evaluated. The results show that of 27 measured ponds only nine had monitoring programs that were correctly designed to reveal anything about pollutant removal . . .". Also, as noted previously, an agency (WSDOT 2009) voluntarily retracted some data for known problems in some cases, and lack of supporting evidence regarding data representativeness or credibility in others. The one Swedish study reported in the ISWBMPDB does not appear in the Persson and Petterson report; but that report and the WSDOT retraction beg the question, if all the programs in the ISWBMPDB were evaluated as were the Swedish and WSDOT programs were, how many would pass muster and be retained in the database, and how would that affect the summary and more detailed data assessments and conclusions?

With regard to data comparability in general (not specific to but applicable to the database), Siu et al. (2008) note, "Historical and present day solids' concentrations data for stormwater often do not contain detailed information on the methodology used during analysis (e.g., filter paper pore size, which methodology by organization used, aliquot size used, particle size distribution)". An additional serious data comparability concern is Vb/Vr. According to Minton (2009), "Depending on the state BMP manual, the design Vb/Vr ratio ranges from about 1.5 to 6 by happenstance, with about 1.5 to about 2.5 being the most common". To evaluate this statement fully would require reviewing each manual for definitions of Vb[†] and Vr to see if they are comparable between manuals, and to consider that different climatic regions might require different Vb/Vr ratios to achieve the same pollutant removal. Regardless, these variables create considerable uncertainty in wet pond pollutant removal rates. Even given identical Vb, ponds may behave very differently depending on number of cells, volume ratios of multiple cells, overall geometry, and vegetation.

As noted previously, for this model TSS and TP removal from wet ponds is based on regulatory design manual assumptions, and not on the current literature survey. As will be discussed further on, there is some cause to reassess the validity of those design assumptions.

For the compiled ISWBMPDB database, and indeed for any survey or data review, the summary statistics – e.g. pollutant median percent removal – will inherently change over time as more data are collected, as indicated by Table 1 and Table 2 following.

[*] In this author's experience and view from two agencies, it is not uncommon for facilities to be installed incorrectly, not according to design, materially defective, and/or not maintained. In addition, this author has seen a number of monitoring setups that could not be justified as capable of delivering representative samples or flow data.

[†] Vb may differ with regard to how side slopes are factored, whether divided into more than one cell – and if so whether the divider berm volume is subtracted.

	Bioretention		Wet Pond	Retention Pond
	2008	2011	2008	2011
Cu-diss	--	--	40%	33%
Cu-solid	--	--	<-25.6%>	< 60%>
Cu-total	--	48%	29%	40%
FC	--	--	--	93%
NH3/NH4+	--	--	--	--
NO3-	--	23%	36%	63%
OP	--	-14%	11%	64%
TKN	--	8%	13%	15%
Total N	--	21%	13%	27%
TP	--	7%	43%	59%
TSS	--	80%	61%	80%

Table 1. International Stormwater BMP Database pollutant removal rate summaries, 2008 and 2011.

Bracketed <> values are inferred from total Cu minus dissolved Cu. Percent removal values are calculated from median influent and effluent values in the database.

This is acknowledged by the ISWBMPDB authors:

> "The BMP Database data set is continually growing; therefore, the statistics reported in this table will change as the data set grows. The analysis data set for Table 1 is based on the August 2010 version of the BMP Database for all parameters except metals, which is based on the December 2010 version of the BMP Database." (ISWBMPDB 2011c)

	Retention / Wet Pond				Bioretention	
	EPA (Heaney et al)	CWP NPRPDB (v3)*	ISBMPDB		CWP NPRPDB (v3)*	ISBMPDB
	1999	2007	2008	2011	2007	2011
Cu-diss			40%	33%		
Cu-solid			<-25.6%>	< 60%>		
Cu-total		57%	29%	40%	81%	48%
FC		70%	--	93%		
NO3-		45%	36%	63%	43%	23%
OP			11%	64%		-14%
TKN			13%	15%		8%
Total N	0 to 80 %	31%	13%	27%	46%	21%
Total P	0 to 79%	12%	43%	59%	5%	7%
TSS	91%	80%	61%	80%	59%	80%

Table 2. Pollutant removal summaries including additional data sources.
Comparison to earlier data from EPA (Heaney et al. 1999) and the Center for Watershed Protection (2007) shows additional change in reported pollutant removal rates over time, for a more limited number of pollutants. Bracketed <> values are inferred from total Cu minus dissolved Cu.
* (v3) = Center for Watershed Protection, National Pollutant Removal Performance Database, v.3. ISBMPDB percent removal values are calculated from median influent and effluent values in the database.

Among these data, there were no bioretention data reported prior to 2007. The terms Retention Pond and Wet Pond are assumed to be synonymous, as are the terms TSS and suspended solids; although we cannot rule out there may be differences in meaning, which could affect results. USGS favors suspended sediment concentration (SSC), which is not analytically the same thing as TSS. The terms TSS and SSC are often used interchangeably, erroneously (Gray et al. 2000; Siu et al. 2008). James, (1999) and Roesner et al. (2007) note lack of agreement on the definition of stormwater TSS. Lack of historic standardization in TSS processing methodology (Bent et al. 2003) and variability in sample collection and lab processing (Roesner et al. 2007)

lead to questionable representativeness and data comparability. These and related sources of error are discussed further in Section 4.5, Experimental, sampling, and analytical uncertainty.

The first point is the differences in pollutant removal rates for the different reporting periods, and that these values have not and will not remain fixed. The second is that known problems with sampling and analysis methodologies render highly questionable, concentration values for TSS and both solid and soluble fractions of phosphorus and metals. The third point is the paucity of data relevant to this current investigation from local regional facilities. The most current compilation to date (ISWBMPDB 2011b) contains local results from only one bioretention facility, two detention basins, and three retention ponds. Design differences and regional climatic differences make applicability of much of the national data questionable. Mobilization of different pollutants and BMP performance are both affected by e.g. storm intensity and duration, which vary regionally. Further, some facilities evolve over time, which can affect pollutant removal rates. For example, Lavielle (2005) and Pettersson et al. (2007) found that changes in stormwater pond morphology over time (about seven years in their studies) "affected nitrogen compounds, Cu and Zn removal efficiency negatively"[*]; and they attribute that to vegetation growth and sediment build-up.

4.5 Experimental, sampling, and analytical uncertainty

4.5.1 Overview

Intrinsic uncertainty is a consequence of highly variable mixtures of highly variable concentrations of pollutants, some in varying speciation forms; e.g. dissolved/solid, and some are more complex in other regards chemically, e.g. speciation of nitrogenous pollutants nitrite, nitrate, ammonia/ammonium, and total nitrogen. Differences in both regional and per-storm intensities and duration affect mobilization and runoff profiles for different pollutants. Additional uncertainty from induced error may result from choice of sampling locations and sampling and analysis methods.

That some pollutants tend to sorb to and/or constitute smaller particles in within a TSS particle size distribution (PSD) range means we cannot assume a proportional decrease in these pollutants commensurate with TSS percent removal, as it is skewed toward higher removal efficiency of larger and more dense particles. Some forms of organic content, e.g. compost, peat, or wood fiber in rain garden mix may aid in removal of some pollutants by sorption, ion exchange, filtration, and/or providing an environment supportive of microorganisms that may break down or sequester some harmful pollutants, yet these media may also be sources of dissolved organic carbon which can facilitate some pollutant mobility (leaching) by formation of colloids (Béchet et al. 2006), (Badin et al. 2008), (Hathhorn et al. 1995), or may while removing some pollutants, release some as well; e.g. recent work at Washington State University[†] indicates that some bioretention mix be a net source of some nutrients in leachate, and may while trapping some copper still release dissolved copper at levels of concern for salmonids.

Vaze and Chiew (2004) say that "Practically all the particulate TP and TN in stormwater samples are attached to the sediments between 11 and 150 *u*m. This suggests that to effectively remove

[*] Lavielle
[†] Puyallup, WA campus, Curtis Hinman principal investigator. Information from a research annual review meeting, but no published proceedings yet.

particulate TP and TN, pollutant treatment facilities must be able to remove pollutants down to 11 *u*m". By modeling, Fletcher et al. (2004) find about 55% of the TP PSD to be 10 *u*m and smaller. The discrepancy likely reflects limitations of small sample sizes and limited geographical representation for empirical data in one case, and presumably calibration data in the other. They also are both subject to some skepticism, since TP is the sum of solid and dissolved P, yet in both cases TP is given PSD, which is not possible for the dissolved fraction. At least this discrepancy gives potential cause for some of the high variability, making general generalized single-value inferences a risky business. Likewise, different studies with different PSD midpoints and extremes point are likely indicative of both environmental variability and differences in sampling and analysis methodologies.

4.5.2 TSS (total suspended solids)

"The performance of stormwater best management practices (BMPs) that rely on sedimentation to remove solids from runoff is heavily dependent on settling velocity and ultimately particle size distribution (PSD) of the solids." (Hettler et al. 2009). It should be added that settling velocity is also dependent on specific gravity (SG), i.e. the particle SG or density distribution, particle shape and porosity, water viscosity as a function of temperature, and to some degree matrix effects[*] may play a role.

4.5.2.1 Historic overview

For purposes of BMP performance evaluation, WA Ecology used to define total suspended solids as "all particles smaller than 500 microns in diameter" (Hoppin 2008), but now defines it by specifying modified analytical methods[†] (Ecology 2011) that are – as modified – functionally the same as SSC methodology (Selbig 2012c). However, Ecology only recommends these modifications, it does not require them, so results will vary according to adherence to the recommendation or not.

A white paper by a stormwater treatment vendor (Rinker 2004) on *vehicular traffic* stormwater solids indicates, e.g. that the National Urban Runoff Program (NURP) found a particle size distribution with ~90% of solids below 100 *u*m and a minimum particle size of 1 *u*m. At the other extreme, a single-site (according to Rinker, 2004) study (Sansalone – see 1998 citation in Rinker, 2004) found a PSD range with a bottom of ~70 *u*m, about 50% below and 50% above a log scale x-axis sigmoid curve mid-inflection point of 500 *u*m, topping out at 10,000 *u*m. Another paper by Sansalone and Buchberger (1997) reports "Solids ranged from smaller than 1 *u*m to greater than 10,000 *u*m" from highway runoff, although the PSD ranges they report in tables and graphs are based on the portion of solids larger than 25 *u*m, and they do not provide the proportion of solids above:below that value.

Rinker (2004) points out that the NURP data as well as Rinker's own monitoring represent multiple sites, whereas "the Sansalone" (1998) "study only considers one site". The Rinker author(s) cite several other papers not graphed, indicating coarser PSD ranges. That the cited studies are all from transportation does not necessarily rule out consideration of use of their data. Transportation is a non-trivial portion of TSS generation in the built-up urban environment. Still, one might expect different TSS profiles between urban and rural roads, and between roads in general and other land uses – not only with regard to concentration, but also with regard to particle density and size distributions.

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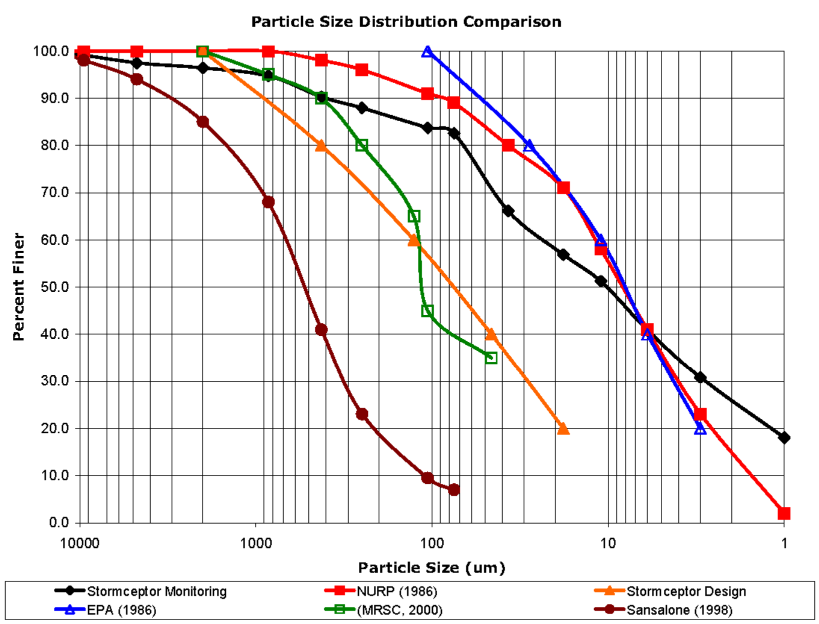
[*] presence of other constituents and concentrations of all constituents

[†] SM 2540B or SM 2540D, modified per TAPE (Ecology 2011) pages 28-29

To assess pollutant removal, we need a PSD curve containing a full range of particle sizes expected to be representative of multiple land uses, down to the smallest particles expected to be a functionally significant portion of the PSD; although we need to reconsider the upper particle size limit if we state at the onset that we are only going to be concerned with removal efficiency for particles < 500 microns. The Udden-Wentworth silt scale has a range of 3.9 to 62.5 *u*m, and the ISO 14688 silt gradation is a close match between 2 *u*m and 63 *u*m. Udden-Wentworth also specifies clay between 3.9 and 1 *u*m, and colloids < 1 *u*m. Unpublished data likely from a single site (Stormceptor monitoring, presented graphically in Figure 3 (Rinker 2004)), indicates close to 20% of the TSS is less than 1 *u*m, the NURP curve bottoms out at ~ 1 *u*m.

Examining NURP PSD data (Rinker (2004; Figure 3), the curves are all sigmoid with a log scale x-axis, as are the curves (more or less) indicated in most other studies. Rinker (2004) does not convey whether the PSDs are based on counts, volume, or mass. DeGroot (2005) uses the same PSD figure – without % units, but also presents many other PSD graphs; of which some are likewise absent % units; but of those with units, all are indicated as mass percent. DeGroot's co-author confirmed units for all the graphs in their report as mass % (Weiss 2012).

Of the curves in Figure 3, the closest match to the silt scales is the EPA curve (3 to ~100 *u*m). The NURP curve coincides almost exactly from 20 *u*m and smaller, but extends all the way down to 1 *u*m; it also extends up to 10,000 um, but is topped out at >99% smaller than 800 *u*m and 98% smaller than 500 *u*m. This suggests that of the available curves, the NURP curve may provide information which is thought to be representative overall and contains empirical data at the low end of the PSD range.



NURP - National Urban Runoff Program (EPA, 1983)
EPA - Detention Basin Analysis (EPA, 1986)
MRSC - Municipal Research & Services Center (of Washington)

Figure 3. Particle size distribution comparison from Rinker (2004)

Still, the ~1 ½ orders of magnitude difference between *50 percent finer* on the NURP and EPA curves vs. the Sansalone curve gives one pause to consider range of possible PSDs. Here it appears that Storceptor was tested using a PSD representing the midpoint between these curves.

If target % removal was achieved but no more with that PSD, lesser performance would be expected for PSDs more closely resembling the NURP and EPA curves.

Note that the NURP curve is dated at 1986 in the graph and 1983 in the key. Other reports generally cite "NURP (1986)" or NURP (Driscoll 1986), or simply (Driscoll 1986) for what appears to be the historical data used for the graph above. That cited document does not contain tabular data, and the log scale graphs (Figures 2 and 3 in the Proceedings paper) are not readily translatable to the data in the graph in Figure 3. Those report dates notwithstanding, the data are from the Nationwide Urban Runoff Program final report (US EPA 1983), which places data collection back 30 years in the past, when we might expect methodologies were not as mature as they are today. More recent observations are more revealing of specific causes of variability, and indicate likelihood of sampling error in the historic and even recent data as a consequence of sample collection methodology, and from inconsistent laboratory methods with intrinsic sub-sampling variability.

4.5.2.2 Stormwater particles are not 'ideal'

Stokes' law is based on settling of a single smooth spherical particle. According to DeGroot and Weiss (2008), Bäckström (2002)[*] found that "Stokes' Law could be used to accurately estimate the settling velocity of particles larger than 20 microns in diameter. Smaller particles could not be modeled with Stokes' Law, however, and Bäckström (2002) hypothesized that the deviation from Stokes' Law at lower velocities could be attributed to lower densities, non-spherical shapes, and/or electrostatic forces" (DeGroot and Weiss 2008). This was reported as a laboratory column study; what is not evident (absent the source paper) is the nature of the particles; i.e. were they manufactured standards, e.g. Sil-Co-Sil, or actual stormwater solids, and/or were shape, smoothness, and porosity evaluated by scanning electron microscopy?

4.5.2.3 Historic derivation of 'typical particle size distribution

The NURP data are almost always presented as PSD, when in fact, the NURP protocol was a particle settling rate methodology developed by US EPA (Driscoll, 1986, cited by (Hettler et al. 2009); and noted by (Gulliver and DeGroot 2010)); i.e. settling rate was measured and converted to PSD using Stokes' law. That means there is some inherent error in the original PSD estimates. This is compounded when converting these PSD values back into settling rates for calculating BMP solid pollutant removal efficiencies, especially given that both particle size and density distributions vary from site to site and over time. Besides size and density, particle settling is affected by particle geometry (overall shape, roughness, and porosity) and the viscosity of water (as a function of temperature)[†]. Variability of all these aspects within and between storms, and among different locations means there are no 'true' values or ranges of values that will be predictive without a large margin of uncertainty.

4.5.2.4 Variability in particle characteristics other than size

4.5.2.4.1 Viscosity of water

Viscosity is inversely proportional to temperature. Stormwater particle settling velocities from EPA (1986) and MOEE (1994) are widely disseminated and used in modeling. Some other

[*] Unable to obtain paper in time to evaluate directly

[†] Viscosity does not itself appear in equations for particulate removal efficiency; settling rate *Vs* is factored in must be affected by the viscosity of water, but temperature is not given in association with standard settling tables, e.g. in US EPA (1986) and MOEE (1994).

authors may derive settling rates independently, e.g. Li et al. (Li et al. 2008). What is lacking in these examples is the temperature at which the settling experiments were done. If settling velocity *Vs* was determined, e.g. at 4 deg. C or standard IUPAC 0 deg. C, settling ponds operating at higher temperatures most of the time may be oversized; whereas if *Vs* was determined at e.g. standard ambient temperature of 25 deg C, settling ponds may underperform during cool to cold periods. Hopefully each test method's temperature exists somewhere, but it looks like some information mining will be required.

Gulliver et al. (2010) find, "From 0 °C to 30 °C (32 °F to 86 °F), the settling velocity of fine silt (0.02 mm diameter, 7.87×10^{-4} inches) approximately doubles". The effect is smaller for smaller particles which have lower settling rates to start with, and greater with larger particles which have faster settling rates to start with. Putting aside the question of whether NURP, MOEE, or any other PSD is adequately representative of local conditions, it is crucial for modeling and facility design to know the temperature at which any settling velocity determinations were made, and to adjust accordingly if necessary.

4.5.2.4.2 Density / Specific Gravity

Settling prediction based on Stokes' law typically assumes a particle density of 2.65/cm^3 (or specific gravity of 2.65 (unitless), for silica sand), whereas density of particles will be lower if organic material, and higher if metallic. Further, Stokes' law presumes spherical particles, which would be expected to be rare if ever found in stormwater particulates. Minton (2011) notes "particles in stormwater are highly pitted and porous", although he also notes the roughness and porosity (as expressed by surface area) may be affected by coating by petroleum organics.

According to Weiss et al.(2010b), settling predictions based on Stokes' Law may have a settling rate error up to 25% when applied to clay, silt, and up to fine sand. Using Newton's law, Li et al. (2008) predicted settling velocities for particles in a range of 2 to 400 microns, and assuming spheres with specific gravity (SG) = 2.6 and 1.35, and cylinders with SG = 1.35. They also ran sedimentation experiments with stored highway runoff samples with stable particle size distributions. They found the actual settling rates to be much lower than predicted. According to the Ontario Ministry of Environment and Energy (MOEE 1994), "Monitoring that was done as part of the National Urban Runoff Study in the U.S. (EPA, 1986), however, suggests that the settling velocities for particles in stormwater are much less than that given by Stokes' Law or Newton's Law. The settling velocities given by the NURP study are 1/100 of that given by Stokes' Law"[*]. While these departures from ideal settling rates may be understood to result from non-ideal particles (not spherical, not smooth, variable porosity, and variable density) and non-ideal settling environments, it means we cannot rely on these classic equations for accurate pollutant-removal assessment.

Modeling pollutant removal by settling is suspect if particle density distribution is not factored in along with particle size distribution, or if settling velocity is given but the temperature at which that was determined is not reported. Extrapolating numbers from a frequency histogram of wet particle specific gravity (SG) from 180 grab samples from 16 runoff events in West Los Angeles, Li et al., (2008) found about 57% of the particles with SG between 1.2 to 1.4, ~ 79% between 1.4 to 1.6, ~ 87% between 1.6 and 1.8 and ~ 99% were less than 2.6 (Figure 4), compared to silica sand's SG of 2.65.

[*] NURP = Nationwide Urban Runoff Program

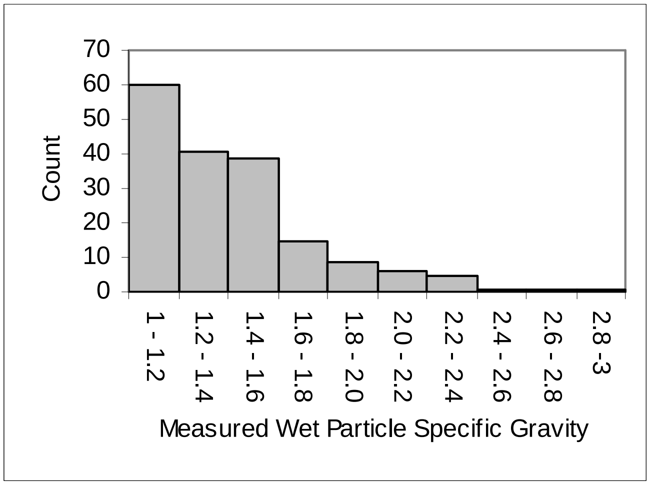


Figure 4. Particle specific gravity distribution
Adapted from Li et al., 2008.

Another example can be found in Brodie and Dunn (2009), where by again extrapolating from a graph, median percent inorganic content of particles < 500 microns is ~ 78%, ~ 64%, and ~ 58% for 'Road', 'Roof', and 'Carpark' (*sic*) land uses respectively. Also indicated in a table are mean percent inorganic differences between particle size classes (medium, fine, and very fine) within each land use type. There is no indication of densities of the inorganic or organic fractions, but this still serves as more evidence of variability in particle density.

Looking at sediment from a roadside gutter, Zanders (2005) found particle density to be < 2.2 mg/g, compared to > 2.6 mg/g "normally modeled for sediment". He notes that the observed lower particle densities could result from sorbed oil and grease ("coatings"), and the presence of tire wear debris. He does not mention – but these could also be factors – pulverized plant material and plastics in the sediment.

A compilation can be found in Degroot and Weiss (2008); presenting data from Li et al. (2006)[*] (Table 3). While some variability is evident, these values are more closely aligned with the silica mineral value of 2.65. However, the particulate sources are sediment and street sweeping, so it is not surprising that the lower specific gravities are not indicated, as those particulates are likely to have remained in the water column and washed out of gutters and catch basins more readily than denser particles.

These are but a few examples. The point is that particle density or specific gravity cannot be assumed or expected to be the same as silica sand, so solids removal by settling should not be modeled based on that assumption. It is equally likely that densities of different particle size ranges could differ, and that may need to be factored when making solids removal efficiency predictions.

[*] Presumed citation; not included in DeGroot and Weiss's References section, but cited as Li et al., 2006. The paper cited here is a highly likely candidate; unable to obtain in time for this paper, but appears to be the only stormwater PSD paper published by Li et al. in 2006.

Size ranges (μm)	Specific gravity	Sampling and experimental methods	References
Stormwater sediments			
<50	2.38-2.65	Manually collected from channel	Andral et al., 1999, Kérault Region, France
50-100	2.53-2.86		
100-500	2.5-2.82	Wet filtration-oven drying at 105° C	
500-1,000	2.51-2.7		
All sizes	2.20-2.27	Manually collected from traps installed on the bottom of detention basin	Jacopin et al 1999. Bordeaux, France
Street sweeping			
<63	2.19-2.56	Manual brushing rind vacuuming:	Paler et al. NV. London
63-150	2.13-2.51		
150-300	2.26-2.83		
300-600	2.02-2.41	Ono drying at 105° C-sieving	
600-1,000	1.99-2.59		
>1000	1.89-2.53		
All sixes	2.70-3.01	Vacuuming; air drying-sieving	Sansalone and Tribouillard 1999. Cincinnati
<75	2.61	Sweeping	Bäckström 2002, Luled. Sweden
75-L25	2.58		

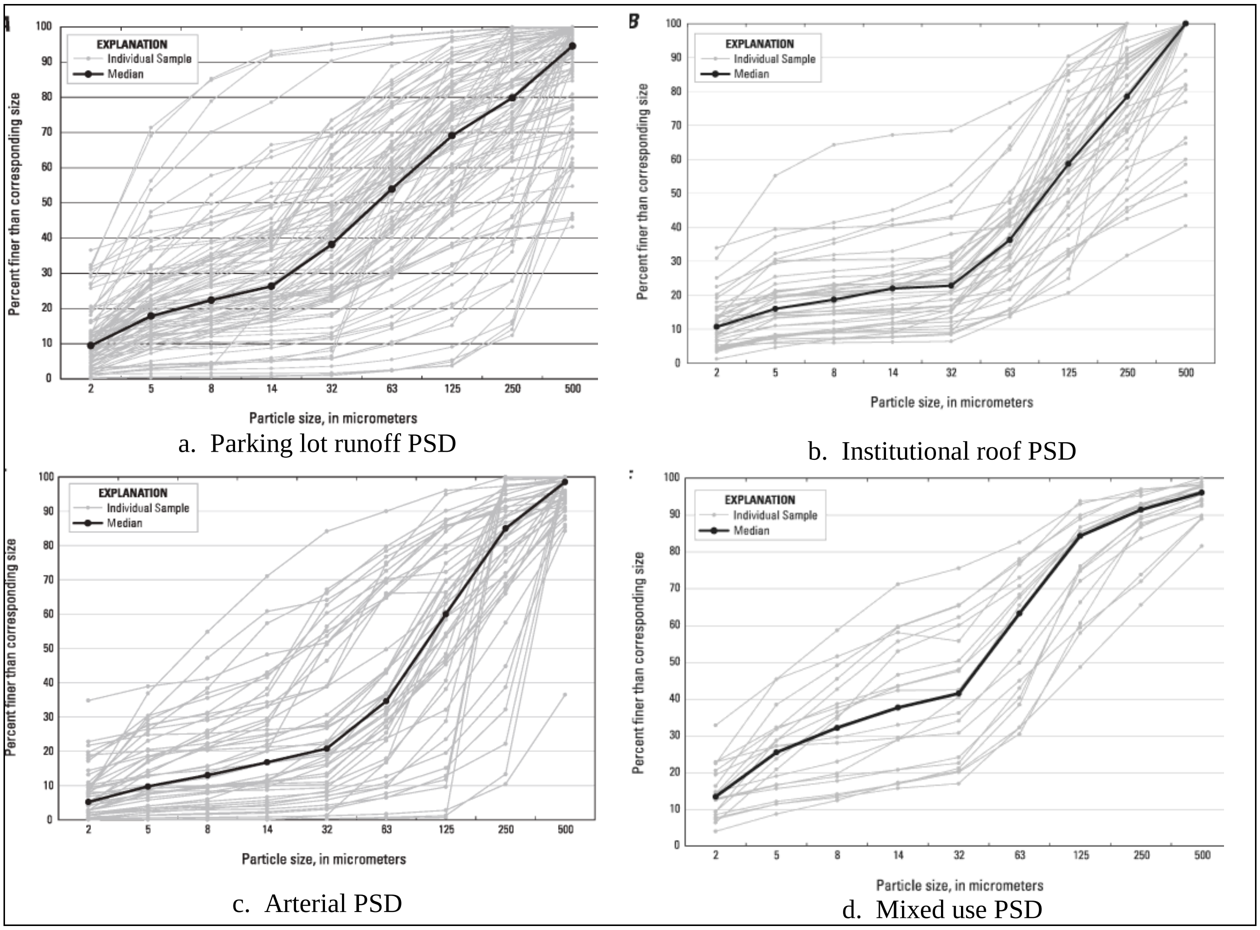
Table 3. Particle size and corresponding specific gravity.
Reproduced from Li et al.2006 as reported in DeGroot and Weiss (2008) as Table 9

4.5.2.5 Variability in particle size distributions

For calculation of particle settling, we are assuming no flocculation. Flocculation of micro-fine particles denser than water into lager aggregates would increase their settling rate. Rinker (2004) says, "flocculation is assumed for particles = (*sic*)[*] 20 μm. The use of the flocculated settling velocity equation provides a consistent settling velocity for particles < 50 μm that is equal to a 20 μm particle with a settling velocity based on Stokes' law with a specific gravity of 2.65"; but they provide no evidence or citation to support this. That about 70% of the NURP and EPA percent fines cited by Rinker are ≤ 20 *u*m seems to work against their own assumption. Minton (2005) also makes what appears to be contradictory statements in saying, "Given that a significant percentage of settleable solids in stormwater are small and have low settling velocities, as well as that most stormwater suspensions appear to be flocculent ....". Li et al. (2005) report, "Particles showed a natural aggregation, which required analysis as soon as posible but within 6 h of sample collection". Again, we refer to the NURP and EPA data (Rinker 2004), which indicate a spread of fines ranging down to 1 *u*m. While we are not ruling out the possibility of flocculation, we are basing solids settling analysis on the widely used NURP data as augmented and presented by MOEE (1994) in Papa et al. (1999), and more recently local regional provisional[†] data provided by Snohomish County (Herrmann 2012).

Aside from obvious differences between the fundamental curves in Figure 3, it is important to note that variability is lacking for all the data summarized in each of those curves. Among other pieces of missing information is the amount of uncertainty in each point along each curve; i.e. there are no error bars (e.g. confidence intervals, with sample sizes). By presenting all particle size distribution (PSD) data collected in one of their studies[‡], Selbig and Bannerman (2011a) show very clearly the amount of variability present (Figure 5).

[*] Should probably be ≤ 20 μm
[†] Not vetted completely yet, so subject to change.
[‡] Sampling locations in Madison, WI. Fixed point sampling methodology.



a. Parking lot runoff PSD

b. Institutional roof PSD

c. Arterial PSD

d. Mixed use PSD

Figure 5. PSD variability indicated by single-point sampling at four land uses at Madison, WI (Selbig and Bannerman 2011a)

Besides the amount of variability behind the median lines, note the discrepancy between the these curves' ~ 10 to 100 *u*m ranges, which are inflected downward, and the Driscoll (1986) data (EPA NURP in Figure 3), which are inflected upwards in the same PSD midrange; see Figure 6.

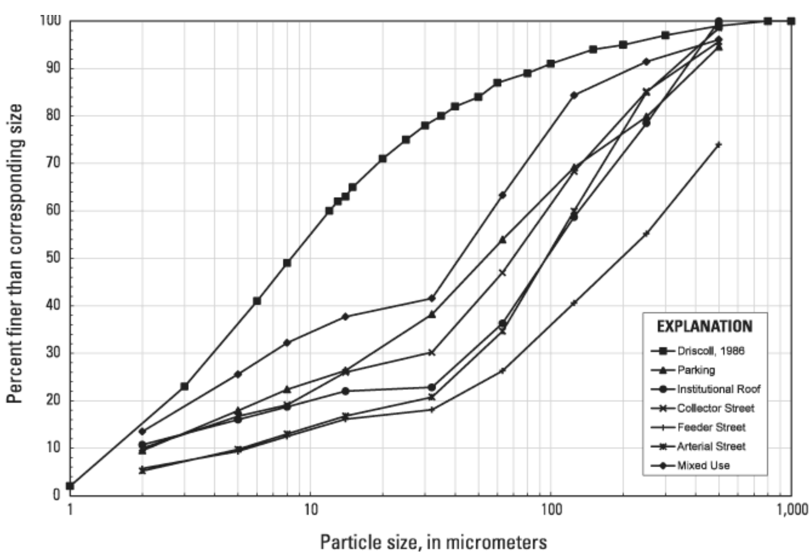


Figure 6. PSD data comparison; Selbig and Bannerman (2011a) Madison, WI, and Driscoll (1986)

Selbig and Bannerman (2011a) note in their Abstract:

> "Much of the variability can be attributed to use of different analytical techniques, sample-collection methods, and reporting between researchers. Results from this study further document the difficulty of deriving a single particle-size distribution that is representative of stormwater runoff generated from more than one source area."

And in their Summary and Conclusions:

> "Distributions of particles ranging from <2 to >500 μm were highly variable both within and between source areas. Results of this study suggest substantial variability in data can inhibit the development of a single particle-size distribution that is representative of stormwater runoff generated from a single source area or land use."

Similarly, we can compare regional PSD data* to the data from MOEE (1994) (Figure 7). The MOEE data is based on NURP settling velocity data (US EPA 1983, 1986), amended with Canadian research noted in MOEE (1994), resulting in insertion of an additional size fraction, improving resolution of particle size ranges at the low end.

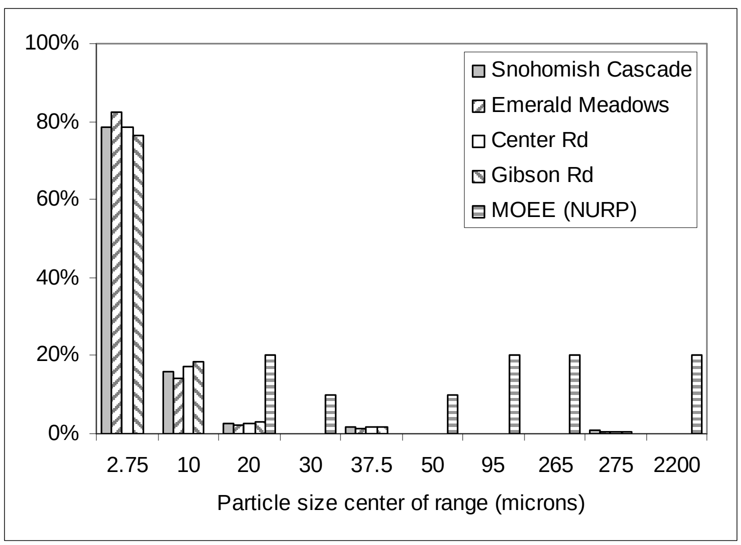


Figure 7. Particle size distributions† comparing four regional sites to the NURP data as given in MOEE (1984)

Another study found that of the non-coarse particles (defined as < 500 microns), for three impervious surfaces "(roof, road and carpark)", PSD "was dominated by particles less than 63 µm". (Brodie and Dunn 2009).

* Snohomish County provisional data (Herrmann 2012). These data are % volume

† The MOEE and NURP data are % mass. Snohomish County data are volumetric by laser diffraction (Herrmann 2012). This method should yield net particle volume within each given particle size range, excluding void space. Assuming equivalent mass densities among particle size ranges, expression as PSD fraction of total for each range should be equivalent to mass fraction as historic data are usually reported. The assumption of equivalent densities is untested and therefore questionable as discussed in the *Uncertainties* section, and itself subjects related analysis herein to some uncertainty.

It should be obvious that if measured without bias, the MOEE and local PSD data will result in very different TSS removal predictions. However, as will be discussed next, it is fair to be suspicious of some kind of bias in both cases; the amount and direction of bias are open questions.

4.5.2.6 Biased sampling and analysis

4.5.2.6.1 Sample analysis: TSS and associated pollutants

James, (1999) notes, "There is generally no agreed upon definition of TSS in regard to storm water runoff". Bent et al. (2003) provide a comprehensive overview of sample analysis methodology for TSS, finding, "In practice, TSS data are produced by a number of variations to the processing methods described in the American Public Health Association and others (1995)"; and following three examples, "The reduction in TSS data comparability by variations in protocols used is not limited to lack of consistency in processing and analytical methods. Gray et al. (2000) note that "TSS data are produced by several methods, most of which entail measuring the dry weight of sediment from a known volume of a subsample of the original". Glysson et al. (2000) put this as, "Subsampling in itself can introduce error into the analysis, and Frederick (Frederick 2006) as, "Heavier solids may not be picked up in the drawn aliquot for the TSS analysis". Law et al. (2008) state, "Specifically, research has shown that TSS measurement methods used for wastewater analysis applied to the analysis of stormwater can underestimate the amount of sediment in natural waters (e.g., Lenhart (2007), Gray et al. (2000))".

TSS, in addition to being particulate solids as a pollutant class of its own with respect to physical effects on receiving waters, is also a surrogate for any number of other pollutants. Brown et al. (2011) cite a number of studies confirming the conventional wisdom that various pollutants are associated with TSS, in some cases partitioned with particle size distribution. Some of this has to do with precipitation of dissolved materials to solids, some with sorption of bacteria and dissolved materials to particulates, and some is a consequence of the amount of TSS that is not simply silica mineral or relatively innocuous organic matter (e.g. mostly cellulosic plant material), but which itself constitutes pollutants in solid form, e.g. solid forms of phosphorus, nitrogen-containing compounds, metals, bacteria[*], and a myriad of other pollutants, most of which are not assessed independently in stormwater treatment monitoring. Sampling and subsampling error with regard to TSS will result in error in measure of any of these associated pollutants.

The same issues noted above with regard to subsampling in the laboratory are also of concern in the field, although to a greater degree because field sampling is less controlled than laboratory sampling.

4.5.2.6.2 Field sampling methodology

4.5.2.6.2.1 Bias introduced by flow splitters

We have to consider that flow splitters used for diversion and/or sample collection may result in biased TSS samples, unless designed specifically to avoid bias, and verified to achieve that. One claim is made to have succeeded (Brodie 2007), and Siu et al. (2008) cites several other others having reported relatively low coefficient of variation results from cone splitters they tested. It is clearly a concern, and absent testing and validation for any particular study, error from splitting has to be considered a potential source of uncertainty.

[*] free floating – in addition to those sorbed to other solid particulates

4.5.2.6.2.2 Grab samples and representativeness

Single or even a few grab samples are generally not considered representative of concentrations over the course of runoff events. Flow-weighted composites (event-mean concentrations, or flow-weighted EMCs) with minimum numbers of aliquots (e.g. 10) are generally considered to be more representative. Kantrowitz and Woodham (1995) calculated EMCs by collecting discrete samples 15 to 24 points along the storm hydrograph, and selecting 5 to 8 of those from specific regions of the hydrograph for chemical analysis and subsequent compositing computationally. While acknowledging that "During periods of stormwater runoff, both the quantity and chemical quality of the flow may change rapidly", they do not provide any evidence that their sampling methodology produces results comparable to EMC sampling with a larger number of aliquots. In comparison, Ma et al. (2009) found that "30 grab samples per storm event generally estimated the EMCs within 20% average error". Yet even EMC sampling with autosamplers can be problematic with autosamplers and solids particles.

4.5.2.6.2.3 Bias introduced by autosamplers

Li et al. (2005) found particle concentrations collected by autosamplers to be lower than flow-weighted averages of 4 to 15 grab samples[*] per runoff event. Bent et al. (2003) note that "samplers operating on older technologies and construction were not able to collect representative samples when the sampler elevation exceeded the sampler intake elevation by 12 ft or more"[†]. James (1999) reports, "There is further evidence that automatic samplers are not capable of collecting TSS larger than about 125 micron or fine sand". James also says, "Commonly used peristaltic automatic sampling equipment does not appear to be capable of collecting representative samples of storm water runoff", citing Field et al. (1997). Field does not actually use the term 'peristaltic', but does say, "Sampling devices must be able to capture the heavier SS or settleable solids" .... "and not manifest biased results due to stratification. For automatic sampling devices, the velocities must be greater than the main stream velocity, and the intake ports must be placed at multiple levels". Whether a standard autosampler peristaltic pump can achieve that or not is the question; but it should be asked in conjunction with standardization of the meaning of TSS and the largest particle size (if there is an upper bound) of interest, or choice to use SSC[‡] instead of TSS. Siu et al. (2008) found statistically significant PSD-dependent recoveries with autosamplers in their study. They report recovery from a PSD with a median particle size of 100 microns was twice that of recovery from a PSD with a median particle size of about 257 microns.

4.5.2.6.2.4 Single point pickup and stratification

Up to this point all the data under discussion area assumed or documented to have been collected using single fixed point sample pickup orifices; yet stratification of suspended solids and related sampling-induced variability in particle size distribution of solids have been recognized by some for over a decade as factors in sampling bias (Bent et al. 2003; DeGroot and Weiss 2008; DeGroot et al. 2009; Field et al. 1997; Gulliver and DeGroot 2010; Kayhanian et al. 2005; Selbig

[*] 1 sample per 15 minutes for the first hour, 1 sample per hour for each of up to 7 subsequent hours, and 2 additional samples for longer runoff events.

[†] A demarcation of pre- vs. post-1993 is noted, although this observation is limited to "several" autosamplers; and one must assume that in many cases samplers built 1993 or earlier would continue to be used for some years past that date, as replacement is expensive and not usually done until equipment breaks down or if and when a serious design flaw becomes known.

[‡] Suspended sediment concentration, as recommended by USGS

and Bannerman 2011a; Smith 2002). Roseen et al. (2011) cite some of these as well as some others regarding autosamplers misrepresenting TSS loads. However, Roseen et al.'s own research demonstrated ability to collect samples unbiased with regard to concentration, but they acknowledge some bias in particle size. They note that a number of potential places where error can be introduced, including the autosamplers (and presumably setup), laboratory sediment concentration methodology, and sample splitters. In contrast, DeGroot et al. (2009) assert that "Automatic sampling inaccuracy is primarily attributable to the distribution of particles in the flow column. As noted above, Bent et al. (2003) found older autosamplers could not get representative solids samples with an elevation of 12 feet or more from the sample pickup point.. To date, the literature appears to contain more weight of evidence indicating sampling bias than not. It would be useful to see if others can replicate Roseen et al.'s results elsewhere. Even if they can be replicated, their acknowledgement of PSD bias is still a concern.

Selbig et al. (2012) found in a laboratory setting that fixed-point sampling overestimated the actual concentration of suspended sediment concentration (SSC) by 96%. Reflecting on this, and thinking about solids remaining in treated stormwater being shifted toward a PSD range containing only very small to fine particles, there should be far less stratification in facility (BMP) effluent. With a single pont sampler placed low, his could lead to greater positive bias at the BMP inlet than at the outlet, and the consequence would be appearance of greater solids removal efficiency than had actually occurred. Selbig (2012b) concurred, but added that compared to SSC analysis, commonly used methodology for TSS biases against coarse particles, which would somewhat counter overestimation from fixed-point sampling. There would still be overestimation of influent TSS concentration (because of sampling bias) and misrepresented PSD (*ibid*). It is worth noting that WA Ecology's TSS analytical protocol given in the 2007-2012 NPDES Phase I Municipal Stormwater Permit and in TAPE (Ecology 2011) is functionally the same as SSC methodology (Selbig 2012c). Degree of systematic bias in both cases (sampling and analytical method) is variable and unpredictable, so there is no way to correct for either. Even if mass biases were to cancel out by chance, PSD will not be representative.

In a laboratory experiment with an autosampler with a fixed pickup near the bottom of a 46 cm (18 in) pipe, DeGroot et al. (2009) found that particles less than 44 microns appeared to be within 127 percent of the fed concentration (overestimated by no more than 27%), while "coarse silts sampled at 153 percent of fed concentration and some sands sampled as high as 6580 percent of the fed concentration".

Smith (2002) recognized the possibility of PSD sampling bias even in a pipe as small as 1 ft, and build a static mixer assembly in an attempt to eliminate this bias. He still found that while, "particles less than 0.062 mm[*] in diameter were evenly distributed throughout the water column", and noted two other USGS studies found less than 4-5% bias for sediment fractions less than 0.062 mm, "Concentrations of particles greater than 0.062 mm in diameter, however, tend to be higher near the bottom of the pipe despite the turbulence created by the static mixers". Smith indicates suspended solids concentration differences between elevated sampling points and a 'standard' sampling point location in the same pipe, monitoring highway runoff (Figure 8).

[*] 62 microns

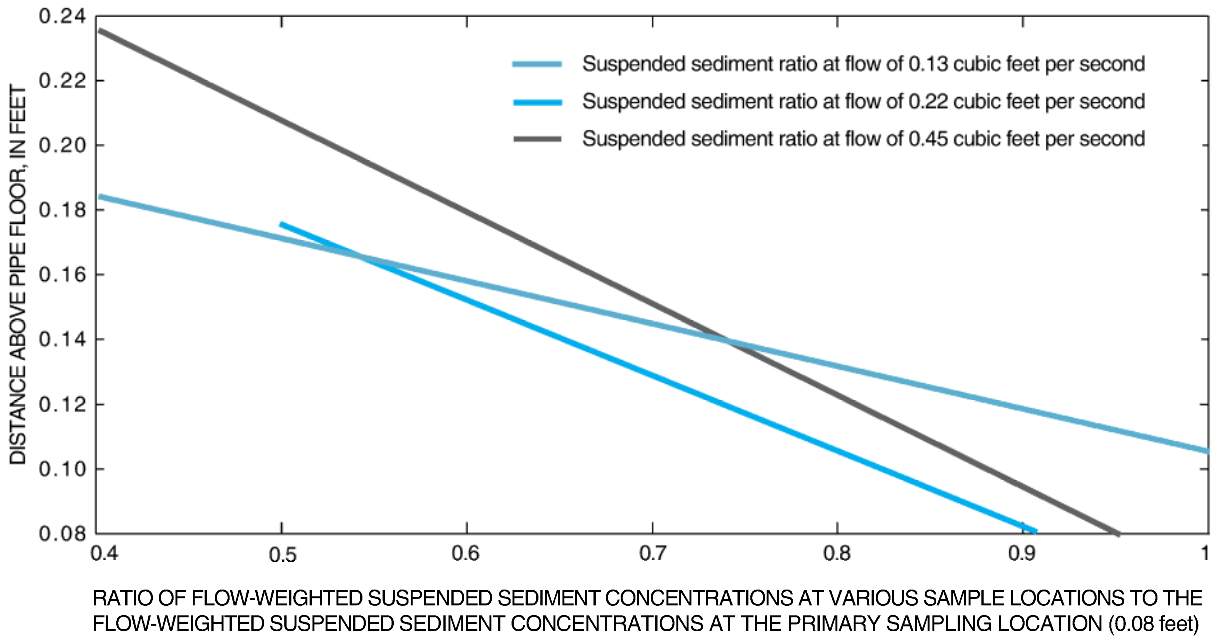


Figure 8. "General trend for flow-weighted suspended-sediment concentrations relative to the sample location in the water column of a 12-inch pipe at the inlet of the oil-grit separator at station 136, along the Southeast Expressway, Boston, Massachusetts"
(Smith 2002)

It is important to note that USGS favors suspended sediment analysis rather than total suspended solids. Gulliver and DeGroot (2010) note that most research regarding particulate sampling methodology has been done in the context of stream monitoring, and that optimizing representativeness in a pipe will differ from optimizing in a wide channel.

Overestimation bias effect is presented graphically below in Figure 9, with calculations for influent bias factor from 1 (unity, no bias) to 4 (300% overestimate) and effluent bias factor from 1 (unity, no bias) to 1.25 (25% overestimate).

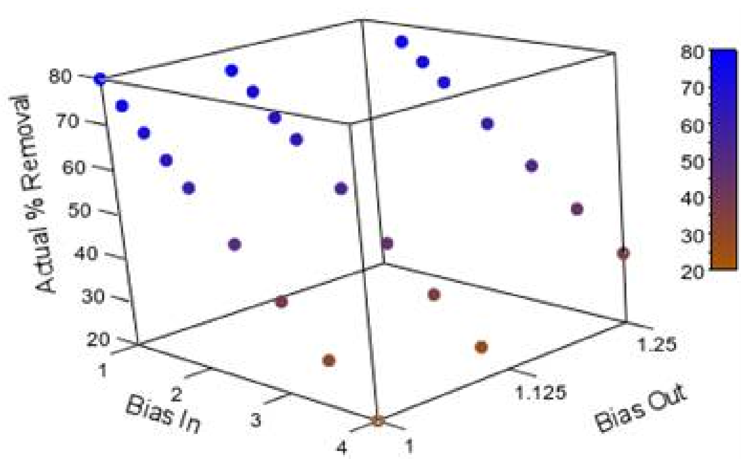


Figure 9. 'True' percent TSS removal assuming 80% apparent measured TSS removal efficiency, and depending on degree of influent and effluent sampling bias.

Gulliver and DeGroot (2010) discuss theory and a number of historic devices proposed or produced to attempt to decrease this sampling bias. Selbig et al. (Selbig and Bannerman 2011b;

Selbig et al. 2012) have developed a "Depth-Integrated Sample Arm" (DISA), "to Reduce Solids Stratification Bias in Stormwater Sampling", and verified performance in a laboratory setting. They still found overestimation, but by 49% and 7% respectively with 3 and 4 sampling points spaced vertically within the water column – compared to 96% overestimation with single-point sampling.

Figure 10 is suggestive that DISA vs. single collection point may result in different PSD curves; although these data were not collected at the same time, so curve differences could result from different pollutant buildup and storm profiles between the two data sets.

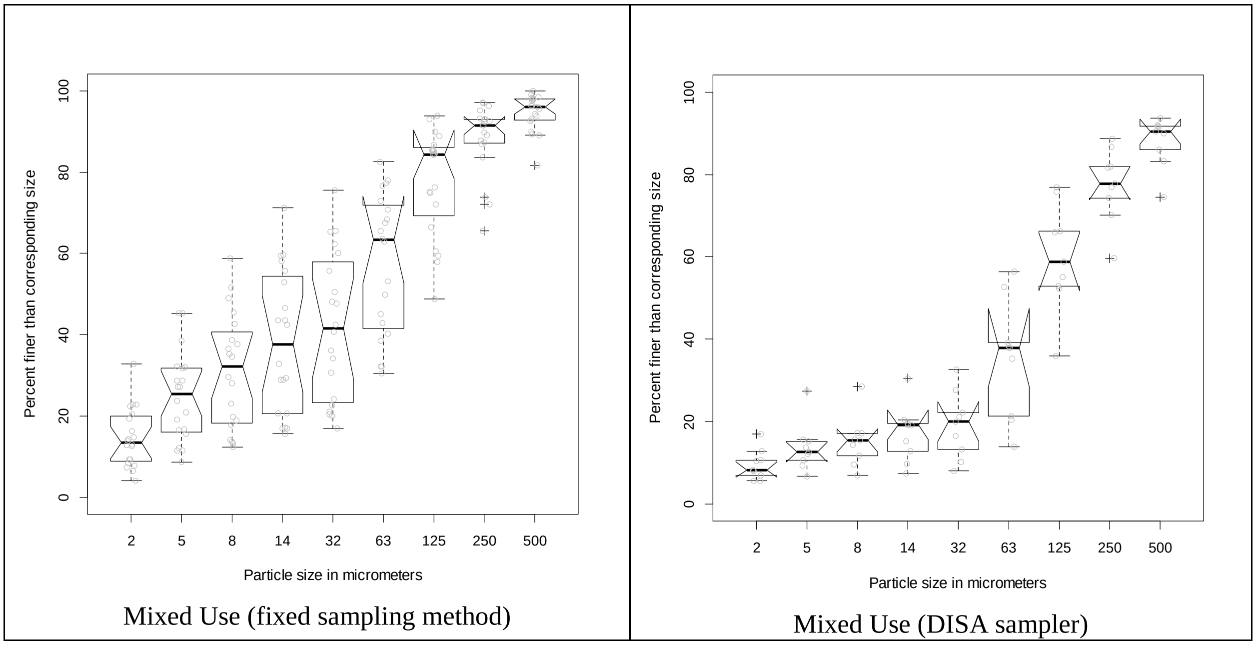


Figure 10. Fixed point and DISA sampling from the same land use; different monitoring periods Provisional data (subject to change) used to generate these graphs from (Selbig 2012a).*

While fundamental curve differences may be attributable to different sampling events, note the tighter 95% confidence intervals for particle sizes ≤ 32 *u*m with DISA sampling, even though it represents fewer sampling events than the fixed sampling method graph.

4.5.2.6.3 Sampling methodology: PSD changes during sample collection and storage

Li et al. (2008) report having shown† that "particles may grow rapidly in the first few hours after collection followed by decreased growth rate"; indicating PSD is not static in a closed basin – that particles may grow for some time both in a settling basin and in a sample collection container. This means that on account of this effect, there is some inherent uncertainty with regard to how closely laboratory measurements reflect field conditions (i.e. aside from any other uncertainty factors). To the extent that particle growth is simple flocculation or sorption of smaller particles to larger ones, TSS mg/L will not be affected. However, if particle growth is also driven by sorption of dissolved material and/or sorption of or flocculation with small precipitates of originally dissolved but then combined materials may result in increased TSS

* Graphs generated using R statistics software (The R Core development team) with R Commander (John Fox, http://socserv.mcmaster.ca/jfox/Misc/Rcmdr/)

† Citing Li et al. (2005); unable to obtain this paper in time for this review.

mass. That aside, particle growth itself will result in change in PSD. Any extent to which sorbing and/or flocculating materials may have different densities will also cause PSD changes.

4.5.3 Phosphorus

Phosphorus is highly reactive; what is found in nature is by and large phosphate complexed with other materials in minerals, and biologically in association with structural, energy and electron transfer[*], and information[†] molecules in cells and tissues. Soluble reactive phosphorus appears in stormwater as a small fraction of total phosphorus. Apatite, which constitutes a variety of calcium phosphate minerals complexed with anions, e.g. OH^-, F^-, Br^-, and Cl^- in different combinations and ratios, has a density range of 3.16 to 3.22 (Wikipedia 2011). The full range of possibilities of phosphate minerals will constitute a wider range of densities. Fletcher et al. (2004) provide some evidence that at least in their setting, the PSD curve for total phosphorus (TP)[‡] constitutes a higher proportion of finer particles than the TSS distribution of the same stormwater. In this case, the density of phosphate minerals is somewhat but not much greater than the density of silica sand; and the PSD appears to skew toward smaller sizes relative to TSS overall, but not by much. As with solid copper (below), increased density and PSD skewed smaller relative to TSS PSD may cancel each other out. Conflicting reports noted above, regarding relative proportions of TP in relation to TSS suggest inadvisability to predict TP or P_{solid} removal based on published TSS removal rates.

4.5.4 Metals

4.5.4.1 Matrix effects and speciation

The degree to which dissolved metals sorb to TSS, the degree to which sorption is particle-size dependent, and the degree to which dissolved metals complex and form precipitates that become part of TSS, affects both removal rates of those metals and overall TSS settling rates. There is more detail on this in the following section on *Matrix effects and sampling bias*; there is a continuum between matrix effects while resident in a treatment facility or BMP and matrix effects in a sample container.

Metals are widely described as being associated more closely with the finer range particles within stormwater PSDs. Hettler et al. (2009), note relatively higher metals sorption with smaller particles, but they note substantial removal of sorbed metals by precipitation with larger particles as well. In at least one case a distinction is made (Sansalone and Buchberger 1997), noting an inverse relationship between Cu, Zn, and Pb mass (but not Cd) and particle size.

McKenzie et al. (2008) note that "on a particle mass basis, anthropogenic constituents are increasingly associated with decreasing particle size". They use the term 'anthropogenic' as opposed to 'crustal' (natural geological / soils) sources of metals. Based on metals enrichment assessment of stormwater particle sizes going down to extremely fine (sub-micron down to 0.1 um) from highway runoff samples (I-80 at Davis, CA), they present a hypothetical calculation whereby "Only 4.5% of the initial particle mass would reach the receiving waters, however 65% of the particle-bound constituent would be associated with the particles that are not eliminated by the BMP". This is despite the fact that any metals portion would have a specific gravity greater

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[*] e.g. ATP (energy transfer) and NADPH (electron transfer)

[†] e.g. DNA, RNA

[‡] The authors designate "TP", yet by definition, TP relative to soluble reactive phosphate (SRP, or orthophosphate (OP)) is the fraction larger than 0.45 *u*m by filtration, and SRP is the < 0.45 *u*m fraction. We infer then that use of the term TP in the PSD curve is erroneous, and that they really mean particulate phosphorus.

than 2.65, although one can't rule out that this might be countered by lighter PAHs or other organics also favoring association with smaller particles*. Analysis was only for metals, not for any organic pollutants.

Degroot and Weiss (2008); presenting data from Li et al. (2006)* show cases where it appears that heavy metals concentrations in stormwater solids are roughly inversely proportional to particle size. Much of this data is from highway runoff sediments and street sweepings, but some is from urban suspended sediments in stormwater. Zanders, (2005) analyzed road sediment and found an inverse relationship between Cu and Zn concentrations and particle size, but found lead to be relatively insensitive to particle size, except at the extreme large particle range of 1 to 2 mm. Herngren et al. (2005)

Anthropogenic-source heavy metals enrichment associated with finer particles is further supported by Kong et al. (2012). Although this is a health risk assessment study, not a stormwater study, the route of pollutant delivery and target species are moot with regard to assessing the partitioning of heavy metals with different soil particle sizes.

Pollutant-removal rate studies rarely if ever report on the solid metals fraction alone (this author has yet to see any). However, it can be expected that of the metals-particle portion of TSS, the heavy metal particles will be denser than comparable sized silica particles. The densities of pure metallic copper and zinc are 8.94 and 7.14 $g \cdot cm^{-3}$ respectively, compared to silica sand's density of 2.65 $g \cdot cm^{-3}$. On the other hand, if metals constitute smaller particles out of the PSD, then their settling rates will be diminished. Absent empirical data, we might hold out the possibility that the higher densities could be cancelled out by the smaller particle sizes. This assumption is worth testing, but could turn out not to be true, or site-specific (more likely) and in any event that is beyond the scope of the current project.

The ISWBMPDB (2011c) indicates total copper removal at a rate 50% that of TSS removal (40% for Cu vs. 80% for TSS); but if solid Cu is a small fraction of total, a greater total Cu removal rate could be hidden in there. Using data from Sansalone et al. (2008)†, Hettler et al. (2009) find that total Cu is removed at a rate of about 92% that of TSS. This would include particulate Cu, the Cu-extractable portion of metal-containing minerals, reacted and precipitated Cu compounds, and sorbed Cu ions. Therefore, where we presume 80% TSS removal, and $Vb/Vr \geq 3$, we would assume about 74% total Cu removal, but again this does not reveal the efficiency of solid Cu removal which could be higher than the totals value. In the absence of data indicating a better removal rate for solid Cu, it seems prudent to go with a value no higher than the total Cu removal rate indicated by Hettler et al., (2009). Solid Cu removal cannot be legitimately inferred from the summary total and dissolved data. If the solid form is a relatively small proportion of total, 74% removal of solid Cu does not seem unreasonable given 40% removal of total. The following section discusses why assessing pollutant removal rates of metals is likely to be biased, making empirical evaluation of actual removal rates challenging, and published values suspect.

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* PAH is not a pollutant in the model, so these remarks are footnoted. There are conflicting opinions regarding PAH in relation to grain size. Zhao et al (2009) indicate PAH association with smaller particles, although the demarcation of 'finer' particles is at < 63 microns rather than the single-digit to sub-micron level indicated by McKenzie et al (2008) for heavy metals. Badin et al. (2008) found that PAH does not correlate well with PSD, e.g. "*Isolated grain size fractions showed dissimilarities (total organic carbon from 3.5 mg/g to 88.6 mg/g*". For eight PAHs for which specific gravity was readily found (http://www.toronto.ca/health/pdf/cr_appendix_b_pah.pdf), median specific gravity is 1.25, with a 95% CI from 1.15 to 1.35.

† Sansalone (1998) not yet obtained and reviewed.

4.5.4.2 Matrix effects and sampling bias

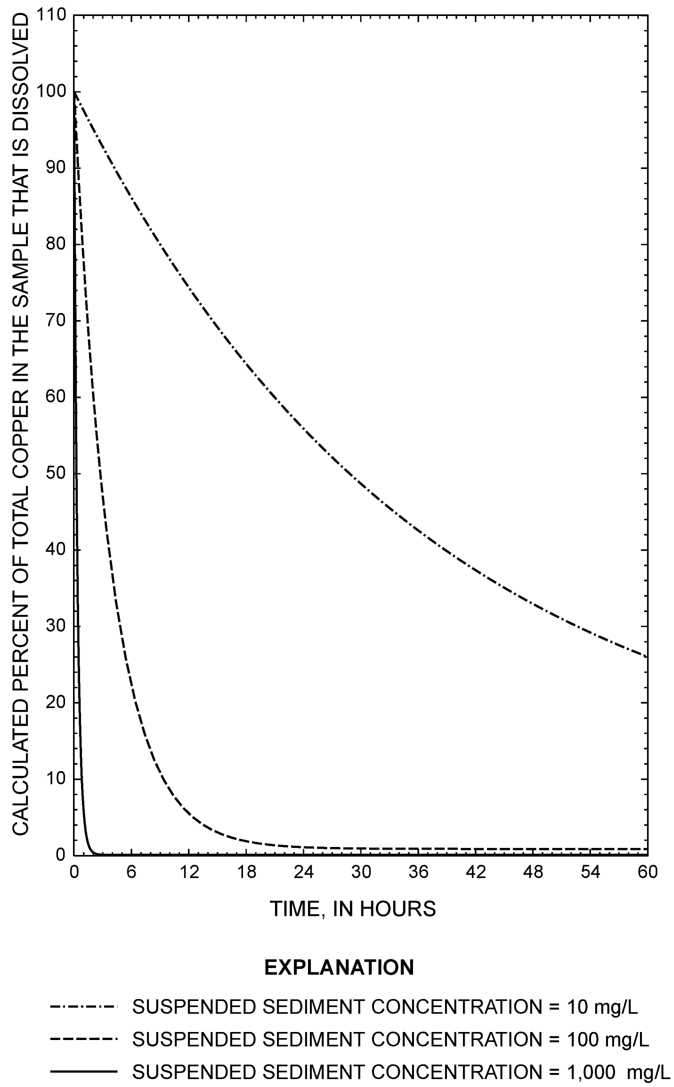

Figure 8. The percent of total copper that is dissolved in a hypothetical two-phase (water and sediment) highway-runoff sample with the median national event mean copper concentration (0.039 mg/L) and various suspended-sediment concentrations (representing a range of typical highway runoff sediment concentrations; Driscoll and others, 1990) as a function of the time from sample collection using published rate equations and rate constants for natural particles in fresh water (Wood and others, 1995) assuming that all of the copper is initially in the dissolved state.

Figure 11. Theoretical decrease in sample dissolved copper over time
(Figure 8 from Breault and Granato (2003)).

According to Breault and Granato (2003), "The existence of dissolved-matrix sampling artifacts raises serious questions about the generation of accurate, precise, and comparable dissolved trace element data and casts doubt on the utility of substantial amounts of historical data, especially in the context of a regional or national-runoff monitoring program."

Consider the theoretical decrease in sample dissolved copper as a function of TSS concentration and time in the absence of competing reactions (Figure 11). Assuming a grab sample, with 100 mg/L TSS and an initial 39 *u*g/L dissolved Cu, within the first six hours of holding time there will be about an 80% apparent decrease in dissolved Cu. With a 24-hour storm and an EMC* autosampler composite, the portion of dissolved Cu collected near the beginning will have decreased to virtually zero; and when the sample is collected , it will represent a mixture of aliquots with varying degrees of decrease, even if filtered within EPA's 15-minute holding time. Any holding time beyond this will exacerbate the problem; but at best, we have to consider the possibility that historic and even current stormwater data are biased low for dissolved metals.

This has implications for modeling, compliance, and toxicity testing. With regard to the latter, recent NPDES required stormwater toxicity testing allowed up to a 36-hour holding time. It seems prudent to re-evaluate this allowance. It also bears questioning the

* Event mean concentration, typically composited from a flow-weighted series of aliquots.

utility of follow-up chemical analysis after a finding of toxicity. This potential decrease in dissolved metal over the course of sampling and holding also makes apparent a substantial need for research to track this empirically, and if the theory is validated, development of a field autosampler capable of in-situ filtration for each aliquot as it is collected. This would also benefit collection of soluble reactive phosphorus (orthophosphate) samples, which also require filtration within a short time frame.

4.5.5 Total vs. solid and dissolved fractions – metals and phosphorus

Where dissolved/solid/total speciation is given, both metals and phosphorus appear to almost always be reported as total and/or dissolved*. Some reports – especially older ones, simply report phosphorus or 'metals'. In the absence of any designation or description of sample collection (e.g. 0.45 micron filtration implies dissolved fraction), it is tempting to assume those reported values must be total, but it is more prudent not to assume, and to not use those data. Where total and dissolved are both reported, on a per-sample basis the solid fraction might be presumed to be the difference between total and dissolved masses or mass concentrations, but concentration differences between sample splits for filtration, and possibly different matrix interferences for the filtered and non-filtered portions, can lead to substantial errors – even leading to the possibility of getting a higher dissolved concentration than total, particularly at low concentrations. In one case (Davis et al. 2003), dissolved metals are identified by 0.23 micron filtration, rather than 0.45 microns, which is likely to result in different percent removal rates than if filtered at 0.45 microns, and demonstrating another source of uncertainty when dealing with results that are based on data reported simply as 'dissolved'. Pressure applied during filtration can introduce variability in results as well.

4.6 Percent Removal as a Function of Concentration

Questions about percent removal as an appropriate metric notwithstanding, from a regulatory point of view, percent removal is still the rubric. Both WA Ecology and King County note that TSS removal efficiency is concentration-dependent, and this notion is ubiquitous in the current stormwater management and research community. According to Ecology, "Ecology's basic treatment menu facility choices† should achieve 80% removal of total suspended solids for influent concentrations ranging from 100 to 200 mg/L. For influent concentrations greater than 200 mg/L, a higher removal efficiency is appropriate. For influent concentrations less than 100 mg/L, the facilities should achieve an effluent goal of 20 mg/L total suspended solids" (O'Brien et al. 2005). King County's assessment differs. "For evaluation purposes, according to King County, typical concentrations of TSS in Seattle area runoff are between 30 and 100 mg/L (Table 1, "Water Quality Thresholds Decision Paper," King County Surface Water Management Division, April 1994)". (King County 2009).

If not negatively biased‡, data from King and Snohomish counties' required NPDES monitoring for the 2007-2012 permit suggest that the King County assumed range might be more applicable

* This author has never seen a case where the solid metal fraction concentration is given for a liquid solution.
† Per WA Ecology's Stormwater Management Manual for Western Washington, Vol. 5 (O'Brien et al. 2005)
‡ If the sample pickup points were elevated, samples could have been biased to miss the heavier/larger particles. Autosampler peristaltic pumps may also be limiting in this regard. Either can affect both concentration and particle size distribution.

at least some land uses[*] than Ecology's range (Figure 12); and this may have implications with regard to percent TSS removal.

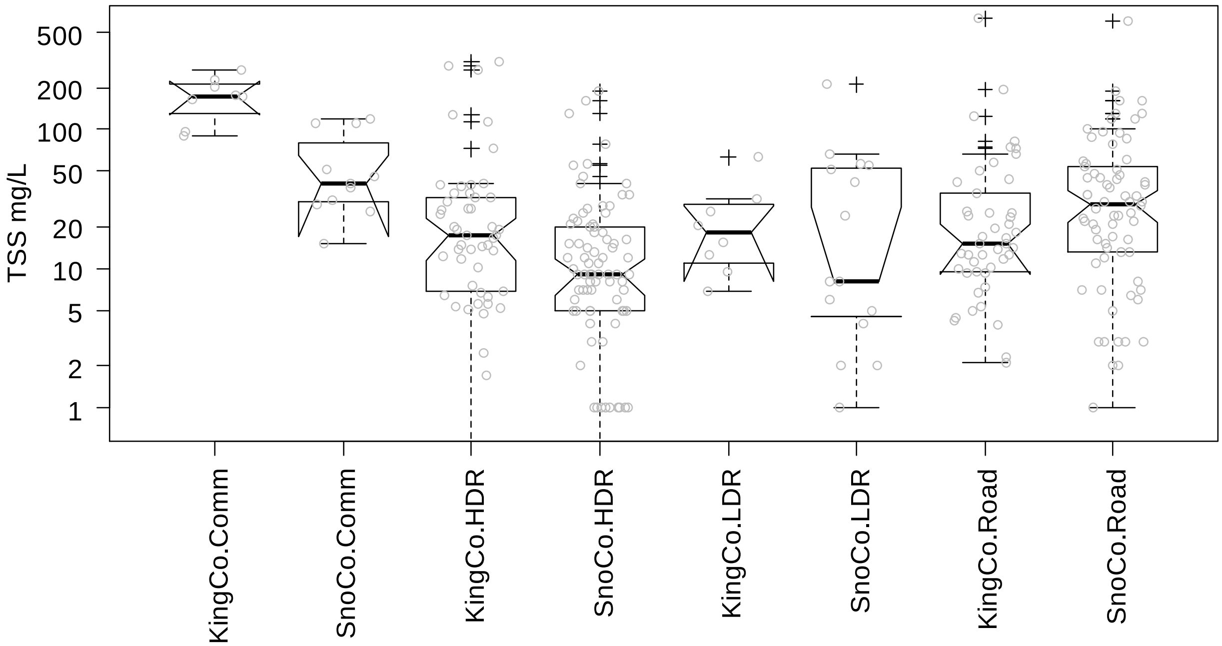

Figure 12. TSS data from King County (KingCo) and Snohomish County (SnoCo) NPDES monitoring. 2007-2012 permit cycle (Herrmann 2012). Snohomish County data are provisional and subject to change[†].. Notched boxplots indicate quartiles and estimated 95% confidence intervals (CI). Plus marks are statistical outliers. Abbreviations are Comm = commercial, HDR = high density residential, and LDR = low density residential. LDR is defined as ≤ 4 single-family houses per acre. The SnoCo LDR notched boxplot area below median is missing lines as a consequence of an undocumented bug in the R statistics boxplot module when including CI and log scale. The y-axis is truncated at the highest TSS laboratory lower reporting limit (0.5 mg/L); there are only three non-detects below this level.

WA Ecology's TAPE[‡] protocols (Ecology 2011) specify influent ranges for TSS, TP, dissolved copper and zinc, total phosphorus, and 'oil treatment', with specific treatment requirements for these influent ranges. For TSS Ecology's stormwater manual says "The performance goal assumes that the facility is treating stormwater with a typical particle size distribution. For a description of a typical particle size distribution, please refer to the stormwater monitoring protocol on the Department of Ecology website." However, there is no specific citation for a stormwater monitoring protocol; and other than TAPE, which specifies 'typical' particle size ranges but not fractions (so no distribution is given), there is no immediately obvious stormwater monitoring protocol found by searching the site. Absence of particle size distribution data makes published TSS percent removal rates and calculations based on published concentrations subject to skepticism with regard to comparability. The same may be said for total phosphorus as well, as it is mostly particulate solids.

[*] King County's high density land use data includes runoff from an apartment complex in Sammamish – that had recently been within King County, but was annexed by the city.
[†] These data have not undergone QA/QC scrutiny, and as such, are subject to change.
[‡] Guidance for Evaluating Emerging Stormwater Treatment Technologies, Technology Assessment Protocol - Ecology (TAPE)

4.7 Treatment Train Effects

Whenever a low-infiltration rain garden pond fills and there is surface discharge to a regional wet pond, we have a treatment train. To the extent that influent concentration is a factor in percent removal creates a challenge in this regard. Treatment by a rain garden wet pond will result in effluent that is low in TSS concentration and has a particle size and density distribution that is skewed toward very fine and /or low density particles, diminishing further removal efficiency compared to an untreated distribution. The question is whether mixing in a regional pond with higher untreated influent TSS concentration with a 'typical' PSD for the area will cause less than optimal pollutant removal from the untreated portion, or whether that is counterbalanced by 'polishing' of the portion from the rain garden pond. But if the latter is primarily very fine particles, unless flocculation is a factor, further removal by settling may be very limited.

Treatment train questions only arise where and when a regional wet pond is being fed in part or in whole by one or more of the rain garden wet ponds on low infiltration soil, and even then only during rain events with an extreme product of intensity and duration. In the Juanita model 80% of a catchment is served by rain gardens, and the remaining 20% of untreated runoff goes directly to regional wet pond. With Vb/Vr = 7, the 0.15 in/hr infiltration rain garden wet ponds will rarely discharge to the wet ponds, and the 3 in/hr infiltration rain gardens are not expected to discharge to the regional wet pond at all.

For the current modeling effort, treatment train effects are not factored, but it seems like they ought to be considered in the future; i.e. we should consider what the second facility can realistically achieve, and at what cost, compared to other options.

4.8 Infiltration and Treatment Media

This author has seen several comments in the literature that it is common for short-term laboratory column and pilot studies to show better pollutant removal rates than field monitoring of actual installations. In addition, Mikula et al. (2007) note,

> ". . . it has been shown in past filter work that the media can be a source of pollutants either due to the release of previously-trapped compounds or of compounds contained in the media itself. It has been well-documented that small concentration gradients between the media and the pollutants in the water results in weak removals, and that when media concentrations of a pollutant are greater than those in the passing water, negative removals occur."

The quality and condition of bioretention media will have an effect on the effectiveness of the media to remove specific pollutants, and will affect the longevity of the media, affecting replacement cycle costs. While there are pollutant-removal data from rain gardens and bioretention facilities, there is a large data gap with regard media pollutant concentrations. In order to assert that compost that marginally meets WAC 173-530-220 heavy metals criteria is as effective at removal of those same metals from stormwater as is compost containing much lower heavy metal pre-loading requires testing that to the best of this author's knowledge has not been done. We do not know what metals levels are acceptable from a broad spectrum of compost from a large number of different sources. The absence of initial media pollutant content and long-term leachate monitoring studies leaves a large shroud of uncertainty around actual long-term bioretention pollutant-removal rates. In addition, there are concerns regarding media containing compost being a net exporter of some nutrients and metals.

5.0 Pollutant Removal Rates

5.1 Assumptions and Observations

The same cautions noted above apply to our current evaluation as well. On one hand, according to Wilgus (2011b), absolute percent removal rates are not critical, because we are modeling different scenarios relative to each other; as such, relative performance (which scenario is better) may be evaluated. On the other hand, different assumptions about pollutant removal performance by full sized wet ponds vs. 'rain gardens' viewed as small wet ponds may affect conclusions re: which scenario is best.

Pollutant percent removal data are usually collected during storms that fall within 'design storm' criteria; i.e., by and large – if not exclusively – they represent percent removal when 100 % of the flow volume is routed through the treatment facility. Percent removal rates do not represent treatment during bypass, at which time the non-bypassing portion will still be treated, but the bypassing runoff volume will be treated to a lesser degree, diminishing as flow rate increases and as bypass becomes a larger proportion of total runoff from the facility. As a wet pond transitions from quiescent to dynamic settling as flow increases during an event, removal efficiency of larger particles increases while that of finer particles decreases (US EPA 1986). This is further complicated by the often cited observation that pollutant removal efficiencies are generally assumed to increase as influent pollutant concentrations increase (e.g. see Claytor and Schuler (1996), pg 2-22, citing "(Bell, et. al, 1995)"); yet at the highest storm flow rates, some pollutant concentrations may drop as a consequence of dilution, while loads, which are independent of dilution effects, increase. Unless otherwise noted, pollutant removal values are usually reported as concentration reductions, not load reductions.

5.2 Total Suspended Solids (TSS)

5.2.1 TSS presumptive approach for Regional Wet Ponds

For this exercise, for conventional wet ponds built to design standards, we are presuming design-manual 80% TSS removal where $Vb/Vr \geq 3$, but this itself is not certain for reasons discussed in Section 4.0, *Sources of Uncertainty*. At $Vb/Vr = 0.75$, the presumption is 50% TSS removal with the same caveat. We are not giving credit for more removal if/where pond $Vb/Vr > 3$; i.e., for the rain garden wet ponds. We are also presuming 0% removal as Vb/Vr approaches zero, but can only approach zero ($Vb/Vr = 0.1$) with a continuous curve because a simple curve fit (using MS Excel 2003) is a log function (Figure 13). Before going there, the reader is cautioned that this is only applicable using the King County Surface Water Design Manual methodology and definition of Vb/Vr [*].

[*] The ratio of the pond volume Vb to the volume of runoff from the mean annual storm Vr, where Vr = mean annual storm depth x runoff coefficient. King County's methodology for calculating Vr and Vb is given in its 2009 Surface Water Design Manual, pages 6-70 – 6-72.

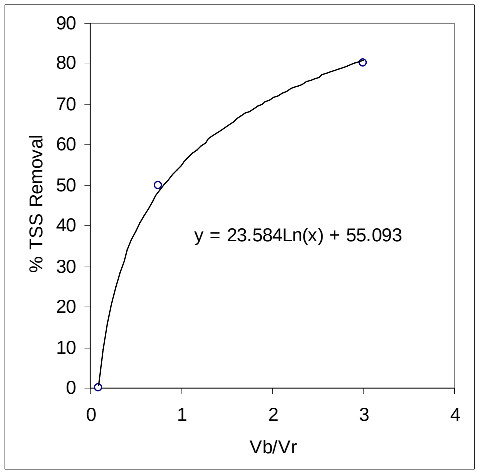

Figure 13. Percent TSS Removal as a function of Vb/Vr (x = Vb/Vr)

We are not attempting to predict speciation effects or reactions or interactions between pollutants in the treatment train. Depending on aerobic vs. anaerobic conditions, and redox state, temperature, and pollutant concentrations, for soluble pollutants, the ratio of solid (generally not bioavailable) to dissolved (generally bioavailable) pollutant may change during pass-through. There will also be interaction with the pond sediments, where the same conditions noted will affect chemistry. In addition, chemical complexes can for and dissociate. Further, the diminishing of TSS concentration and the skew toward smaller PSD average size in the second facility wet ponds will likely result in lower sorption rates of other pollutants to TSS because lower concentrations will result in fewer interactions in any given period of time.

WA Ecology's SMMWW (2005, vol. V), indicates in Table 2.2, TSS removal is the only major pollutant removal process in wet ponds. Dissolved metal, total phosphorus, pesticides/fungicides, and hydrocarbons removal are all listed as minor processes. There is no indication of ability to remove nitrogenous nutrients or bacteria.

5.2.2 TSS modeling approach for Rain Garden Ponds

Referring back to design from Section 2.1.3, the rain garden ponds over high infiltration soils (3 in/hr) are expected to function solely as infiltration ponds with no surface discharge. The rain garden ponds over low infiltration soils (0.15 in/hr) are expected to function primarily as infiltration ponds, but during prolonged precipitation of some intensity, some surface discharge is expected. Minton (2011) notes that "stormwater treatment systems that are dry between storms experience only the dynamic settling process". At Vb/Vr = 7, we expect these ponds to fill rarely if ever, and at 0.15 inches infiltration per hour, a filled pond with no sediment build-up will drain down fully in 80 hours, which is only ½ hour longer than the average antecedent dry period*. Consequently these are modeled here as dry ponds with exclusively dynamic settling.

There are a number of sources of equations for modeling settling rates of soil particles (Fentie et al. 2004; Jiménez and Madsen 2003) and mixtures of natural and anthropogenic particulates found in stormwater (Minton 2011; MOEE 1994; Papa et al. 1999; US EPA 1986). This is not an exhaustive list. For dynamic settling, MOEE (*ibid*) uses "the method presented by Fair and Geyer which is the standard methodology approved by the U.S. EPA for detention basin analysis

* As determined by US EPA (1983)

(USEPA, 1986)" – noted by Pitt (2005) as "the basic Hazen theory presented by Fair and Geyer (1954) that considers short-circuiting effects"; which is:

$$R_d = 1 - \left(1 + \frac{1}{n} \cdot \frac{V_S}{Q/A}\right)^{-n}$$

Rd = TSS removal* under dynamic conditions (m^3/m^3)
Q = average of inflow and outflow (m^3/s)
A = surface area of pond (m^2)
V_S = settling velocity of particulate matter (m/s)
n = turbulence factor†; see Figure 1 and associated text

Papa et al. (1999) rework this as

$$E_d = \sum_i F_i \left\{ 1 - \left[1 + \frac{V_{si} \cdot S_A}{nh_A \cdot 2\Omega} \right]^{-n} \right\}$$

E_d = overall % TSS removal
Fi = decimal fraction of total mass within the *i*th particle size fraction (range)
S_A/Ω = detention time when inflow rate > outflow rate and the pond is full (symbols not defined individually)
V_{si} = average settling velocity of total mass within the *i*th particle size fraction (range)
t_s = is the average detention time (hours) of the active storage zone
t_d = drawdown time to drain a full pond
$t_s = ½\ t_d$
n = turbulence factor†; see Figure 1 and associated text
h_A = pond depth (m)

Papa et al. having previously given

$$t_s = \frac{1}{2} \cdot td = \frac{1}{2} \cdot \frac{S_A}{2\Omega}$$

The overall equation can be simplified to:

$$E_d = \sum_i F_i \left\{ 1 - \left[1 + \frac{V_{si} \cdot t_s}{nh_A} \right]^{-n} \right\}$$

Fentie et al. (2004) note that "the choice of suitable formula depends on the sediment size range under investigation". Particle density and shape heterogeneity should also be factors. Absent PSD data representing the Juanita watershed basin, we rely here on the model above, using widely used but likely not very representative data, and consider this in our evaluation. Applying F_i and V_{si} from MOEE (1994) as given in Papa et al. (1999), with a pond depth of 0.3048 m (1 ft) and assuming a turbulence factor of 3, we get 91% TSS removal. Applying the same equation to road runoff and high density residential PSD provisional data obtained from Snohomish County (Herrmann 2012), and applying settling velocities‡ from MOEE (1994) and Li et al. (2008)*, we get lower efficiency values.

* proportion of particles removed having settling velocity V_S

† Inversely proportional to turbulence (MOEE 1994); they assign default n=3 for dynamic settling and n=5 for quiescent settling. Also referred to as *short circuiting factor* (Papa et al., 1999), and *short-circuiting factor (number of hypothetical basins in series)* (Pitt et al., 2005).

‡ The Snohomish County (SnoCo) provisional data did not include settling velocities, and the PSD range definitions did not match either MOEE (1994; NURP 1983 data) or Li et al. (2008), which also differ from each other. In order to estimate settling velocity (Vs) for the SnoCo data, for each of MOEE and Li et al., Vs was regressed against particle size (Li et al.) and average of particle size range (MOEE; using averages of PSD range categories to obtain single values for regression. MOEE data regressed with and without the high end 4 mm particle size; without as it is likely far out of range of SnoCo max PSD value of > 50 microns; while the latter has no upper limit, the range represents a very small fraction of the SnoCo PSD, and given the rest of the distribution, 4mm would likely represent an extreme outlier for SnoCo). The regression equations were then applied to the SnoCo PSD range averages (a high end of 500 microns was assumed for > 50 *u*m). As noted with caveats in a prior

PSD data source	TSS removal efficiency settling rate basis		
	MOEE (1994) Vs regression against full PSD range	MOEE (1994) Vs regression against PSD range less 4mm max	Li et al. (2008) Vs regression against their own PSD range
MOEE	91%	---	---
SnoCo Center Rd	86%	53%	68%
SnoCo Gibson Rd	87%	56%	69%
SnoCo Emerald Meadows	87%	54%	68%
SnoCo Snohomish Cascade	86%	54%	68%

Table 4. Predicted rain garden pond TSS removal efficiency
Based on Snohomish County (SnoCo) PSD data, NURP (1983) PSD and settling velocities (Vs) as reported in MOEE (1994), and PSD and Vs reported in Li et al. (2008).

Retaining out of range values for regressing V_S against particle size can, as expected, have a large effect on predicted pollutant removal by settling. Predicted TSS removal efficiency indicated in Table 4 above is based on midrange PSD values from SnoCo, except at the upper end, which is open-ended, a maximum particle size of 500 microns is assumed here. That value is inconsequential in this case however; substituting 5000 microns (changes midpoint value from 275 to 2525) has no effect on outcome because this PSD range represents less than one percent of the overall PSD. At the low end however, the 0.5-5 micron range represents up to 82.8 percent of the particle mass (at the Center Rd sampling location). The midpoint of that range is 2.75 microns. Depending on what actually best represents the range, predicted percent removal may change substantially (Table 4 and Figure 14).

		TSS removal efficiency settling rate basis		
	Assumed min PSD midpoint microns	MOEE (1994) Vs regression against full PSD range	MOEE (1994) Vs regression against PSD range less 4mm max	Li et al. (2008) Vs regression against their own PSD range
MOEE	10	91%	---	---
Snohomish County	4	92%	66%	72%
	2.75	87%	56%	68%

Table 5. Predicted rain garden pond TSS removal efficiency showing effect of effect assumed minimum PSD range midpoint.
Same basis as noted in Table 4 above, except Snohomish data are averages for all four sites.

prior footnote, % volume by laser diffraction is assumed to be equivalent to % mass for any given particle size range.

* Vs obtained by extraction from graph

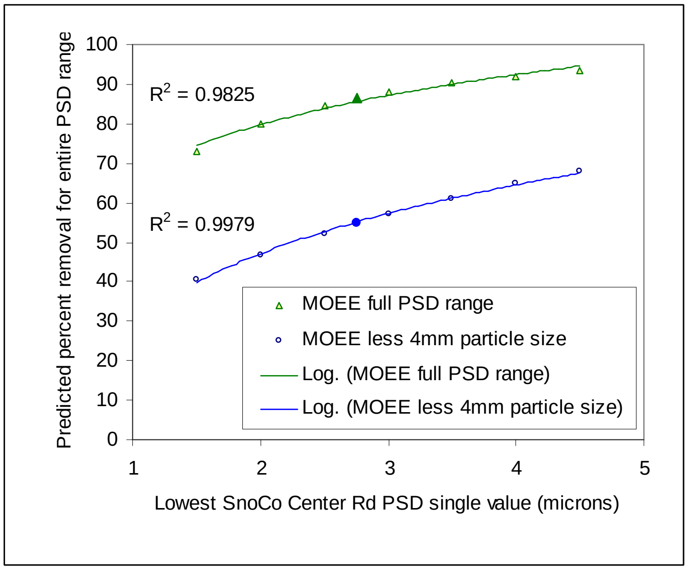

Figure 14. TSS percent removal as a function of particle size range representation

The figure above also indicates the effect of including the 400 to 4000 micron range (midpoint is 2200) when regressing settling rate vs. particle size, with the MOEE data. In this case, the effect is large, because this represents 20% of the particle mass.

In reality, during the wet season in this region, there will usually be some prolonged periods when the pool holds water at some depth, but only with continued precipitation and flow; i.e., that which sustains pool depth rules out quiescent settling. Even with the lower infiltration rate of 0.15 inches per hour these ponds will drain as infiltration ponds between most storms. At the 3 inch per hour infiltration rate, the rain garden pond is effectively simply an infiltration pond, with no surface discharge.

Given the known high variability in stormwater, and unknown specifics regarding particle shape variability and size and density distribution in the Juanita basin, it seems prudent to use a value lower than the MOEE prediction of 91%, but a bit higher than the lowest Snohomish PSD prediction using settling rate from one of the MOEE regressions (56%). Using Snohomish PSD and obtaining settling rates from Li et al. (2008), we get 68 to 72% removal efficiency assuming 2.75 and 4 microns respectively represent the low PSD range. Given that the Snohomish PSD appears skewed very much to smaller particles, and thinking that might be extreme and not be representative of the Juanita basin, a judgment call of 70% is made, acknowledging this is not an empirical or statistical determination, and there is considerable uncertainly about it.

5.3 Phosphorus

5.3.1 Total Phosphorus presumptive approach for Regional Wet Ponds

The King County Surface Water Design Manual claims 50% total phosphorus (TP) removal when Vb/Vr = 4.5. From a presumptive point of view, both King County and WA Ecology require either larger than normal basic facilities – e.g. large wet ponds (Vb/Vr = 4.5) – or treatment trains to achieve the target 50% TP removal. Referring back to our presumptive approach for TSS in Section 5.2.1, applying same approach to TP which is mostly particulate

solids, and presuming 50% removal at a Vb/Vr = 4.5, 0% removal as Vb/Vr approaches zero, and assuming the same log relationship, we get the following curve (Figure 15).

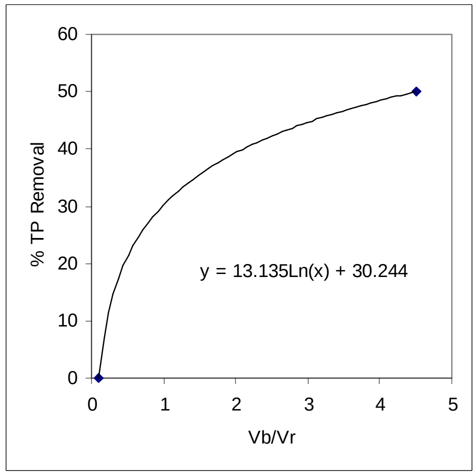

Figure 15. Total phosphorus removal as a function of Vb/Vr
(x = Vb/Vr)

Plugging in x = 3 for Vb/Vr gives us 45% TP removal, which is somewhat discouraging from a design manual point of view; i.e. if this is true, we only get a marginal efficiency increase of 5% going from Vb/Vr = 3 to Vb/Vr = 4.5.

5.3.2 Total Phosphorus empirical, other approaches, and observations

5.3.2.1 Wet ponds

Fletcher et al. (2004) indicate > 50% TP removal with non-specified design wet ponds. The International Stormwater BMP Database (Geosyntec Consultants and Wright Water Engineers (2010a, b) in summarizing 38 studies with 578 data points indicate 57 - 59% TP removal*, with effluent concentration significantly† lower than influent. Hafner and Panzer (2011)‡ suggest§ 12-13 days pond residence time for 50% TP removal, and about 1 day required for 10% removal. These rates appear to be not affected much by pond depth. On the other hand, (Minnesota Pollution Control Agency 2008), in their stormwater manual, assign an average TP removal rate of 50% for wet ponds. We have not had the time to evaluate whether their pond sizing for achieving this rate is equivalent to current WA design standards for large wet ponds. Absent time to obtain and evaluate the studies individually, we cannot know how anyone else's wet ponds are sized relative to WA Ecology and King County requirements for basic treatment and phosphorus removal. We also don't know if sampling bias or dilution by additional surface or groundwater input may have inflated some of these results; on the other hand of the results, some could be compromised in either direction. For this modeling exercise, given the uncertainties in empirical data, for this exercise we are going to use the modified presumptive approach indicated above in Section 5.3.1.

---

* 59% TP removal for median and 75th %ile; 57% removal at 25th%ile
† As determined by non-overlapping 95% confidence intervals
‡ University of MN stormwater web site
§ Based on Figure 1. (graph) on the cited web page

5.3.2.2 Rain garden wet ponds

The stormwater particle size distribution curves indicated in Figure 16 and Figure 17 below from Fletcher et al (2004)* are claimed to be for typical Melbourne and Brisbane catchments. The data are from field monitoring in Melbourne and are assumed by the authors to be representative of both Melbourne and Brisbane. Figure 16 is for fully developed catchments.

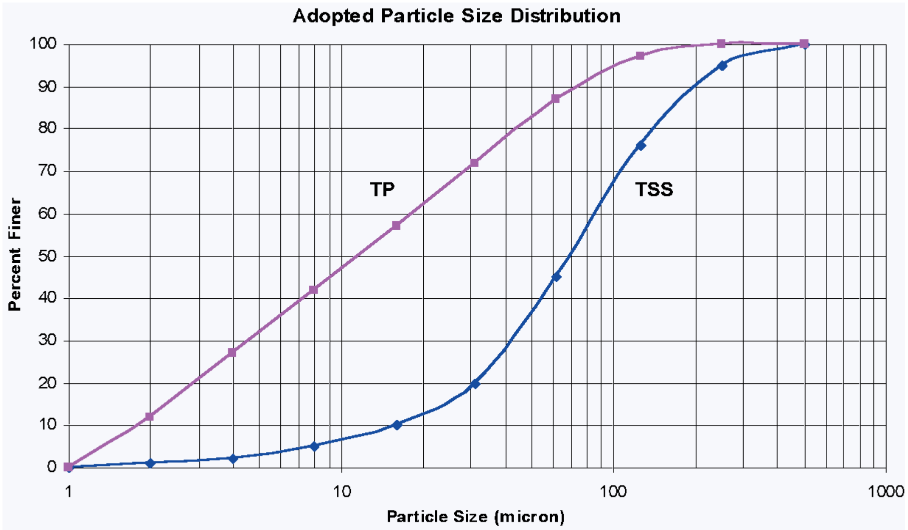

Figure 16. Possible PSD for Melbourne and Brisbane fully developed catchments.
adapted from Lloyd et al, 1998 in Fletcher et al., 2004.

It is important to note that while TP includes soluble reactive phosphorus, the authors presume a high proportion of TP is particulate (Fletcher et al. 2004). The authors also present curves to represent TSS and TP in developing catchments in Melbourne and Brisbane (Figure 17).

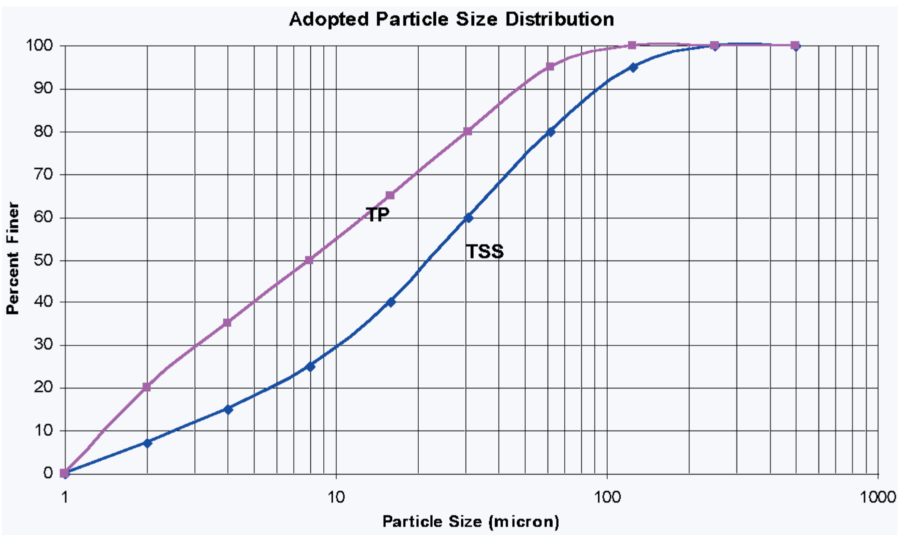

Figure 17. Possible PSD for developing catchments in Melbourne and Brisbane.
adapted from Lloyd et al, 1998 in Fletcher et al., 2004.

* Figure III-1; "(adapted from Lloyd et al., 1998)"

The utility of these graphs here is that they provide total phosphorus (TP) PSD curves, for which there seems to be short supply, and they indicate TP PSD relative to TSS PSD. TSS data in both graphs fit WA Ecology's definition of TSS as being "all particles smaller than 500 microns in diameter" (Hoppin 2008). For the rain garden wet pond, viewing these curves as representing particulate TP gives us the opportunity to follow the same assumption of dynamic settling and choice of analysis applied to TSS (Section 5.2.2).

Extracting PSD fractions from the graphs in the same particle size ranges as the Snohomish data, and applying settling rates derived from MOEE (1994) and Li et al. (2008) to the dynamic settling equation in Section 5.2.2, we get respectively 93% and 82% TP removal efficiency for developing catchments and 76% and 67% for fully developed catchments. These exceed the 57-59% ISWBMPDB TP removal rate (Geosyntec Consultants and Wright Water Engineers 2010b; ISWBMPDB 2011c) for conventional wet ponds, and greatly exceed 24% for detention basins (dry, grass lined (ISWBMPDB 2011c)). We can consider that while conditions favor dynamic settling, dry basins have bottom outlets and the rain garden ponds do not; so higher removal efficiency is expected from the latter. What we have are functionally infiltration ponds with overflow, and the ISWBMPDB does not have that as a category.

Using the same means to calculate TSS removal for the Fletcher et al. (*ibid*) data for fully developed catchments, yields 97% and 90% TSS removal efficiency using derived MOEE and Lie et al. (*ibid*) settling rates. While these values seem high, it is important to recall that the calculations are based on settling rates from entirely different sources. That said, Fletcher et al., TP removal is about 76.5% of TSS removal. Appling this factor to the 70% TSS removal obtained using the Snohomish data yields 54% TP removal.

5.3.2.3 Rain garden infiltrate

The ISBMPDM (2011c) indicates a bioretention with underdrains percent removal efficiency of 7% for total phosphorus. Comments from the TAPE Board of External Reviewers indicate concern regarding export of phosphorus (speciation not specified) from compost-amended facilities (Howie 2011). This was also raised as a concern at a recent low impact development research annual review at Washington State University, Puyallup (Hinman 2012). As net phosphorus export may occur from the compost media, and speciation is variable and site-specific, rounding down to 0% is appropriate.

5.3.3 Soluble Reactive Phosphate ((SRP), aka OrthoPhosphate (OP))

The Stormwater Database reports median SRP removal as 11% (2008) and 64% (2011c). Pitt (Pitt, R.M., 2003. Stormwater Quality Controls in WinSLAMM. Chapter 4) cites "(Stanley 1996) as finding wet pond phosphate removal rates ranging from -5 to 36%; however, checking the original source material, data are for a dry retention pond, not for a wet pond; still, this may be indicative of performance from a rain garden pond, which as noted earlier is closer to a dry pond in hydrology than to a wet pond.

5.4 Nitrogen

5.4.1 Nitrate

The Stormwater Database reports median nitrate removal as 36%. Pitt (Pitt, R.M., 2003. Stormwater Quality Controls in WinSLAMM. Chapter 4) cites "(Stanley 1996) as finding wet pond ammonia N removal rates ranging from -52 to 21%; however, as with SRP above, checking the original source material, data are for a dry retention pond, not for a wet pond; still, this may

be indicative of performance from a rain garden pond, which as noted earlier is closer to a dry pond in hydrology than to a wet pond.

Limited King County site characterization data (2011) indicates a median NO_2^-+NO_3^-:TKN ratio of 0.25, but the 75th percentile ratio is 1.44 (more nitrite-nitrate than TKN), and the extreme is a ratio of 4.8. This confirms errors introduced by sample splitting and different analytical methods; i.e., nitrate-nitrate cannot actually be higher than TKN, so these ratios > 1 represent sampling and analysis artifacts.

Some bioretention research indicates that anaerobic conditions – especially in organic matter – can result in net export of nutrients (Clark and Pitt (2009), Hatt (2007), and Hunt et al. (2006). Clark and Pitt (2009) find that N nutrients are released under anaerobic conditions, while P nutrients and metals are not. Other reading more often indicates for hypoxia/anoxia – P release, no effect on metals retention/release, and N speciation toward denitrification with N_2 release; although some other papers suggest that variable redox conditions including some period of hypoxia/anoxia is what promotes denitrification. Reading also suggests that decreasing nitrate can result in increased NH_4^+ release. Bearing in mind that these studies are of bioretention cells rather than wet ponds, we should consider that anaerobic conditions may be possible in the rain garden media, and in the sediments at the bottom of the conventional ponds, and that since there is some hydraulic interchange between rain garden media and pool water, and wet pond sediment and pool water, we cannot rule out that there may be conditions where more ammonia and/or nitrate will leave these ponds than enter. Therefore, the ISWBMPDB (2008, 2011c) data notwithstanding, it is prudent to temper their pollutant removal efficiency and assume it should be no greater than TKN removal efficiency, which for the ISWBMPDB is 13.5%. Another consideration is that while there is a basis for removal efficiency for the solid portion of TKN (e.g. plant detritus), there is little basis for nitrate removal, as it is a highly mobile, not particularly reactive molecule.

5.4.2 Ammonia/ammonium (NH_3/NH_4^+)

The Stormwater Database does not present any summary data for ammonia/ammonium (NH_3NH_4). Pitt (Pitt, R.M., 2003. Stormwater Quality Controls in WinSLAMM. Chapter 4) cites "(Stanley 1996) as finding wet pond ammonia N removal rates ranging from -66 to 43%; however, as with SRP and nitrate above, checking the original source material, data are for a dry retention pond, not for a wet pond; still, this may be indicative of performance from a rain garden pond, which as noted earlier is closer to a dry pond in hydrology than to a wet pond.

5.4.3 Total Nitrogen

(ISWBMPDB 2011c) BMP Performance Data Summary Table:
Wet Ponds: 27%
Bioretention (underdrain): 21%

5.5 Bacteria

In a wet pond, potential bacteria removal processes include sedimentation, aggregation between bacteria with each other and with other matter, e.g. clay and silt, predation, and die-ff. Bacterial removal may be countered by interaction between the water column and bacteria sequestered in sediment, bacterial growth, turbidity and/or water depth shielding bacteria from damaging UV sunlight, shading by overhanging vegetation, and secondary input, e.g. waterfowl defecation.

5.5.1 Bacteria die-off

Per Minton (2005), die off mechanisms "include natural dieoff, predation by other bacteria and higher organisms such as nematodes, ultraviolet radiation, and exposure to toxins from microorganisms and plants". Minton asserts that the die-off rate is unaffected by temperature "(unless very low)", but does not define "very low". US EPA disagrees, stating "Temperature plays an important role in microorganism die-off and has often been cited as the most important environmental factor"(US EPA 2006). Minton posits a die-off first order reaction rate:

$$C = \frac{Co}{\left(kH_{RT} + 1\right)^N}$$

Where C = effluent count
Co = influent count
k = die-off rate
H_{RT} = hydraulic residence time
N = number of cells in facility

The formula can be rearranged to calculate the expected treatment efficiency:

$$\frac{C}{Co} = \frac{1}{\left(kH_{RT} + 1\right)^N} \qquad \text{where percent removal efficiency} = \left(1 - \frac{C}{Co}\right) * 100$$

Minton suggests k values from 1 to 5, and considers 2 to be average; and says these values hold for E coli, fecal streptococci, and total coliforms – so presumably for fecal coliform as well. Unfortunately, units are not given for k or time. Whether time is hours or days (and k is h^{-1} or d^{-1}) substantially affects predictions. Plugging in k = 2 (Minton's 'average' value) and a range of 1 to 24 for H_{RT} gives C/Co = 0.5, or 50% die-off at one hour and 90% in five hours. Off-hand this seems unlikely, given US EPA's note (2006) that for *E coli*, e.g., one study showed survival for at least 28 *days*, and another showed 36 hours needed at 10 deg C compared to 8.4 hours at 42 deg C to decrease the population by 90%. Changing k to 1 yields 90% die-off at ten hours, independent of temperature, except to the extent that k may well be a function of temperature among other things.

The idea of first order decay is also put forth by US EPA (2006), but this might be viewed with some skepticism given multiple pathways for die-off, some of which may be in play to one degree or another at the same time and varying over time, e.g., retention and travel time in a pond may easily span a considerable window of diurnal cycle, exposing the pond to varying light intensity.

US EPA's proposed first-order die-off equation is:

$$Ct = Co * e^{-kt}$$

Where Ct = concentration of organism at time t
Co = concentration of organism at time = 0
k = die-off rate (h^{-1})
t = elapsed time since t = 0

The formula can be rearranged to calculate the expected treatment efficiency:

$$\frac{Ct}{Co} = e^{-kt} \qquad \text{where percent removal efficiency} = \left(1 - \frac{Ct}{Co}\right) * 100$$

Not surprisingly, using the same *k* value, this gives different results than Minton's first-order die-off rate; e.g. with $k = 2$, die-off at 1 hour is predicted to be 87%, compared to Minton's 50%.

US EPA (2006) notes that "First-order models that do not consider background concentrations or re-suspension, may underestimate actual bacterial concentrations". Struck et al., (2008) found this to be the case experimentally in a laboratory setting, finding:

> "Bacteria inactivation generally followed the first-order, KC* model, which includes irreducible or background concentrations of a stressor. Sediment analyses indicate bacteria accumulated in sediments which may maintain background concentrations could be reintroduced into the effluent of these BMPs by turbulent flow causing resuspension or by accumulation through lack of maintenance. *First-order models that do not consider irreducible concentrations may underestimate actual bacterial concentrations*."[*]

To better account for background concentrations, Struck et al. (2008) present more complex formulae from the literature (Kadlec and Knight 1996), (Wong and Geiger 1997). These citations are about treatment wetlands, not wet ponds; however, Struck et al. (*ibid*) apply the findings to wet ponds. In fairness, what is similar between wetlands and ponds, and indeed other stormwater treatment facilities is "the stochastic nature of storm-water-related systems" (Struck et al. 2008) citing (Wong and Geiger 1997); as well as most of the unit processes. In fact, all of the same unit processes are in play when plants are part of a pond system; although the degree to which different pollutant-removal processes are at work will vary – e.g., planting density clearly differs between wetlands and ponds, and with respect to bacteria, this will cause differences in e.g. adsorption to vegetation, sedimentation, sunlight inactivation, and predation.

Struck at al. (2008) present a more complex k_{overall} (from the literature) as a function of K_{tmep}, k_{light}, and k_{others}; and that the *k* values vary according to facility type (i.e., retention pond and treatment wetland), bacteria species, and seasonality.

In summary, Struck et al. (*ibid*) point to Kadlec and Knight's (1996) recommendation:

$$C_{out} = C^* + (C_{in} - C^*) \cdot e^{-KA/Q}$$

Where C^* = background concentration
- K = pollutant rate constant (see note above re: applicability of k_{overall})
- A = pond area
- Q = steady state flow

That all said, Kadlec (2000) did a modeling exercise in which he demonstrated "the inadequacy of first-order treatment wetland models". He notes that "the parameters (rate constants and apparent background concentrations) were found to be very strong functions of hydraulic loading and inlet concentration".

[*] *emphasis added*

Minton (2005) cautions that the "many studies cited are from small laboratory or field units. These results must be viewed with caution". Further, what do we do with disagreement on the role of temperature in die-off, multiple *k* values, and different die-off equations overall? e.g., note a graphical presentation of Struck et al. (2008) experimental data indicating variability in $k_{overall}$ as a function of time of year.

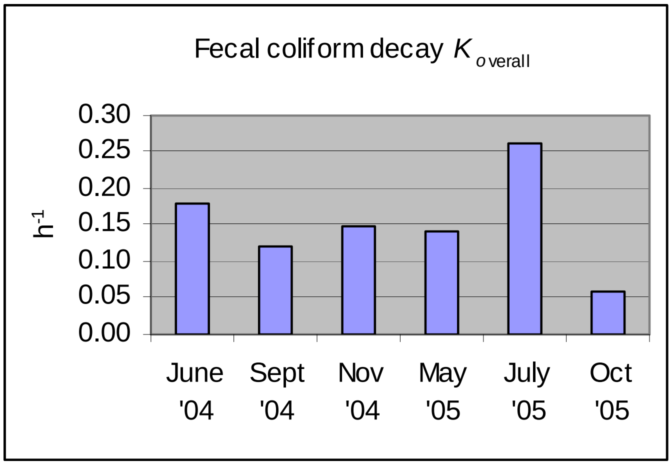

Figure 18. Overall fecal coliform decay rate at different times of the year.
From retention pond data from Struck et al., (2008).

Between these observations, and the fact that the different proposed fundamental equations yield different answers, we cannot simply model percent removal from wet ponds, but must rely on empirical data as integrating not only die-off (inactivation and death by physical stressors and predation), but also aggregation, sedimentation, interaction with vegetation, water column interaction with bacteria sequestration in sediments and re-suspension, potential bacterial growth, and secondary bacterial inputs, e.g. waterfowl on the pond. Struck et al., (2007) note that, "The difference in predatory effects on the dominant species on indicator bacteria concentrations in each system cannot be adequately quantified". US EPA (2006) has a good term for this: "*inactivation rate due to collective environmental factors*". Given multiple factors each with their own inherent variability, and especially considering the known very large variability in bacteria concentrations, we should expect the empirical data to be highly variable with an inherent large amount of uncertainty.

5.5.2 Assessment of bacteria inactivation with % removal as a metric

Kurz (1998) indicates fairly high fecal coliform (FC) load removal efficiencies for wet ponds, 69% to 98%, but the experimental settling time was 5 days and 14 days, compared to the 'typical' Juanita regional wet pond hydraulic travel time of 5.6 hours, and 7 hours for the rain garden ponds. Further, some of Kurz's ponds were designated 'shallow', without any indication of vegetation – which means these would be devoid of shading from solar UV light. Last, Kurz says "On a few occasions, however, concentrations of total coliform* bacteria . . . were greater in outflow samples than inflow samples".

* includes fecal coliform (FC)

5.6 Copper

5.6.1 Solid

On one hand, we could assume that solid copper as part of TSS is removed at the same rate as TSS overall. On the other hand, heavy metals tend to bind to finer TSS particles, or the metals themselves constitute smaller TSS particles. Small diameters suggests slower settling rates; on the other hand, the higher density of heavy metals (~ 5.x) compared to silica TSS (2.65), suggests higher settling rates. The original assumption was net cancellation, leading to the assumption that solid copper would be removed at the same rate as TSS overall, or 80%. However, we found one paper (Hettler et al. 2009) indicating Cu removed at a lesser rate than TSS. Appling their ratio with a margin of safety gives 90% x 80% TSS removal = 72% solid Cu removal. As with TSS in the rain gardens, we are assuming re-suspension at high flows.

Potentially undermining those assumptions, Gharabagi et al. (2007) found net export (-12.3% removal) of total copper from compost socks. As total copper, we don't know what fraction was solid, if any. We also don't know if this was unique to the compost being used; i.e., e.g., it could have contained yard waste that had been exposed to copper-based pesticide.

5.6.2 Dissolved

The International Stormwater BMP Database (2011c)indicates a median value of 33% removal for dissolved copper for wet ponds. WA Ecology's TAPE program has recently found 30% removal with specified statistical significance and power, to be achievable by proprietary enhanced basic treatment facilities; the stated performance goal is > 30% removal. It seems unlikely then that 33% removal is achievable by a non-proprietary wet pond; it seems more likely that a wet pond will achieve < 30% removal. In the absence of further research and analysis, for the purposes of this exercise, we will go with the more conservative 30% removal rate.

The International Stormwater BMP Database (*ibid*) indicates a median value of 48% (50% in summary) removal for total copper; dissolved Cu is not reported for bioretention. In review of the draft Juanita report, Ecology has noted column studies by one researcher suggesting dissolved copper removal rates up to 80% or higher (O'Brien 2012); but this is based on admitted limited unpublished data; and as noted previously in this assessment, laboratory column studies are rarely if ever borne out in field studies. Consequently, for this exercise, it seems more prudent to use the lower limit value from TAPE, i.e., 30%.

5.6.3 Leachate: Removal efficiency unknown

Several researchers have found leachate from various compost mixes to contain copper at levels exceeding surface water quality criteria[*] and/or levels of concern regarding salmonid olfactory effects (Bugbee et al. 1991; Gove et al. 2001; Hinman 2012; Kirchhoff et al. 2003). This is something that should be considered and under ongoing review with respect to rain gardens and bioretention facilities.

[*] Most results are total copper, but levels are high enough that even with a conservative translator to estimate dissolved fraction, dissolved Cu levels would still exceed criteria and/or olfactory effects threshold.

6.0 Percent Removal Rates for This Exercise

For this exercise, for conventional wet ponds built to design standards, we are presuming design-manual 80% TSS removal where Vb/Vr ≥ 3, but this itself is not certain for reasons previously discussed. At Vb/Vr = 0.75, the presumption is 50% TSS removal with the same caveat. We are not giving credit for more removal if/where pond Vb/Vr > 3. We are also presuming 0% removal as Vb/Vr approaches zero, but can only approach zero (Vb/Vr = 0.1) with a continuous curve because a simple curve fit (using MS Excel 2003) is a log function (Figure 19). We are applying the same logic for wet pond removal of total phosphorus, as described in Section 2.2.2 (regional wet pond design and modeling assumptions); applying 50% removal at Vb/Vr = 4.5 gives us 45% removal at Vb/Vr = 3.

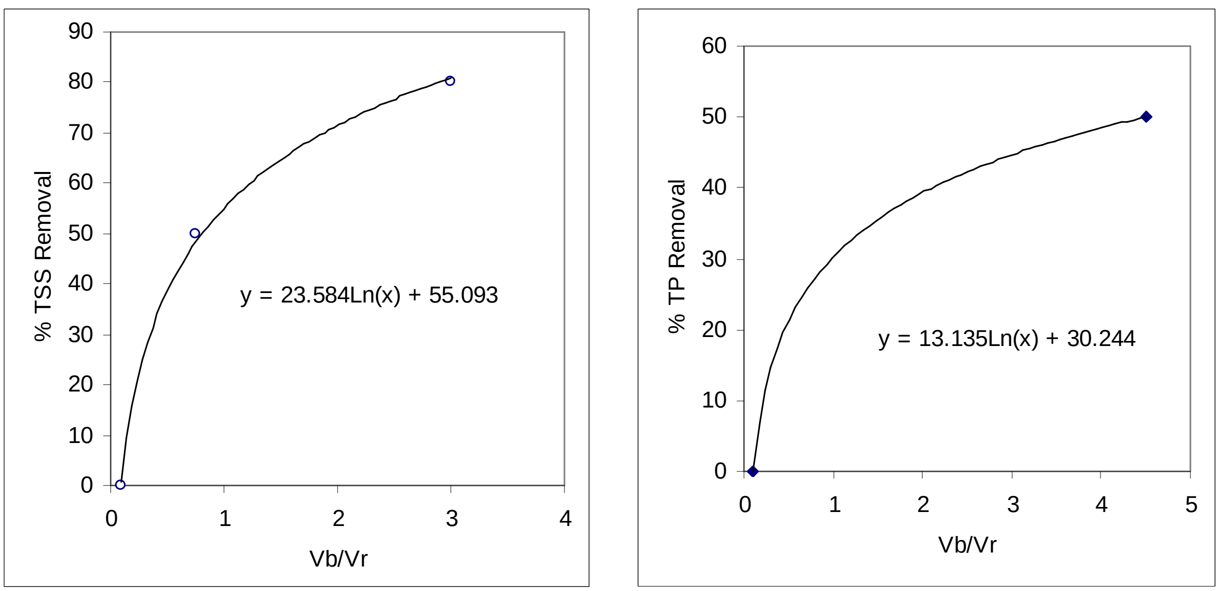

Figure 19. Percent TSS and TP Removal as a function of Vb/Vr
(x = Vb/Vr)

For rain garden ponds we are applying a dynamic settling equation to national and reasonably local TSS PSD data, and the same equation to a "possible" TP PSD, acknowledging uncertainties in the applied data. For other pollutant removal rates, we are relying on published empirical pollutant removal rates with a high degree of uncertainty of the representativeness and overall quality of the data, and a variety of treatment facilities, many not known to be built according to our design standards. We are also apply a conservative override value of 0% removal e.g. where published average or median % removal > 0. We are also using 0 % removal where individual studies or the published range indicates < 0 (net export), or a range that spans both removal and export; e.g. with nitrate and bacteria in wet ponds. We are likewise applying 0 % removal where data are insufficient or inconclusive because site-specific redox conditions affecting speciation; e.g. for ammonia/ammonium.

With the exception of TSS removal for conventional wet ponds – which is based on design manuals, many pollutant removal rate estimates following differ from our original estimates, which were used for the model. The reason for this is that pollutant removal rates information has changed during the course of this study. Of particular note, pollutant-removal rates reported in the International Stormwater BMP Database changed from June 2008 (the data we used initially) to November 2011, as shown in Table 1.

Parameter	Percent Removal		
	Regional Ponds	Rain Garden Ponds	
	Surface	Surface	Infiltrate
Total suspended solids (TSS)	80[*†]	70[‡]	80[*†]
Copper – solid	74[§]	74[§]	74[§]
Copper – dissolved	30[†]	30[†]	30[†**]
Phosphorus – total	45[††]	54[‡,‡‡]	0[†, §§]
Phosphorus – SRP (aka OP)	0	0	0
Nitrate (NO3-)	0	0	0
Ammonia/ammonium (NH3/NH4+)	0	0	0
Total Nitrogen	27[†]	27[†]	21[†]
Fecal coliform bacteria (FC)	0	0	0[***]

Table 6. Summary of pollutant removal rate assumptions to be used for modeling

We have not differentiated rain garden pollutant removal efficiencies between the high and low infiltration rates. In retrospect, and for future modeling efforts, we should consider for at least some pollutants, it is reasonable to expect greater or lesser removal efficiencies at lower and higher infiltration rates respectively. Whether there are enough qualified data to quantify this is highly questionable, but it is an important question as we strive to infiltration more stormwater to solve surface water quality problems.

[*] Presumptive (SWDM & SWMMWW)
[†] ISWBMPDB, 2011c
[‡] Based on dynamic settling equation and PSD fractions and settling rates in Papa et al., 1999, with additional analysis using PSD data from Snohomish County, and settling rates correlated to PSD from MOEE (1994) and Li et al. (2008). See Section 2.2.2 Rain Gardens.
[§] No empirical data; based on thought process from very limited data (see text narrative). Could be as high as 80% (presumptively as great as presumptive TSS rate), but taking a conservative view.
[**] Based on qualitative analysis of SWDM assumptions and very limited empirical data; taking a conservative view in the absence of weight of evidence to the contrary. See section 2.4.10.1.
[††] Logarithmic equation derived from presumptive 50% removal when Vb/Vr = 4.5, and applied to Vb/Vr = 3.
[‡‡] Based on Fletcher et al. (2004) TP and TSS data. Calculated relative TP and TSS rates and applied that fraction to estimated Snohomish TSS removal efficiency.
[§§] Rounded down to be conservative, as net export may occur from compost media (see Howie 2011 reference).
[***] Insufficient data

7.0 References

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APPENDIX E. WATERSHED MODEL CALIBRATION REPORT

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Appendix E

Watershed Model Development and Calibration Report for Juanita Creek Stormwater Retrofit Study

Ecology Grant: G0800618

August 2012

King County

Department of Natural Resources and Parks
Water and Land Resources Division
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Appendix E

Watershed Model Development and Calibration Report for Juanita Creek Stormwater Retrofit Study

Ecology Grant: G0800618

Prepared for:

Stormwater Management Implementation Grant Program
Washington State Department of Ecology
Water Quality Program
Northwest Regional Office
Bellevue, Washington

Prepared by:

King County Science and Stormwater Services Sections
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1.0. INTRODUCTION

A hydrologic watershed model was developed for the Juanita Creek drainage basin to evaluate various future stormwater management and mitigation scenarios. Juanita Creek drainage basin is predominately located within the City of Kirkland and drains to Juanita Bay on the east shore of Lake Washington. The model simulations will provide guidance on techniques and their relative effectiveness towards meeting the long-term objective of fully restoring beneficial uses to Juanita Creek.

The Juanita Creek hydrology model is based on Hydrologic Simulation Program- FORTRAN (HSPF). The model simulates water quantity and water quality at multiple catchments throughout the drainage. The model output is used to calculate flow, water quality, geomorphology, and biology metrics to evaluate the projected improvements under different mitigation scenarios. This appendix documents the development and calibration of the watershed model. This is presented to provide context to the level of applicability and relevancy when evaluating simulation results (e.g. model accuracy, inferring conclusions, etc.) and informing policy makers.

2.0. MODEL DEVELOPMENT

Development of the model included assembly of several types of data including:

- land use,
- land cover,
- surficial geology,
- channel/wetland/lake hydraulics,
- measured atmospheric conditions and stream flows, and
- sampled water quality.

The water quantity mechanics are modeled using three flow pathways representative of surface runoff, shallow subsurface (interflow), and shallow active ground water resurfacing in the stream within the same catchment. Water quality elements are modeled using “build up/wash off” for constituents defined to accumulate (build up) at constant rates creating the potential of wash off, as well as user defined seasonally variable interflow and groundwater concentrations. The rate of wash off varies depending on antecedent conditions leading up to a storm, as well as the rainfall intensity and duration. Further detail on the individual parameters is given in subsequent sections of this appendix.

2.1 Geographic Information System Layers

- Several data layers were used to develop the watershed model land use and land cover was identified based on a coverage derived from King County 2002 imagery.
- Topography was identified based on a digital elevation map derived from King County LiDAR (Light Detection And Ranging) imagery
- Soil characteristics were identified and generalized based on the King County surficial geology map.
- Catchment delineations were developed using topographic conditions generated from the afore mentioned LiDAR

2.2 Channel Hydraulics

Stream channel width, depth, slope, roughness, and shape were estimated using various methods depending on the data available and the types of features controlling the behavior of the stream system. Relationships between water depth, surface area, channel volumes, and flow rates were defined based on the most hydraulically controlling feature in a stream reach per catchment. These control points were either a result of culverts, channel roughness, or combinations of both. The transient surface area and storage volumes were determined based on the length of the channel through the catchment and an assumed typical cross-section. Regional stormwater ponds and large storage area (e.g. lakes or wetlands) volumes were based on either topographic contours using King County LiDAR or bathometric data obtained from City of Kirkland.

The distributed storage volumes of the existing storm sewer pipes/vaults/ponds in the basin were integrated into the assumed stream reach representing the channel for a given

catchment. While these volumes can be relatively small compared to the stream reach, the added storage could have an effect on frequent small storms, which are more sensitive to some of the metrics used in analysis. Where HEC-RAS modeling was performed, those outputs were used to define the primary stream channel stage-area-storage-discharge relationships.

2.3 Survey Storm Sewer Network

A detailed survey of the storm drainage network including all pipes, underground vaults, ponds, and open channels was completed (Figure 1). The information was used to refine catchment delineations originally derived from landscape topography to more accurately identify interceptions and redirection of surface runoff. Further detail on these data can be obtained from City of Kirkland, Washington.

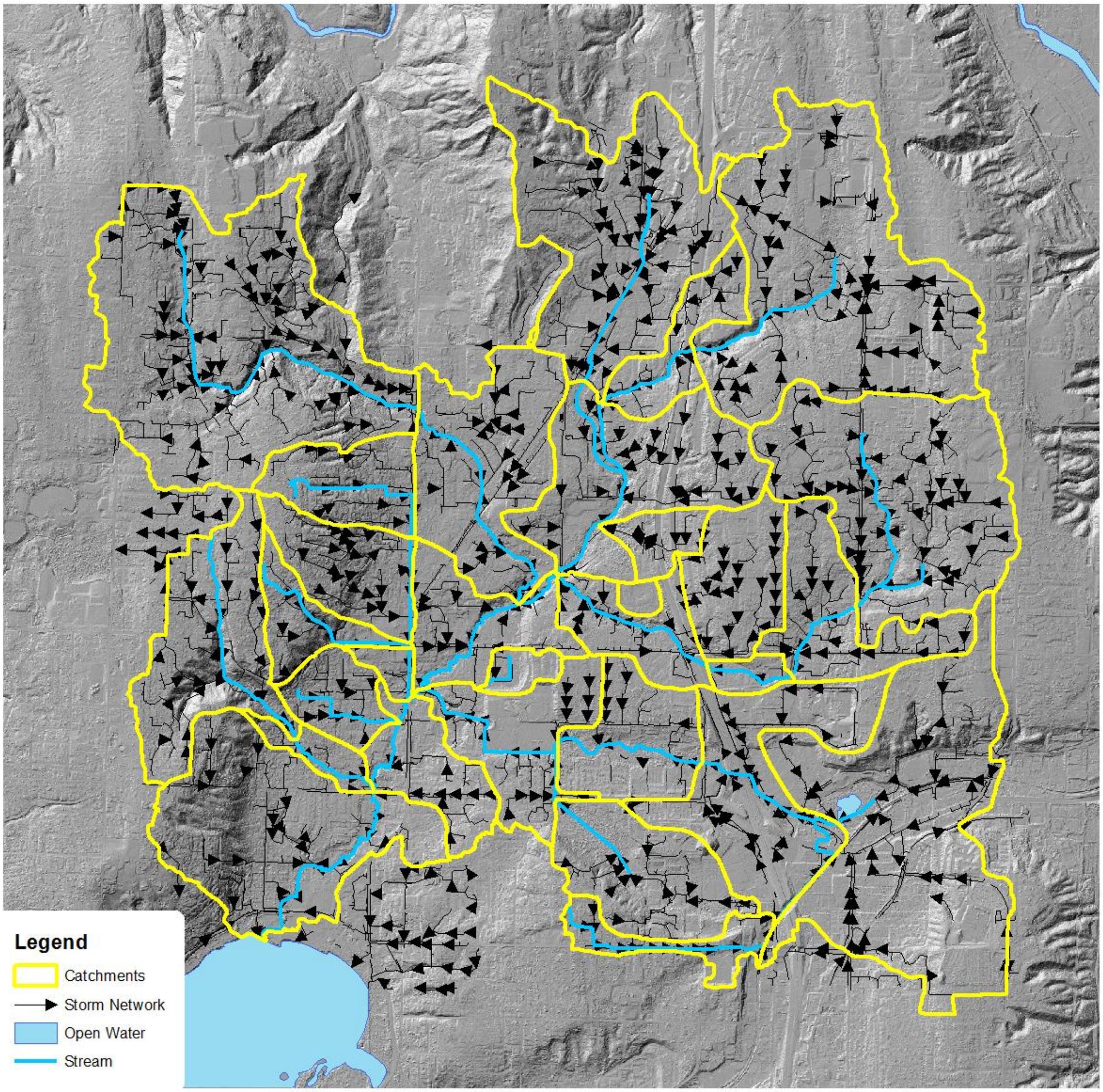

Figure 1 Map of storm drain network with arrows depicting direction of flow. Yellow lines are derived catchment delineations using topography and stormwater network.

2.4 Model Catchment Segmentation

The 6.8-square mile Juanita Creek drainage was segmented into 30 catchments with similar meteorological conditions, topographical features, land use practices, and/or are a region of interest (e.g., non-point source loads need to be quantified). Once the catchments and channel segments have been defined, these catchments must then be further refined to: 1) develop the model categories to represent; 2) define the physical parameters (e.g., elevation, slopes, channel length) for HSPF using available data; and 3) establish initial calibration parameters for HSPF based on past applications within the region and past

experience with the model. Figure 2 presents a map of the catchments and Figure 3 illustrating the linkages of the catchments and channel reaches. Given the densely piped storm network and fast travel times (measured in minutes), catchments were kept relatively small in size. The median catchment area is 99 acres, with catchments ranging from 9 to 484 acres based on landscape conditions covering 4,343 acres (Table 1).

Table 1. Individual catchment areas in acres.

Catchment	Acres	Catchment	Acres
WA3001	203	WA3016	27
WA3002	20	WA3017	484
WA3003	134	WA3018	199
WA3004	202	WA3019	307
WA3005	77	WA3020	417
WA3006	101	WA3021	46
WA3007	119	WA3022	409
WA3008	81	WA3023	178
WA3009	318	WA3024	264
WA3010	93	WA3025	97
WA3011	54	WA3026	122
WA3012	84	WA3027	34
WA3013	69	WA3028	18
WA3014	53	WA3029	9
WA3015	16	WA3030	108

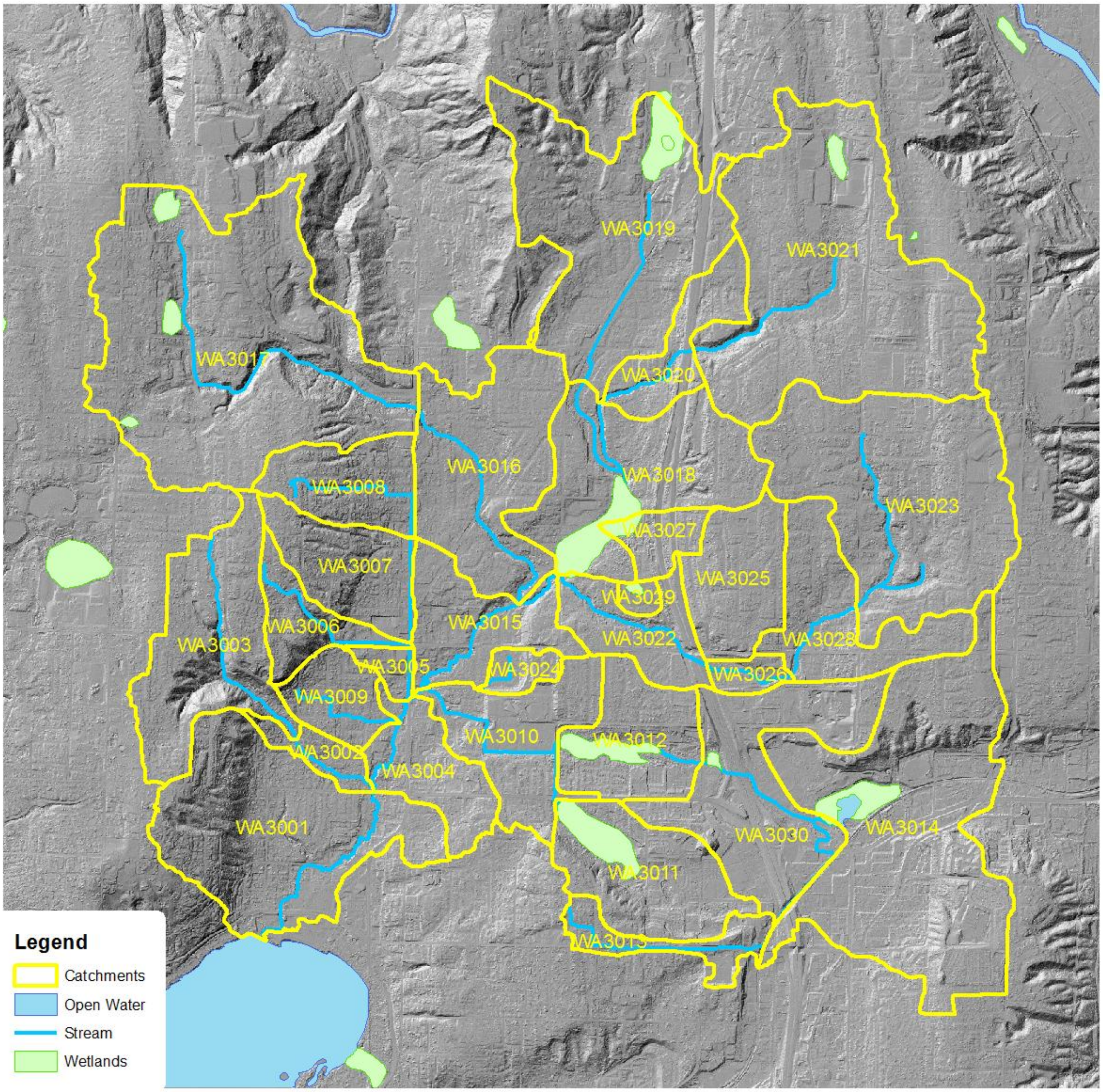

Figure 2 Catchment map illustrating model segmentation. Light green are significant wetland areas, the blue lines and areas are modeled stream reaches and lakes.

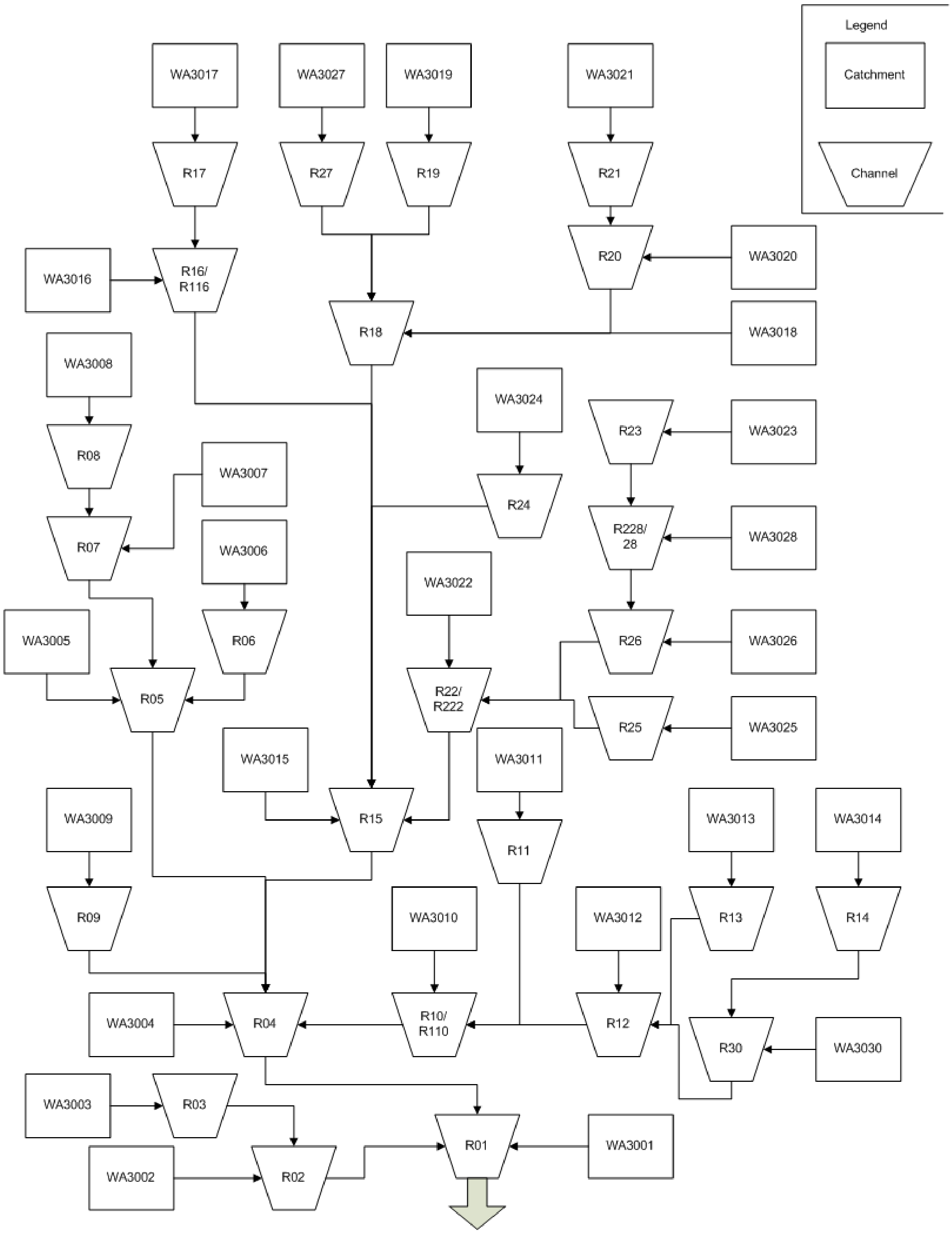

Figure 3 Model schematic

2.5 Monitoring Locations

Detailed stream flow and water quality monitoring was conducted between October 2008 and March 2010. Stream flow gauges were installed in the five major tributaries (stations 27i, 27C, 27DN, 27H, and 27J) and in the mainstem near the mouth (station 27A). Water quality samples were either collected at or near the same location (stations 2G, 3G, 5G, 6G, 7G, and 1G). Additional water quality samples were taken near the inlet (station 4GI) and outlet (station 4GO) of a large wetland downstream of Totem Lake where the majority of commercial development in the basin area exists (Figure 4). Further detail on the monitoring conducted as part of this study can be found in a monitoring report located in appendix A.

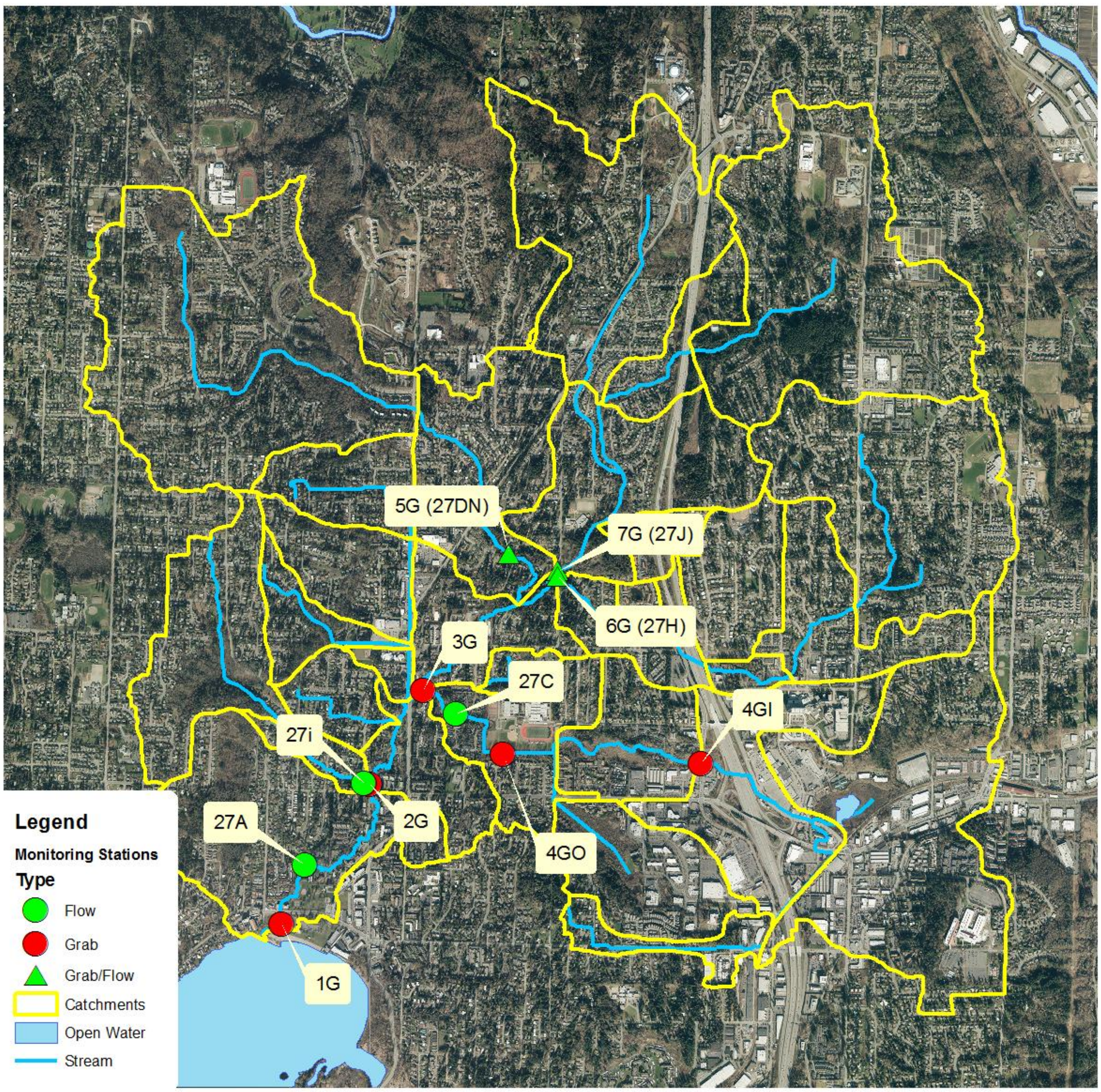

Figure 4 Map of stream flow and water quality measurements.

2.6 Local Weather Data

Precipitation, air temperature, dew point, wind speed and cloud cover were derived from National Weather Service Sea-Tac monitoring station. Solar radiation was obtained from NOAA Integrated Surface Irradiance Study (ISIS) network Sand Point station. Lastly, computed evapotranspiration was obtained from Washington State University Agricultural Weather Network, Puyallup station.

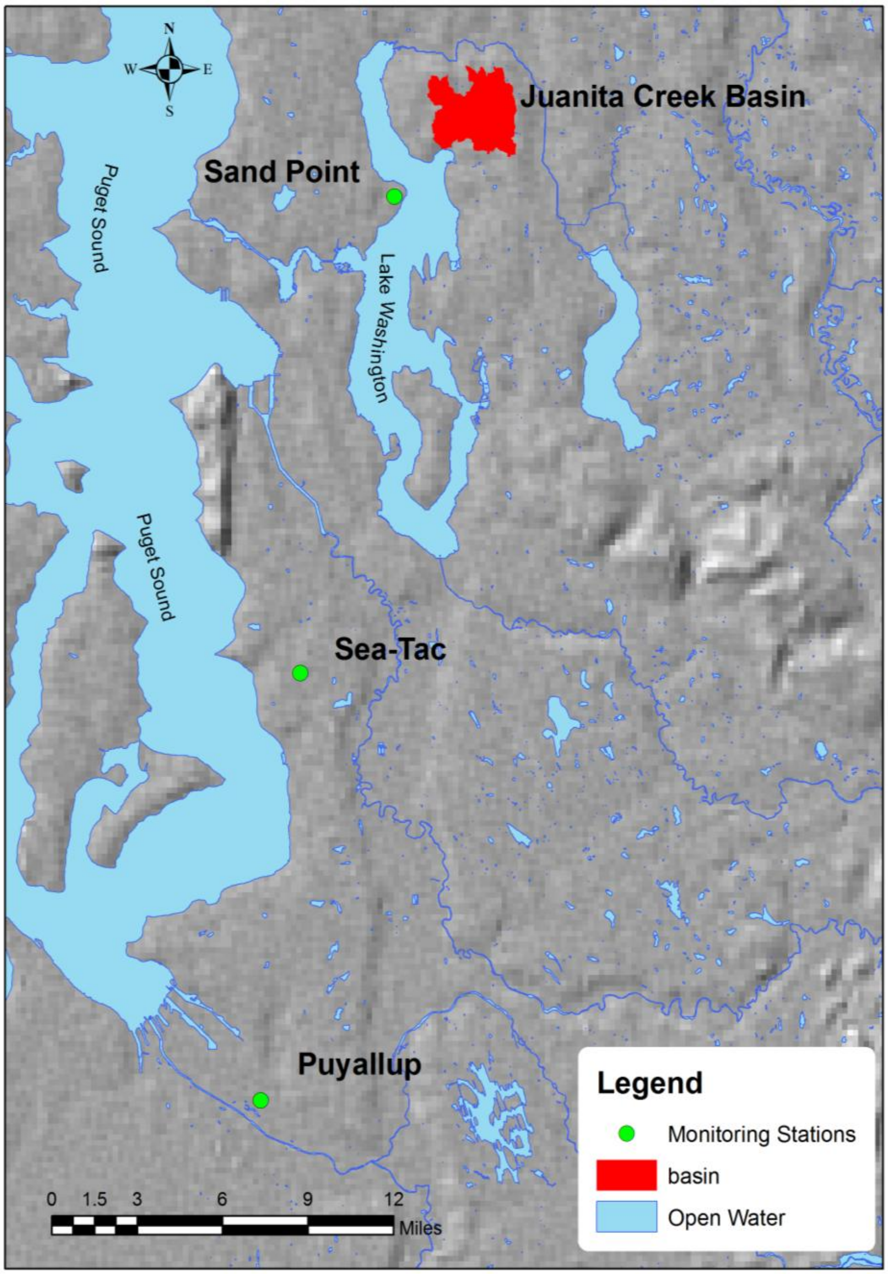

Figure 5 Map of weather monitoring stations used.

2.7 Hydraulic Response Units (HRUs)

For modeling purposes, a distinction is made between total and effective impervious area. Total impervious area includes all surfaces that do not infiltrate runoff. Roofs, paved streets, sidewalks, driveways, and parking lots are all part of the total impervious area. Effective impervious area (EIA) is defined as the area where there is no opportunity for surface runoff from an impervious site to infiltrate into the soil before it reaches a conveyance system (pipe, ditch, stream, etc.). Because it is extremely expensive and time consuming to look at every impervious surface in a watershed to determine whether or not it is an effective impervious area, average EIA values are used instead. Each average EIA value is based on the land use (forest, low density residential, high density residential, commercial, etc.) and previous experience in other Puget Sound lowland watersheds. The following percentages in Table 2 are representative values used in development of the Juanita Creek model, and are similar to other calibrated model schemes in King County.

Table 2 Conversion of land use to HRUs for pervious and impervious land segments (PERLNDs and IMPLNDs).

Land cover / GIS	Forest	Pasture / Grass	Grass-Lt Urban	Grass-Dens. Urban	EIA-Lt. Urban	EIA-Dens. Urban	EIA-Roads	Wetlands
Mixed Forest	100%							
Forest	100%							
Scrub/Shrub	50%	50%			0%			
Dry Ground		50%	50%		0%			
Light Urban (< 75%)		5%		70%	25%			
Dense Urban (> 75%)				25%		75%		
Bare Ground				70%	30%			
Wetlands								100%
Open Water[1]								
Roads				15%			85%	

Integrating the slope and surficial geology into the above listed land covers (Table 2), generates a list of 37 unique hydrologic response units (HRUs) used in the model (Table 3). Not all HRUs may be present depending on presence of a given element during the integration process.

Table 3 Hydrologic Response Units.

HRU #	Soil	Land Cover	Slope
11	Till	Forest	Flat
12			Mild
13			Moderate
14			Steep
21		Pasture	Flat
22			Mild
23			Moderate
24			Steep
31		Residential Forest	Flat
32			Mild
33			Moderate
34			Steep
41		Light Density Grass	Flat
42			Mild
43			Moderate
44			Steep
61		High Density Grass	Flat
62			Mild
63			Moderate
64			Steep
71	Outwash	Forest	n/a
72		Pasture	
73		Residential Forest	
74		Light Density Grass	
75		Moderate Density Grass	
76		High Density Grass	
81	Saturated	Forest	n/a
82		Pasture	
83		Residential Forest	
84		Light Density Grass	
85		Moderate Density Grass	
86		High Density Grass	
87		Wetlands	
91	Impervious Surfaces	Low Density Residential	n/a
92		High Density Residential	
93		Commercial/Industrial	
94		Roads	

3.0. WATER QUANTITY CALIBRATION

An iterative procedure of parameter evaluation and refinement was used to determine parameter values to use in the watershed model. Calibration was based on fifteen years (1995-2010) of simulation for the mainstem and slightly more than one year for the tributaries to evaluate parameters under a variety of climatic, soil moisture, and water quality conditions.

Calibration includes the comparison of both monthly and annual values, and individual storm events, whenever sufficient data are available for these comparisons. In addition, when a continuous or large number of observed record are available, simulated and observed values are analyzed on a frequency basis and their resulting empirical cumulative distributions (e.g., duration curves) compared to assess the model behavior and agreement over the full range of observations.

A weight of evidence approach is most widely used and accepted when models are examined and judged for acceptance as no single procedure or statistic is widely accepted as measuring, nor capable of establishing, acceptable model performance. Therefore, the calibration relied on numerous statistical tests (e.g., correlation tests, model fitness, test of distributions) and graphical plots (e.g., scatter, time series, frequency) to determine the model's ability to represent the system. Parameters that are used to evaluate exceedances of specific thresholds are compared using more rigorous statistical measures (correlation tests and model fitness), whereas parameters used for annual loading rates are tested for similar distributions using Mann-Whitney U-Test.

The following four characteristics of the watershed hydrology were evaluated (in the order shown): (1) annual water balance, (2) seasonal and monthly flow volumes, (3) baseflow, and (4) storm events. Simulated and observed values for reach characteristic are examined and critical parameters are adjusted to attain acceptable levels of agreement (discussed further below).

The critical parameters that govern the annual water balance are as follows:

LZSN - lower zone soil moisture storage (inches).

LZETP - vegetation evapotranspiration index (dimensionless).

INFILT- infiltration index for division of surface and subsurface flow (inches/hour).

UZSN - upper zone soil moisture storage (inches).

DEEPFR- fraction of groundwater inflow to deep recharge (dimensionless).

Changes in LZSN and LZETP affect evapotranspiration by making more or less moisture available to evaporate or transpire. Both LZSN and INFILT also have a major impact on percolation and are important in obtaining an annual water balance. In addition, on extremely small watersheds (less than 200 to 500 acres) that contribute runoff only during and immediately following storm events, the UZSN parameter can also affect annual runoff

volumes because of its impact an individual storm events (described below). While there was no assumed loss of groundwater via DEEPFR (i.e. DEEPFR = 0), there was some intra-basin transfers of groundwater among the catchments connected to contiguous outwash soils but with topographic divides to achieve mass balance.

The portion of stream baseflow is adjusted in conjunction with the seasonal/monthly flow calibration (previous step) because moving runoff volume between seasons often means transferring the surface runoff from storm events in wet seasons to low-flow periods during dry seasons. By adjusting INFILT, runoff can be shifted to either increase or decrease groundwater or baseflow conditions. The shape of the groundwater recession; i.e., the change in baseflow discharge is controlled by the following parameters:

AGWRC- groundwater recession rate (1 / day).

KVARY- index for nonlinear groundwater recession.

AGWRC is calculated as the rate of baseflow (i.e. groundwater discharge to the stream) on one day divided by the baseflow on the previous day; thus AGWRC is the parameter that controls the rate of outflow from the groundwater storage. These values are adjusted as needed through calibration. The KVARY index allows users to impose a nonlinear recession so that the slope can be adjusted as a function of the groundwater gradient. KVARY ranges were based on soil types to account for changes in recession rates between wet and dry seasons. Parameters associated with impervious surfaces (HRUs 91through 94) are not differentiated for water quantity and are characterized with a different set of parameters based on flow length (150 ft), slope (0.01 ft/ft), surface roughness (n= 0.15) and surface storage (0.10 inches). The list of parameter values are summarized in Table 4 below.

Table 4 List of calibrated HSPF parameters

HRU #	LZSN	INFILT	LSUR	SLSUR	KVARY	AGWRC
11	4.6	0.493	350	0.028	0.405	0.997
12	3.8	0.462	300	0.072	0.405	0.997
13	3.2	0.431	250	0.116	0.405	0.997
14	2.6	0.4	200	0.195	0.405	0.997
21	3.2	0.308	350	0.026	0.405	0.996
22	3.2	0.277	300	0.07	0.405	0.996
23	3.6	0.224	250	0.116	0.45	0.997
24	3.6	0.196	200	0.186	0.45	0.997
31	4.6	0.493	350	0.028	0.405	0.997
32	3.8	0.431	300	0.072	0.405	0.997
33	4.3	0.336	250	0.116	0.45	0.998
34	4.3	0.28	200	0.195	0.45	0.998
41	3.2	0.308	350	0.028	0.405	0.995
42	3.2	0.277	300	0.07	0.405	0.995
43	3.6	0.224	250	0.117	0.45	0.996
44	3.6	0.196	200	0.18	0.45	0.996
61	2.9	0.224	350	0.03	0.45	0.996

HRU #	LZSN	INFILT	LSUR	SLSUR	KVARY	AGWRC
62	2.9	0.196	300	0.071	0.45	0.996
63	2.9	0.168	250	0.114	0.45	0.996
64	2.9	0.14	200	0.172	0.45	0.996
71	5.8	3.36	300	0.089	0.27	0.995
72	5.8	3.36	300	0.06	0.27	0.995
73	5.8	0.678	300	0.089	0.27	0.995
74	5.8	0.678	300	0.077	0.27	0.995
75	7.2	0.462	300	0.067	0.3	0.996
76	7.2	0.323	300	0.067	0.3	0.996
81	2.3	4.4	150	0.048	0.45	0.997
82	2.6	3	150	0.043	0.5	0.998
83	2.6	4.4	150	0.048	0.5	0.998
84	2.3	3	150	0.043	0.45	0.997
85	2.6	2.4	150	0.046	0.5	0.998
86	2.6	2.4	150	0.075	0.5	0.998
87	2.6	3	150	0.043	0.5	0.998

Table 5 represents model accuracy based on continuous hourly time steps for the available period of record per calibration point. Values closer to unity for r-square (i.e. explanation of variance) and the Nash-Sutcliffe (i.e. model accuracy taking into account "signal to noise" ratio) denote a more accurate model for estimating flow rates.

Table 5 Summary statistics for calibration of hourly time increment flow rates.

Statistic	Mainstem (27a)	Billy Creek (27i)	Totem Lake Trib. (27c)	West Branch (27dn)	East Branch (27h)	North Branch (27j)
Pearson Coefficient	0.86	0.76	0.88	0.8	0.91	0.91
Mean Error (cfs)	-0.2	0.06	0.31	0.08	0.1	0.23
RMSE (cfs)	8.1	0.45	1.99	1.35	0.74	1.49
r-square	0.74	0.58	0.78	0.64	0.83	0.82
Mean Absolute Error (cfs)	4.19	0.2	1.05	0.64	0.34	0.69
Nash-Sutcliffe	0.69	0.44	0.61	0.52	0.81	0.75
Skill Score[1]	0.45	0.25	0.37	0.31	0.56	0.5

[1]Skill Score = 1 – RMSE/Std. Dev.

Table 6 summarizes how well the model simulates a broader spectrum of flow rates grouped into five thresholds for analysis. One subbasin (Totem Lake tributary) is poorly calibrated to annual low flow conditions such that simulated flow rates are about twice observed with a relative percent difference of 187% (e.g. simulated might be 2 cfs while observed may be 1 cfs).

Table 6 Summary of model accuracy for flow rates using quantiles characterizing various magnitudes of flow rates.

Statistic	Mainstem (27a)	Billy Creek (27i)	Totem Lake Trib. (27c)	West Branch (27dn)	East Branch (27h)	North Branch (27j)
Mean	-2%	18%	13%	6%	8%	10%
90-Percentile	5%	22%	11%	27%	-2%	26%
75-Percentile	-2%	12%	-15%	-8%	3%	10%
50-Percentile	-19%	26%	-2%	-14%	20%	-5%
25-Percentile	-23%	-3%	38%	-35%	32%	-8%
10-Percentile	-13%	2%	187%	-13%	25%	-26%

The differences between simulated and actual annual 7-day minimum flow rates, instantaneous maximum flows were assessed (Table 7). This analysis shows the model characterizes observed conditions quite well except for low flow conditions as previously mentioned for Totem Lake (27c) and now East Branch (27h). However, these types of statistics based on annual events is tenuous at best, given the short period of available data for all calibration points except near the mouth (27a).

Table 7 Summary of annual 7-day minimums and instantaneous maximums.

Metric	Statistic	Mainstem (27a)	Billy Creek (27i)	Totem Lake Trib. (27c)	West Branch (27dn)	East Branch (27h)	North Branch (27j)
Annual 7-Day Low Flow	Difference (cfs)	-0.06	0.05	0.79	0.06	0.31	0.23
	RPD	-1%	26%	90%	9%	55%	17%
Instantaneous Daily Maximums	Difference (cfs)	1.9	0.02	0.8	0.03	0.29	0.68
	RPD	11%	3%	24%	1%	12%	16%
Instantaneous Annual Maximums	Difference (cfs)	1.99	-0.51	1.4	-3.76	1.82	-7.39
	RPD	1%	-8%	4%	-19%	11%	-18%

In addition to the typical methods of testing model accuracy, an assessment was also performed on the hydrologic flashiness metrics used in this study (Table 8). However, these metrics depend on annual summaries and only the mainstem (27a) had sufficient amount of data for evaluation. Using the Mann-Whitney *U*-test, two of the metrics individually fail the alternative hypothesis of equivalence with p-values less than 0.05 (HPC and RBI); however, combining all metrics together does pass the test (*p*-value ≈ 0.70).

Table 8 Summary of flashiness metrics used in this study and resultant *p*-value using Mann-Whitney *U*-test comparing observed and simulated flow rates for mainstem Juanita Creek (station 27a).

Metric	Name	Description	*p value*
LPC	Low Pulse Count	Number of times each calendar year that discrete low flow pulses occurred	0.08
LPD	Low Pulse Duration	Annual average duration of low flow pulses during a calendar year	0.70
HPC	High Pulse Count	Number of days each water year that discrete high flow pulses occur	0.03[2]
HPD	High Pulse Duration	Annual average duration of high flow pulses during a water year	0.39
HPR	High Pulse Range	Range in days between the start of the first high flow pulse and the end of the last high flow pulse during a water year	0.12
QR	Flow Reversals	The number of times that the flow rate changed from an increase to a decrease or vice versa during a water year. Flow changes of less than 2% are not considered	0.09
TQmean	TQ_{mean}	The fraction of time during a water year that the daily average flow rate is greater than the annual average flow rate of that year	0.54
RBI	R-B Index	Richards-Baker Index – A dimensionless index of flow oscillations relative to total flow, based on daily average discharge measured during a water year	0.00[2]
P2YR[1]	Peak 2-yr:Winter Baseflow	Ratio of the estimated 2-year peak flow to winter baseflow (i.e., mean flow for October through April)	n/a

[1]Relationship between metric and BIBI still in development as part of EPA WRIA 9 grant.
[2]Simulated metric is statistically different from observed, simulation fails test.

To understand model accuracy beyond the statistics previously provided, scatter plots are included to visually inspect for added value and illustrate possible aspects not encapsulated statistically. For reference, a 1:1 line is drawn illustrating a perfect fit of simulated to observed. Mean daily flows are illustrated in Figure 6, while daily maximum flows are shown in Figure 7. Additionally, visualizing model predictions of seasonality vis-à-vis mean monthly flow rates (Figure 8) highlights any volumetric bias that otherwise may cancel out looking at annual mass balances. For those series of graphs, each month is color coded to more easily see a consistent bias for any given month.

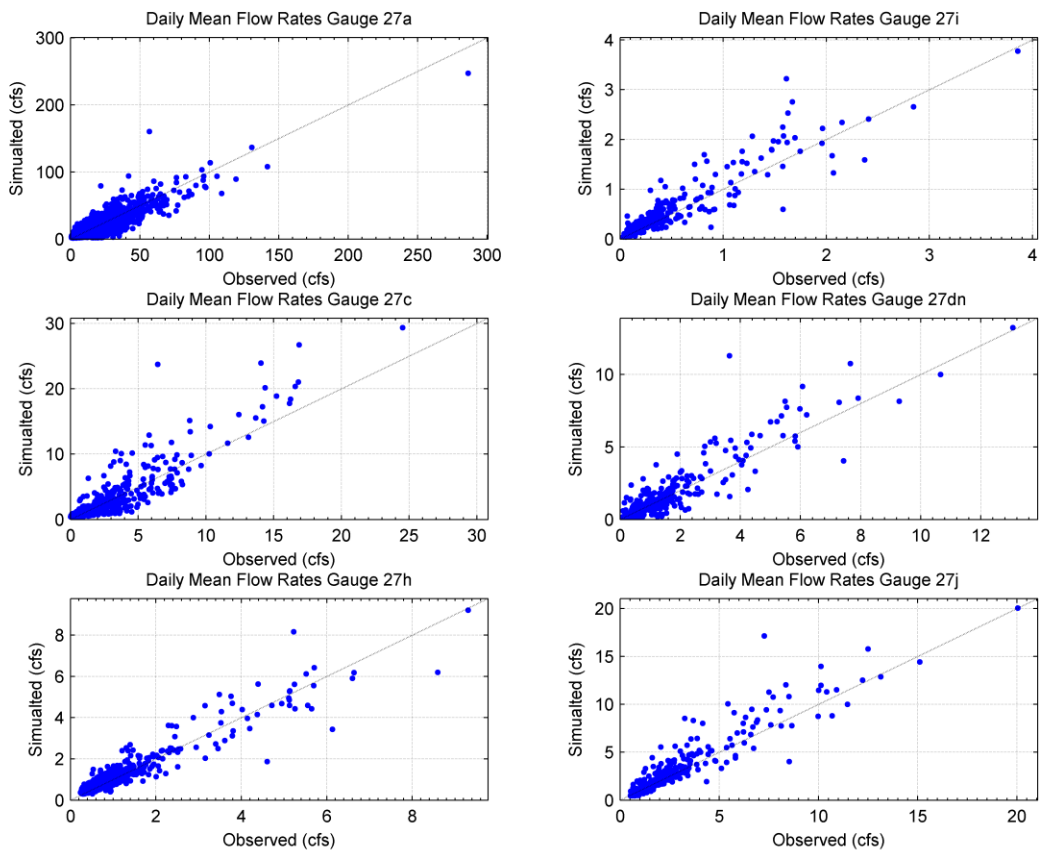

Figure 6 Scatter plots comparing observed to simulated mean daily flow rates

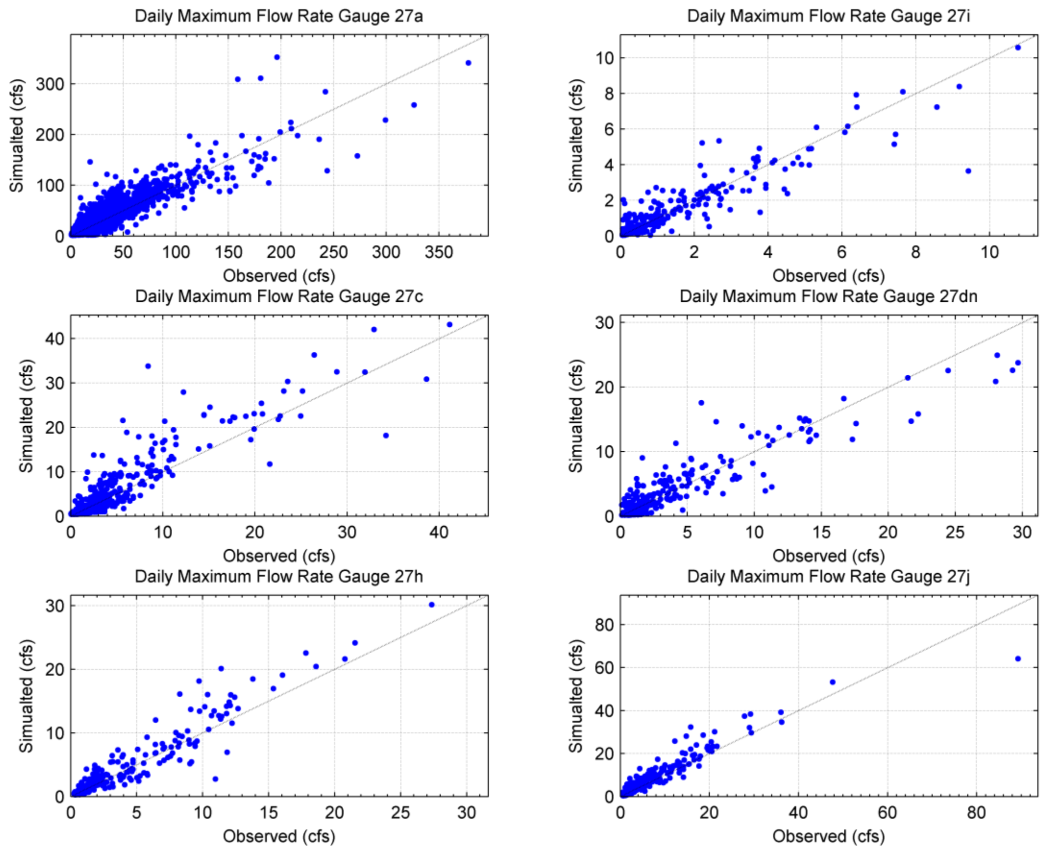

Figure 7 Scatter plots comparing observed to simulated daily maximum flow rates

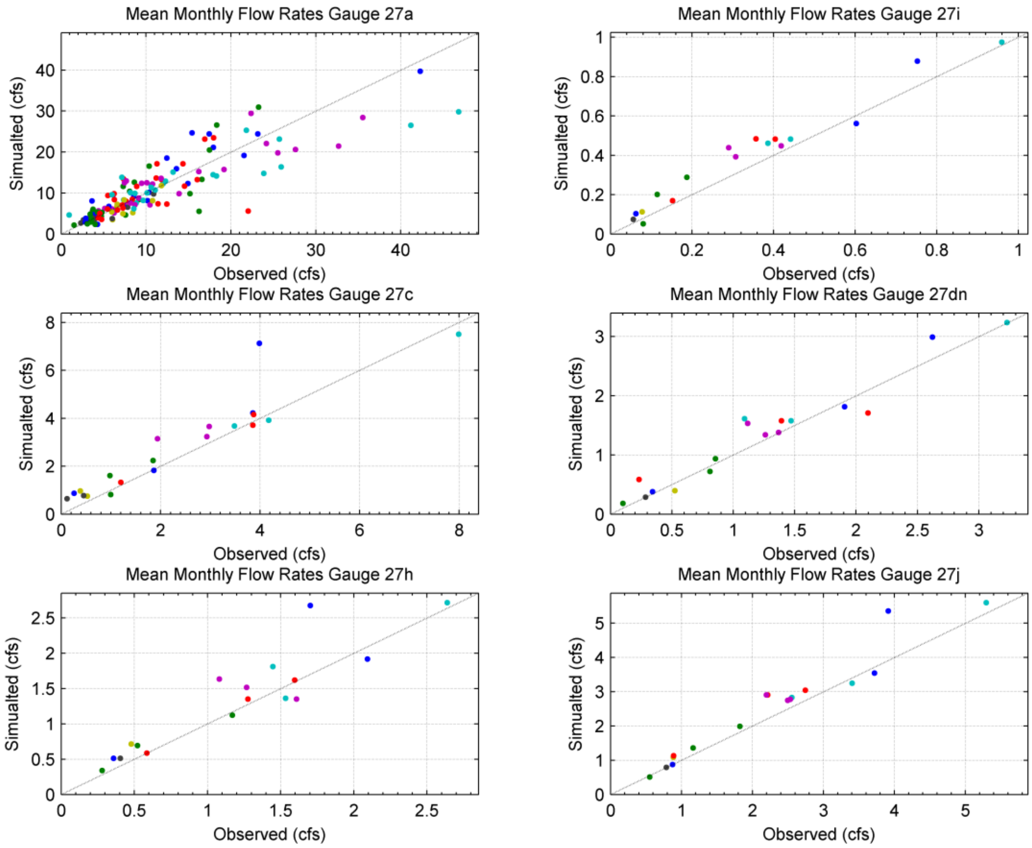

Figure 8 Scatter plots comparing observed to simulated average monthly flow rates. Each of the 12 months is a different color dot.

4.0. BIOTIC CALIBRATION

Biotic calibration for this study is based on relationships between hydrology and Benthic Index of Biotic Integrity (BIBI), but includes no parameter adjustments to the watershed model. These relationships are derived from a previous study identifying significant correlations between hydrologic flashiness metrics (previously defined in Table 8) and BIBI data collected at 16 different locations within King County (DeGasperi, et al., 2009). Model accuracy is dependent on the correlation between predicted outcomes from the regressions and observed. For this study, defined regressions are characterized into three types: log-linear (Equation 1), linear (Equation 2), and exponential (Equation 3). Summarized in Table 9 are the coefficients of the regressions and its corresponding explanatory power as measured using R^2. Scatter plots characterizing the data as well as the regression predictions are illustrated in Figure 9 and Figure 10.

Table 9 Regression coefficients (a,b) for predicting BIBI (y) from hydrologic flashiness metrics.

Metric (x)	Equation Used	a	b	R^2
LPC	1	45.331	-22.466	0.44
LPD	1	-5.1273	23.214	0.59
HPC	1	53.05	-30.106	0.71
HPD	1	8.9753	23.498	0.64
HPR	2	44.167	-0.1148	0.73
QR	2	66.994	-0.7664	0.42
TQ_{mean}	2	-21.493	147.3	0.47
RBI	2	38.616	-51.851	0.49
P2YR	3	57.277	-0.311	0.22

Equation 1 Regression used for LPC, LPD, HPC, and HPD.

$$y = a + b\log_{10} x$$

Equation 2 Regression used for HPR, QR, TQmean, and RBI

$$y = a + bx$$

Equation 3 Regression used for P2YR.

$$y = ax^b$$

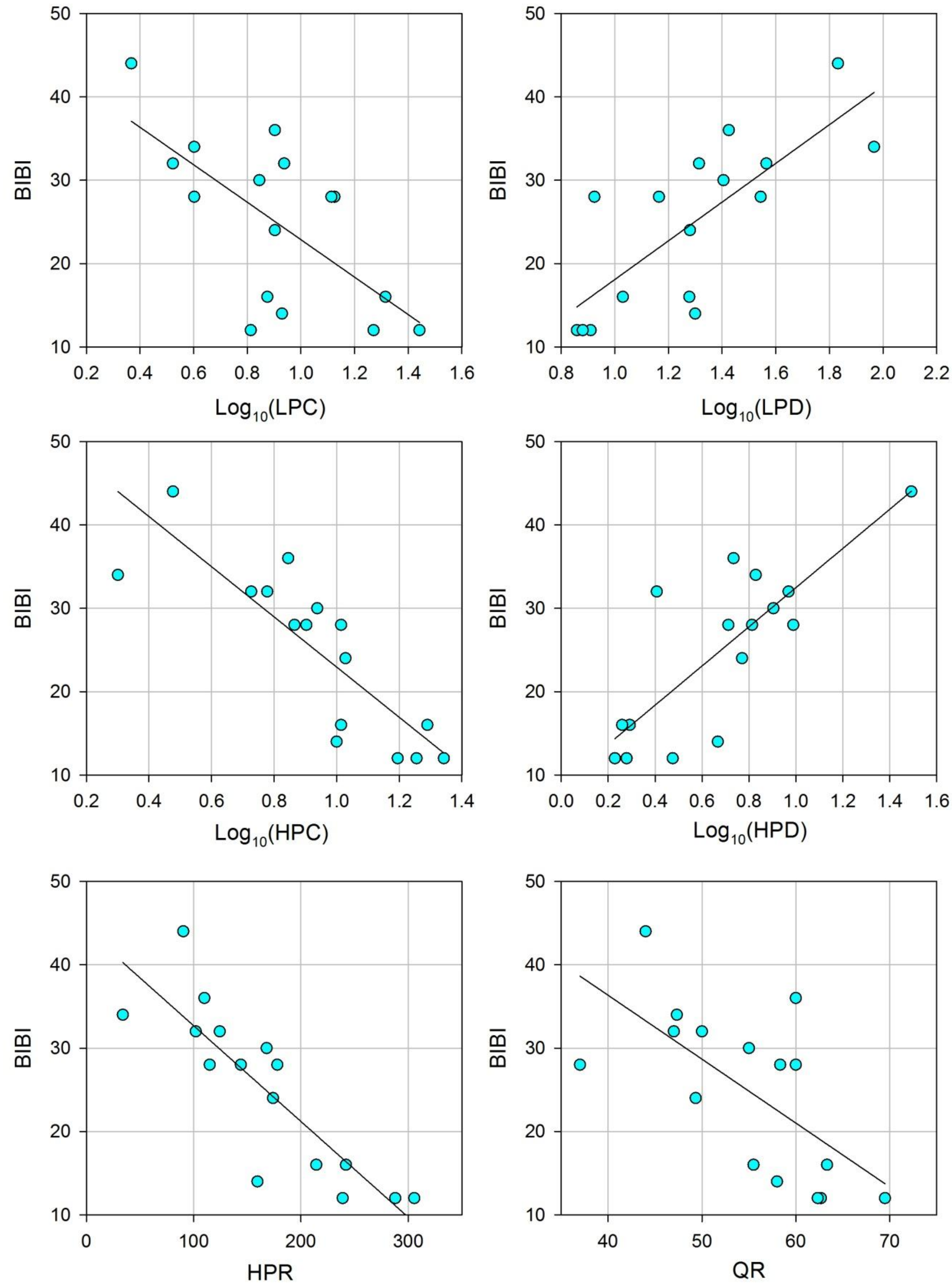

Figure 9 Scatter plots with regressions lines for LPC, LPD, HPC, HPD, HPR, and QR.

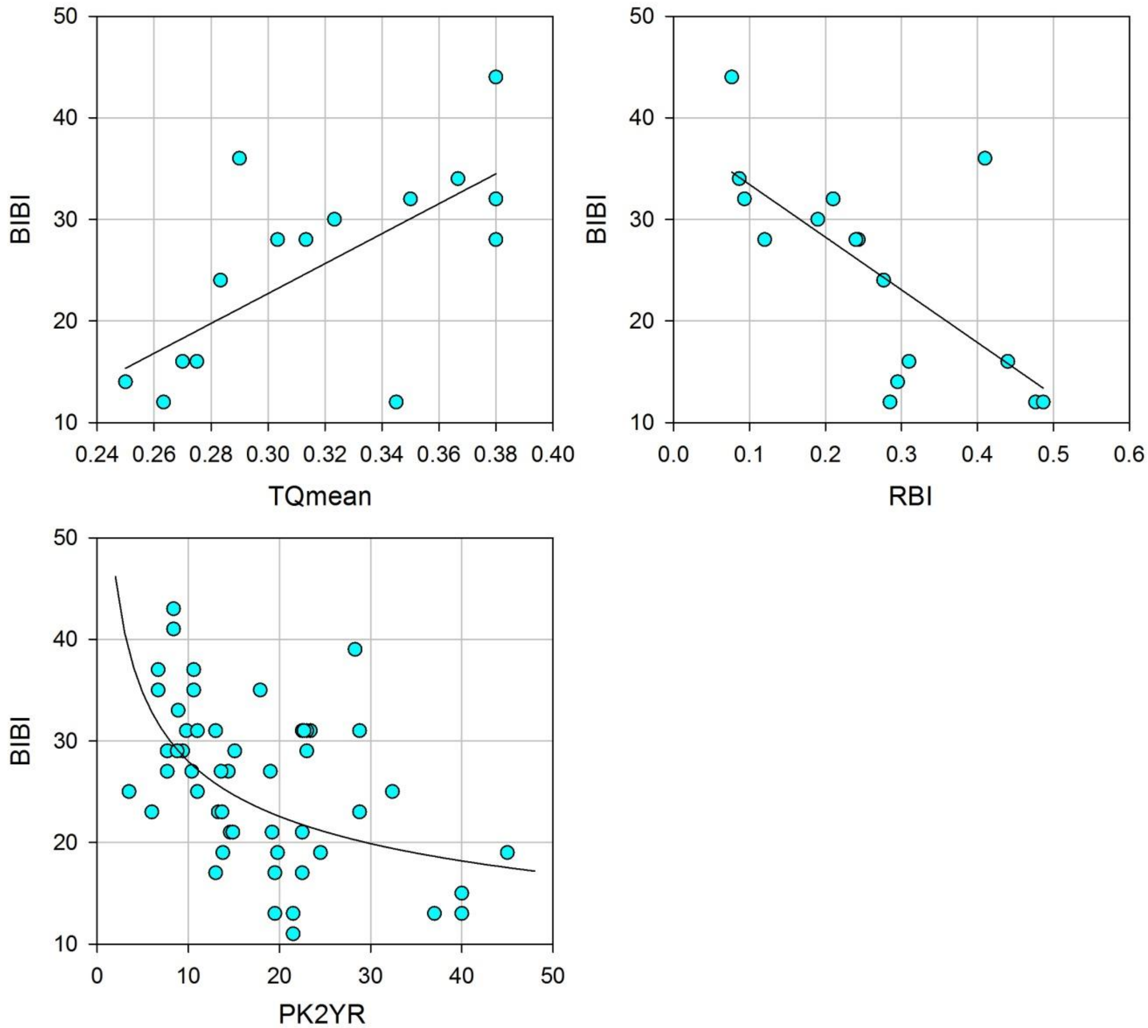

Figure 10 Scatter plots with regression lines for TQmean, RBI, and PK2YR

Simulated scores representing existing conditions (i.e. 2002 land use), on average, were three points higher (BIBI = 17) than averaged observed (BIBI = 14.4) over a similar time period (Table 10) at four locations within the Juanita Creek study area. This level of accuracy, with 18% relative error, is far greater than the identified accuracy among the defined relationships as previously shown in Figure 9 and Figure 10. Thus this application is judged adequate for use in this study (in conjunction with prior stated caveats in previous sections and in the main report this appendix is supporting).

Table 10 Simulated versus observed BIBI scores.

Site Code	Date Range	Observed	Simulated	RPD
		Mean	Mean	$\frac{(S-O)}{O} * 100$
JuanitaKirk1	2002 - 2008	14	16	14%
JuanitaKirk2	2005 - 2008	14	16	14%

JuanitaKirk3	2002 - 2008	13	18	38%
JuanitaKirk4	2002 - 2008	15	18	20%
E1186	2006 - 2009	16	17	6%
Average		14.4	17.0	18%

5.0. WATER QUALITY CALIBRATION

The hydrologic model was calibrated for the following water quality parameters

- Water temperature
- Total suspended solids (TSS)
- Dissolved oxygen (DO)
- Benthic algae
- Fecal coliforms
- Total copper
- Dissolved copper
- Ammonia nitrogen
- Total nitrogen
- Ortho phosphorus
- Total phosphorus

Each parameter was specified whether its loadings are primarily a function of sediment, surface runoff, subsurface (interflow and/or groundwater) runoff, or a combination. For the impervious land surfaces, only sediment loads and surface runoff are used to determine the loadings. All constituent loadings except fecal coliforms are assigned units of pounds. The mass units for fecal coliforms are 10^9 CFUs (colony forming units or organisms).

5.1 Water Temperature

Water Temperature is modeled by performing an energy balance in each stream segment. Heat and energy inputs to the stream are determined from the temperature of nonpoint, point, and boundary inflows; and from meteorological data

Model accuracy for water temperature is assessed by comparing hourly model output to discrete in-situ measurements for calibration points (2G, 3G, 4GO, and 4GI) and continuous measurements recorded at (1G, 5G, 6G, and 7G). Juanita Creek mainstem (1G) has a longer period of record available and was used for this calibration (2007 – 2010). Final calibration shows the model under simulating water temperatures between 2 and 4 degrees Fahrenheit which is approximately 3 to 9 percent error for all but one location (4GO). The wetland outlet was slightly over simulated averaging 2% above observed. The variance is well characterized at all eight locations (Table 11) as conveyed using r-square statistics.

Table 11 Summary statistics for calibrated water temperature.

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
Water Temperature	RMSE	3.31	3.64	2.78	2.62	3.26	3.99	5.40	2.75
	ME	-2.78	-2.53	-1.85	1.32	-2.74	-3.13	-4.65	-1.76

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
(deg-F)	RPD	-5%	-5%	-4%	2%	-5%	-6%	-9%	-3%
	r-square	0.95	0.94	0.95	0.92	0.96	0.92	0.92	0.91

Visual representation allows for the added interpretation of model accuracy not expressed in the statistics above. In Figure 11 below, a set of four graphs representing Juanita Creek mainstem (1G) are illustrated summarizing a time series plot of water temperature (upper left), scatter plot (upper right), cumulative distribution (lower left) and for reference the time series of stream flow for observed and simulated (lower right). Observed are shown in red while simulated in blue. As revealed in the statistics above, the mainstem is generally under simulated by a few percent and is most visible in the cumulative distribution plot (lower left), but overall tracks exceptionally well over multiple seasons and years.

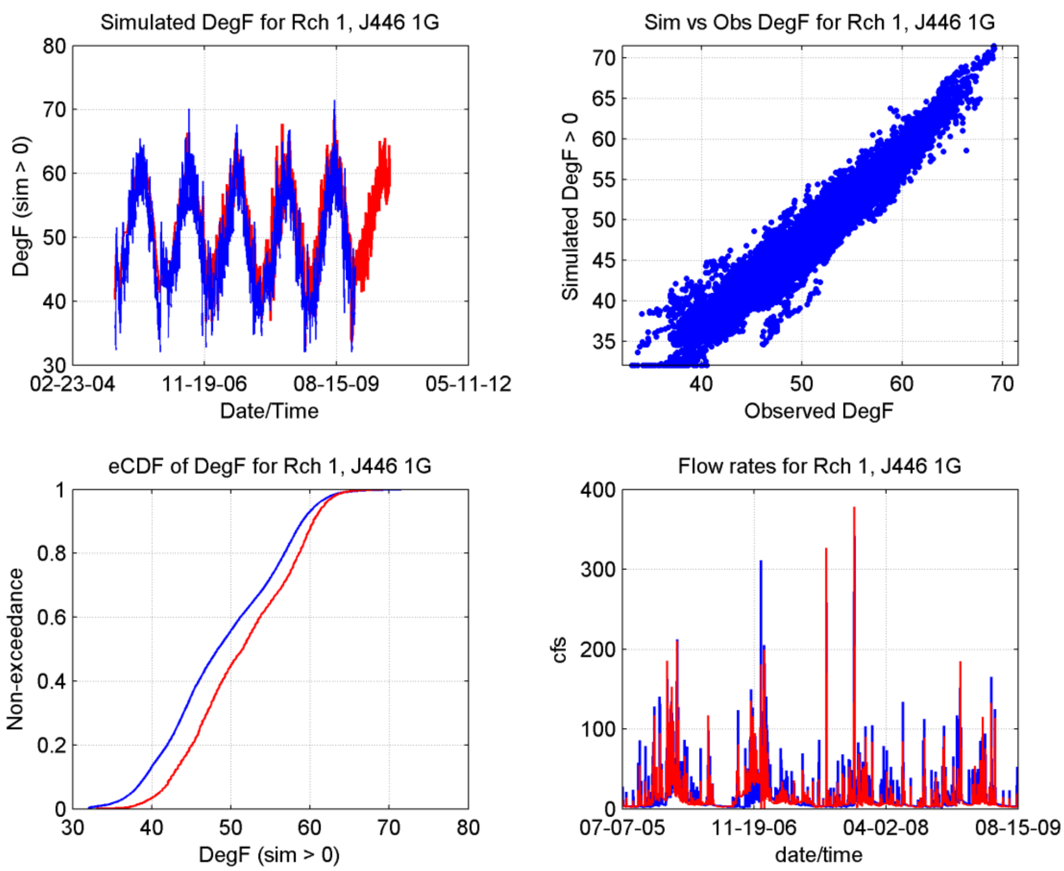

Figure 11 Time series, scatter plot, and cumulative distribution plots of calibrated water temperature for the mainstem. Red is observed, blue is simulated.

5.2 Total Suspended Solids

Sediment is simulated in-stream as three separate size fractions, referred to as sand, silt, and clay. The sediment loadings (in units of tons) are generated using the surface storage and surface runoff results from the hydrologic simulation.

Simulating TSS does not take into account any episodic events that are discrete in nature (e.g. bank failure) and not easily predictable. The goal for TSS calibration is to reasonably simulate annual mass loadings. However, since copper and phosphorus are modeled as adsorbed to solids, characterizing instantaneous concentrations is also important. Statistical comparison of simulated and actual TSS concentrations is presented in Table 12. During parameter adjustment, the mainstem was the focus for minimizing error and improving model predictions.

Table 12 Summary statistics for calibrated total suspended solids.

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
Total Suspended Solids (mg/L)	RMSE	67.95	358.76	47.81	22.92	11.80	188.76	65.34	67.97
	ME	-24.87	-73.73	10.05	13.99	6.27	-98.77	-16.19	-12.88
	RPD	-43%	-50%	48%	240%	66%	-71%	-31%	-24%
	r-square	0.66	0.04	0.00	0.02	0.36	0.11	0.34	0.06

Simulated mainstem concentrations of suspended solids compare well for most of the observed conditions (approximately 70% of observed concentrations are 10 mg/L or below). Between 10 and 300 mg/L, the model under-simulates concentrations. One large event was sampled, and the model generally reflects a similar magnitude in concentrations (Figure 12).

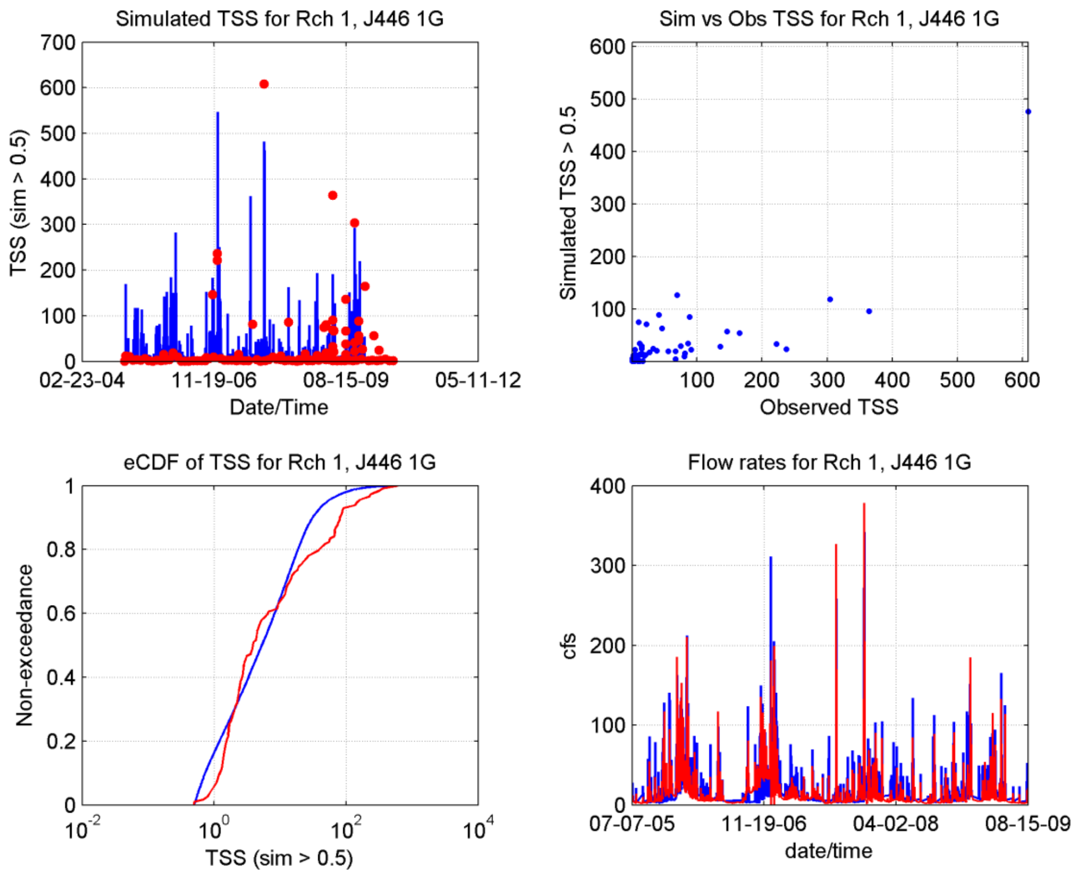

Figure 12 Time series, scatter plot, and cumulative distribution plots of calibrated total suspended solids for the mainstem. Red is observed, blue is simulated.

As reported in Table 12 above, some of the calibration points have statistically poor model accuracy of simulated instantaneous concentrations. However, those statistics are largely affected by observed concentrations most likely a result of some discrete release of fines not included in the model design or calibration, and/or due to very few data points affected by a few outliers. The large observed concentration may be a bank failure, or possible flushing of catch basins in the storm drainage network; either way those source mechanisms are not represented in the model. As an example using Billy Creek (2G), the four graphs in Figure 13 reflect variable concentrations over the period of available data except for the before mentioned condition. Except for Totem Lake tributary (i.e. 3G and 4GO), the other calibration points were similar in results as presented in above Figure 12 and Figure 13 below. Comparing TSS concentrations in the Totem lake tributary again show poor explanation of the variance among instantaneous observed concentrations, but do reflect the general variability of concentrations over time as shown in the time series graph and cumulative distribution graph in Figure 14.

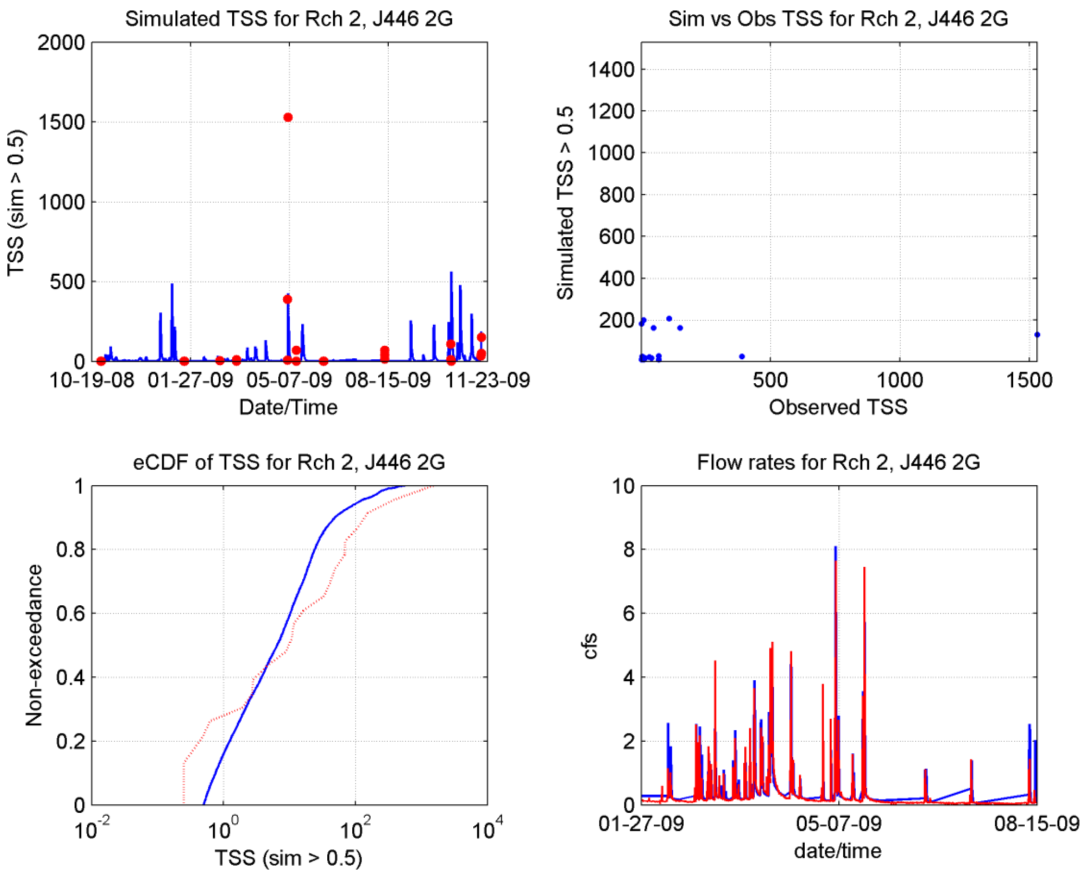

Figure 13 Time series, scatter plot, and cumulative distribution plots of calibrated total suspended solids for Billy Creek (2G). Red is observed, blue is simulated.

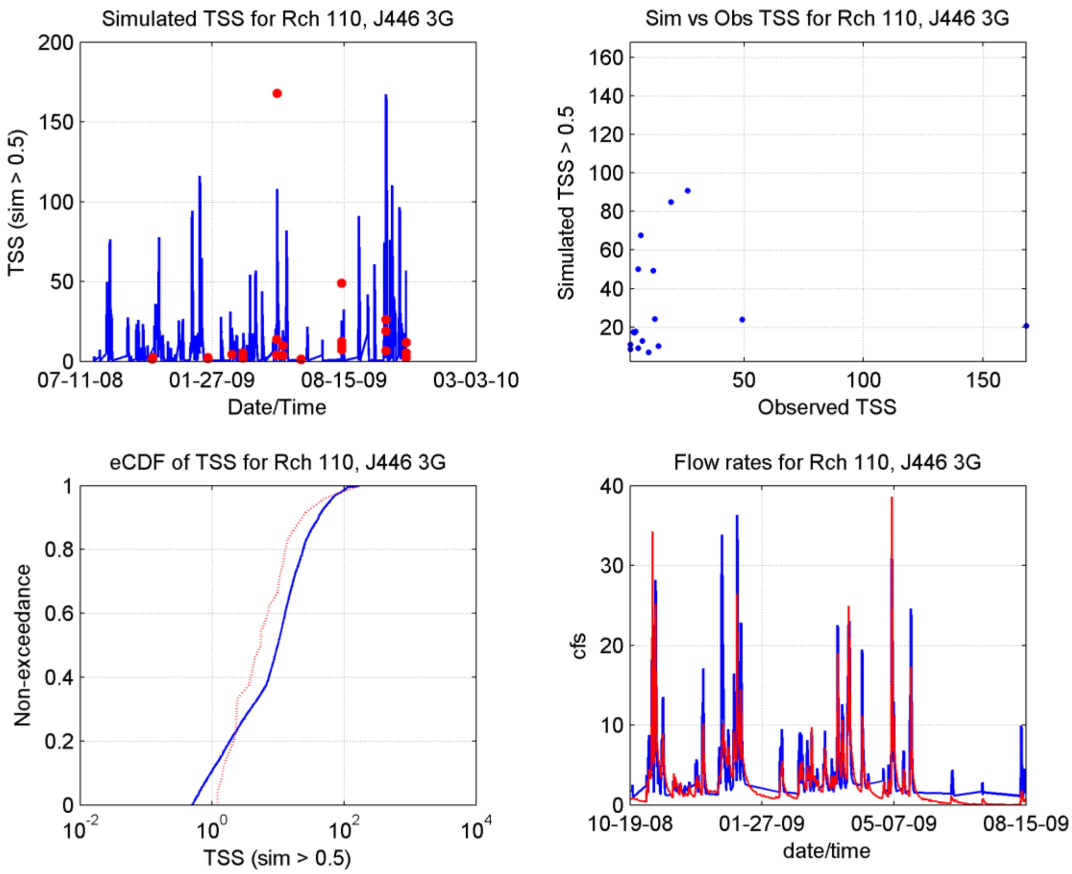

Figure 14 Time series, scatter plot, and cumulative distribution plots of calibrated total suspended solids for Totem Lake tributary (3G). Red is observed, blue is simulated.

5.3 Dissolved Oxygen

Dissolved Oxygen is simulated in the stream by defining the oxygen loads in nonpoint and point runoff, and representing reaeration and biological/chemical processes in the stream. As shown in Table 13, model accuracy has good agreement among calibration points that are more representative of faster moving stream reaches with r-squares ranging from 0.59 to 0.79 (with the exception of Billy Creek—2G), and less accurate for the Totem Lake subbasin (r-squares of 0.42). Billy Creek (2G) dissolved oxygen was measured in a plunge pool below a 3-ft drop out of a culvert. This supersaturated condition of air entrainment in the water column is not represented in the model design or calibration process.

Table 13 Summary statistics for calibrated dissolved oxygen.

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
Dissolved Oxygen (mg/L)	RMSE	0.82	1.55	1.27	3.58	1.59	1.51	1.79	0.71
	ME	-0.45	-1.22	-0.11	2.97	1.14	-1.26	-1.57	-0.06
	RPD	-4%	-10%	-1%	70%	14%	-11%	-14%	-1%
	r-square	0.69	0.43	0.42	0.42	0.79	0.71	0.59	0.59

Further evaluation is illustrated using the same four types of graphs in the following two figures, Figure 15 representing mainstem conditions (1G) and, as an example, illustrating the least accurate calibration location (3G) downstream of Totem Lake and a large wetland (Figure 16). Contrary to the lesser r-square, the model does reflect the seasonal fluctuations of dissolved oxygen over the period of available data except for the late fall/early winter in 2009 where observed concentrations are substantially lower than the previous year.

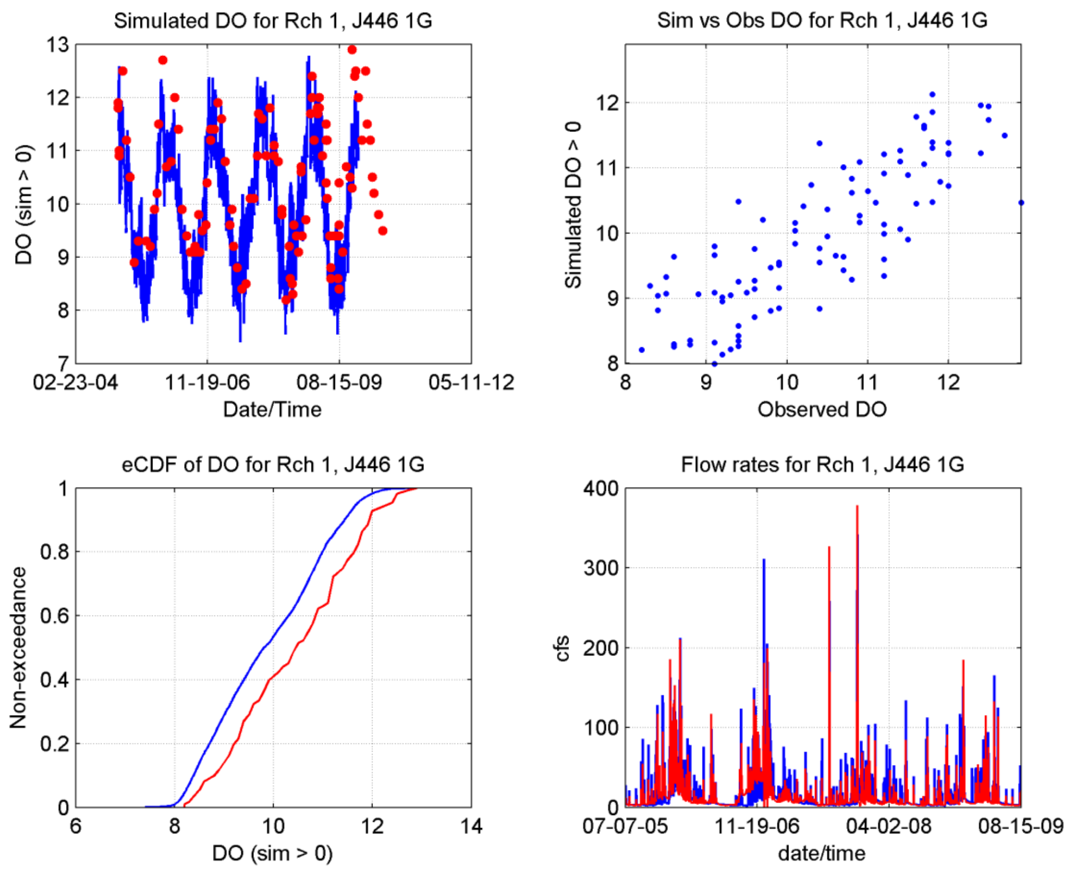

Figure 15 Time series, scatter plot, and cumulative distribution plots of calibrated dissolved oxygen concentrations for the mainstem. Red is observed and blue is simulated.

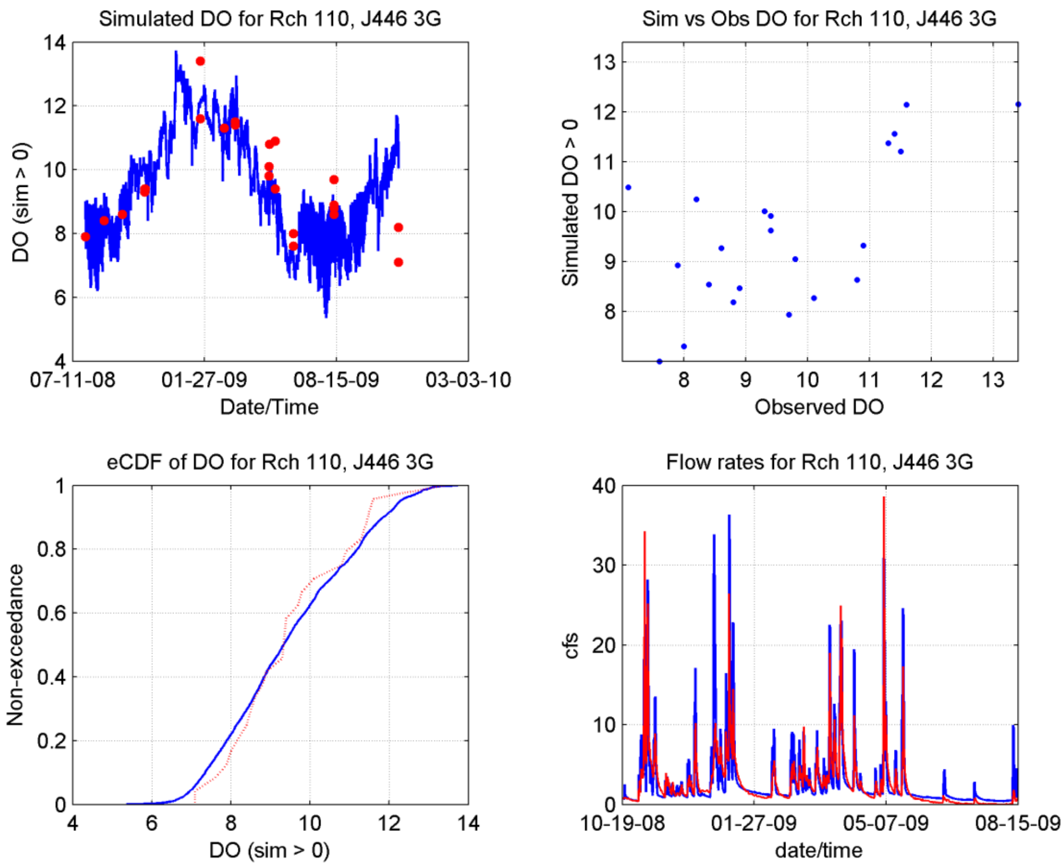

Figure 16 Time series, scatter plot, and cumulative distribution plots of calibrated dissolved oxygen concentrations for the Totem Lake tributary (3G). Red is observed and blue is simulated.

5.4 Benthic Algae

Benthic Algae (or periphyton) is stationary, living organic matter that is modeled using the same Monod growth kinetics that is used to simulate phytoplankton. The material grows, respires, and dies in response to light, nutrient (N and P) availability, and temperature. It takes up and releases nutrients and releases organic matter (detritus) during respiration, sloughing, and death. To adequately characterize the diurnal swings in dissolved oxygen, benthic algae densities, growth and death rates were adjusted sometimes specific to a given catchment reach containing ponds, lakes, and/or wetlands; otherwise the values were consistent among all reaches. However, no data was collected for this parameter to evaluate model accuracy.

5.5 Fecal Coliforms

Fecal coliform concentrations are extremely variable and difficult to predict. One reason for this is that many of the larger loadings of bacterial material probably occur not only during storms, but also during somewhat random but "catastrophic" events, such as failure or illicit sewer connections of waste disposal facilities, which can produce large, unpredictable concentrations. Therefore, efforts were made to attain general agreement between the simulated concentrations by adjusting loading rates, both surface and subsurface runoff-associated by land use. Because of the difficulty in matching actual observed values, the explanatory regression coefficient (i.e. r-square) is used more as guidance than a test of acceptability but still necessary for evaluation given metrics used in scenario analyses are dependent on absolute thresholds of concentrations. Due to the high concentrations and variability, calibrated loading rates for this study should not be used for any other basin.

Model accuracy simulating fecal concentrations is substantially less than the other parameters receiving similar scrutiny. The variance in regressions (r-square) range from 0.0 to 0.26 (Table 14). Only two locations were above r-squares of 0.10, mainstem (1G) and inlet to the large wetland (4GI). Focusing on the mainstem (1G) and the two least accurate tributary calibrations (4GO and 5G), the four graphs in each figure illustrate relevancy comparing instantaneous observed concentrations to simulated over a period of record. Figure 17 illustrates the quality of simulation for the mainstem of Juanita Creek with concentrations within the same order of magnitude except for a large simulated spike early in the model time span. The two least accurate calibrations (4GO and 5G) with r-squares equal to 0.0 visually compare well to measured except during the largest observed concentrations in excess of 4000 cfu/100ml in both cases (Figure 18 and Figure 19). The west branch tributary was observed to have two high concentration events, both substantially under simulated. The other headwater tributary (6G) was similar to 5G in response and (2G, 3G, 4GI, and 7G) were more characteristic of 4GO accuracy.

Table 14 Summary statistics for calibrated fecal coliforms.

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
Fecals (cfu/100 ml)	RMSE	1500	2466	724	1508	910	6265	5815	2274
	ME	-402	-607	101	232	-283	-2405	-2784	-517
	RPD	-49%	-40%	23%	33%	-46%	-70%	-76%	-37%
	r-square	0.18	0.02	0.08	0.00	0.26	0.00	0.01	0.01

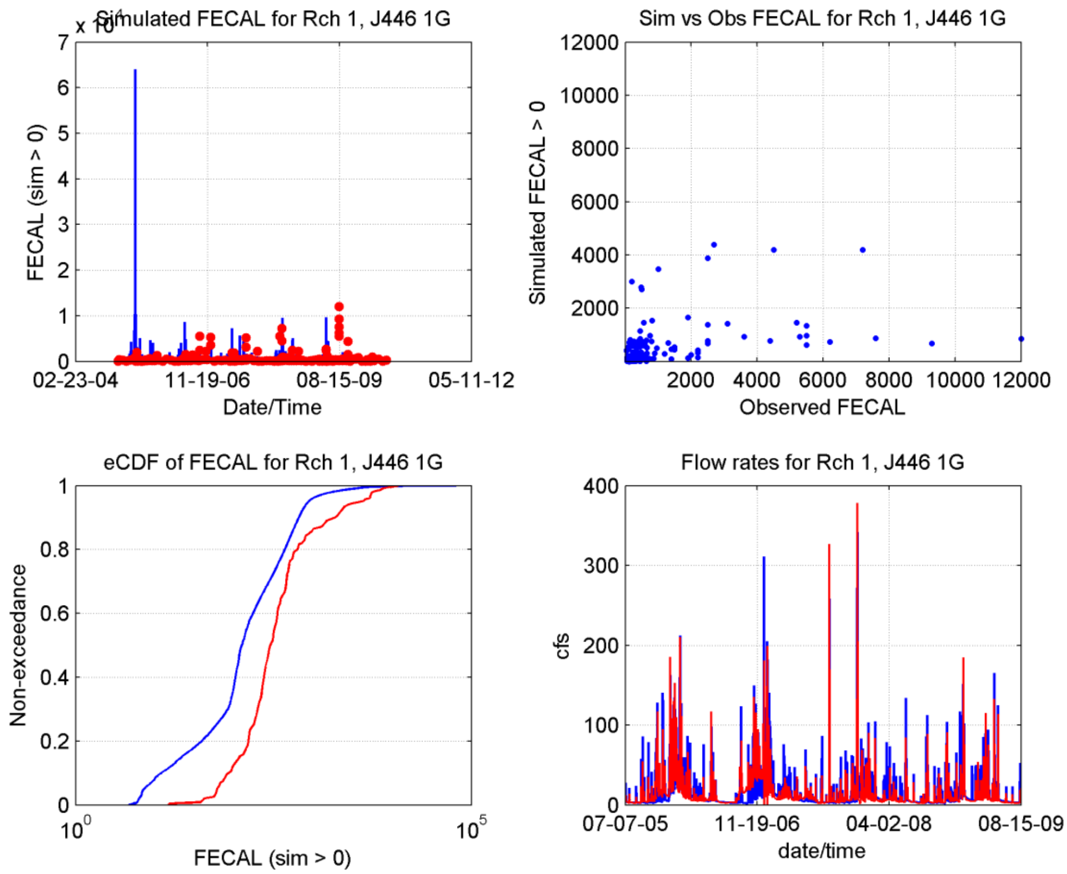

Figure 17 Time series, scatter plot, and cumulative distribution plots of calibrated fecal colony forming units for the mainstem. Red is observed, blue is simulated.

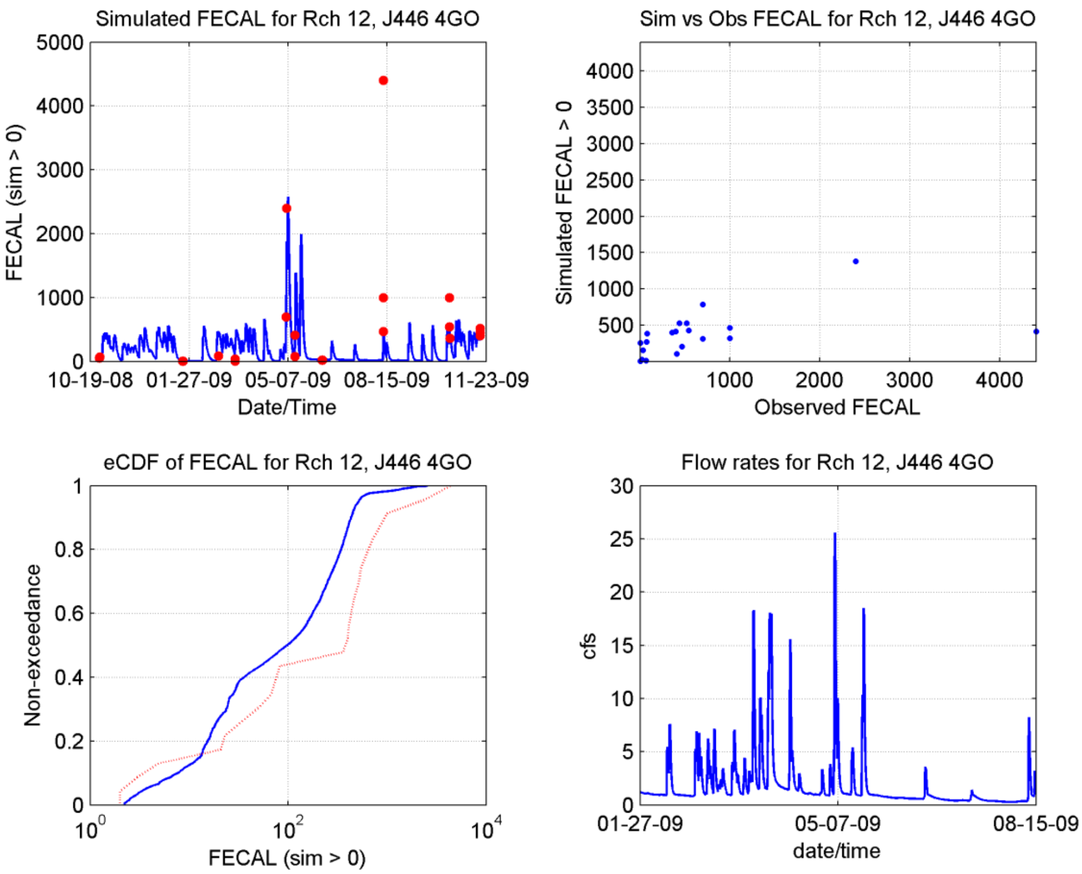

Figure 18 Time series, scatter plot, and cumulative distribution plots of calibrated fecal colony forming units for the outlet of wetland. Red is observed, blue is simulated.

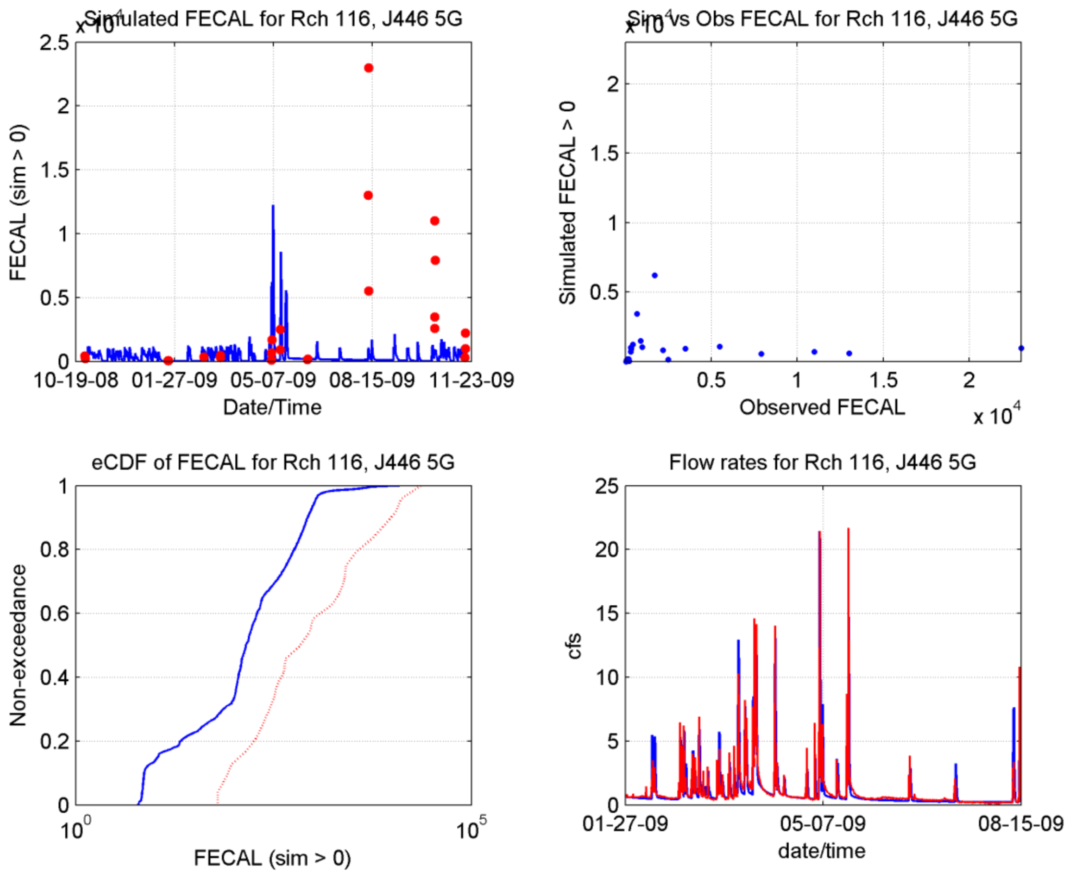

Figure 19 Time series, scatter plot, and cumulative distribution plots of calibrated fecal colony forming units for the west headwater branch tributary. Red is observed, blue is simulated.

5.6 Total Copper

Total copper concentrations were calibrated by adjusting the land use-specific interflow and groundwater concentrations and the surface parameters (potency factors) to achieve a statistical fit with the available data. Copper is sediment-associated, so all surface loading was modeled in the sorbed (i.e. attached) phase.

Total copper r-square values range from .02 to .52 with the wetland outlet (4GO) and north branch (7G) having the lowest r-squares of .02 and .14, respectively (Table 15). However, reviewing the four types of graphs in Figure 20, the model's ability to simulate total copper is better than statistically reported due to substantially under simulated concentrations during one suspiciously large observed event—the Totem lake tributary (3G) r-square value was similarly affected.

Table 15 Summary statistics of calibration for total and dissolved copper.

Parameter	Statistic	Mainstem (1G)	Billy Creek	Totem Lake	Wetland outlet	Wetland inlet	West Branch	East Branch	North Branch

			(2G)	Trib. (3G)	(4GO)	(4GI)	(5G)	(6G)	(7G)
Total Copper (ug/L)	RMSE	10.18	8.00	3.25	6.74	1.71	7.62	8.12	8.08
	ME	0.00	2.10	2.13	1.86	0.94	-0.53	0.66	2.55
	RPD	0%	29%	100%	32%	50%	-6%	9%	38%
	r-square	0.45	0.52	0.26	0.02	0.42	0.35	0.20	0.14

While the wetland outlet concentrations were the least accurately simulated (r-square 0.02), the four graphs in Figure 21 illustrate accuracy seemingly better than reported. Furthermore, the majority of the error can be seen associated with the defined concentrations in the active groundwater component of the model (Figure 22), which is user specified, Adjusting those would require a separate set of parameterization counter to the overall model calibration method—no unique submodels.

The north branch calibration point (7G) shows a divergence between under and over simulating events (scatter plot in Figure 23). Further reviewing the time series, there appears to be a shift in timing between like magnitudes (Figure 24). The cause is unknown, but peculiar. The other simulated calibration points were similar to or better in model accuracy using visual inspection.

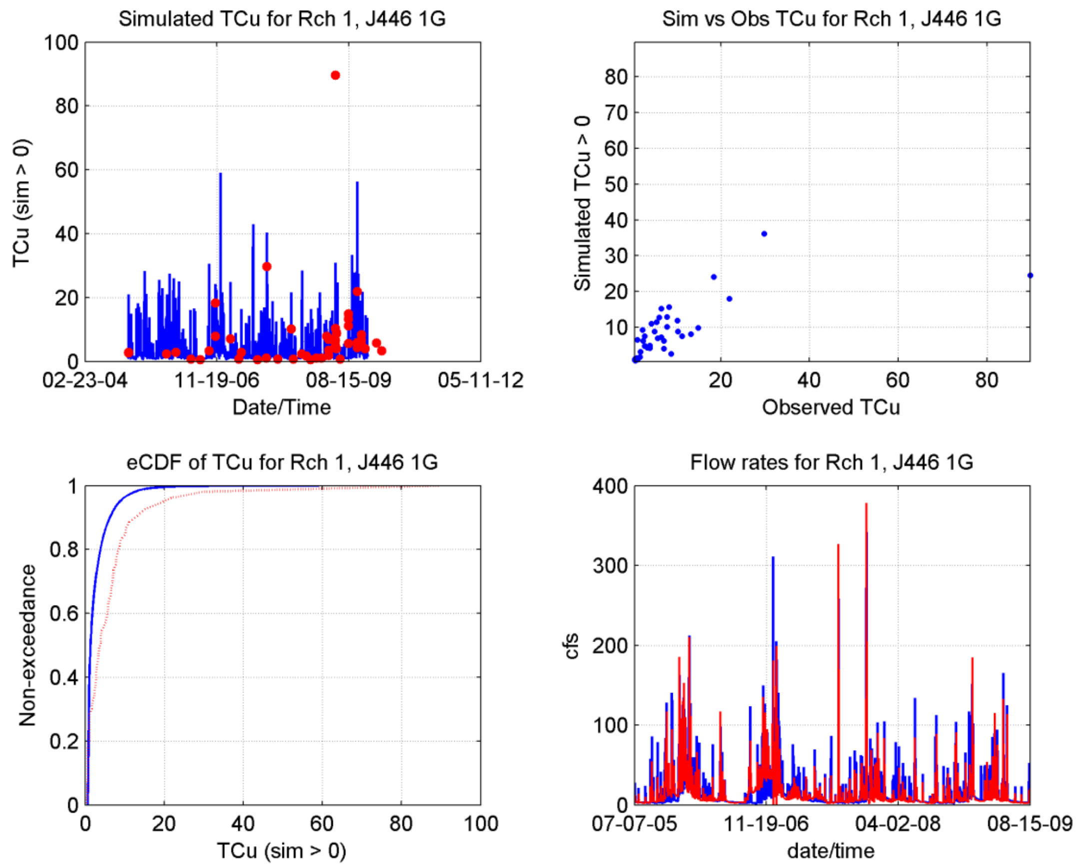

Figure 20 Time series, scatter plot, and cumulative distribution plots of calibrated total copper concentrations for the mainstem. Red colors are observed while blue is simulated.

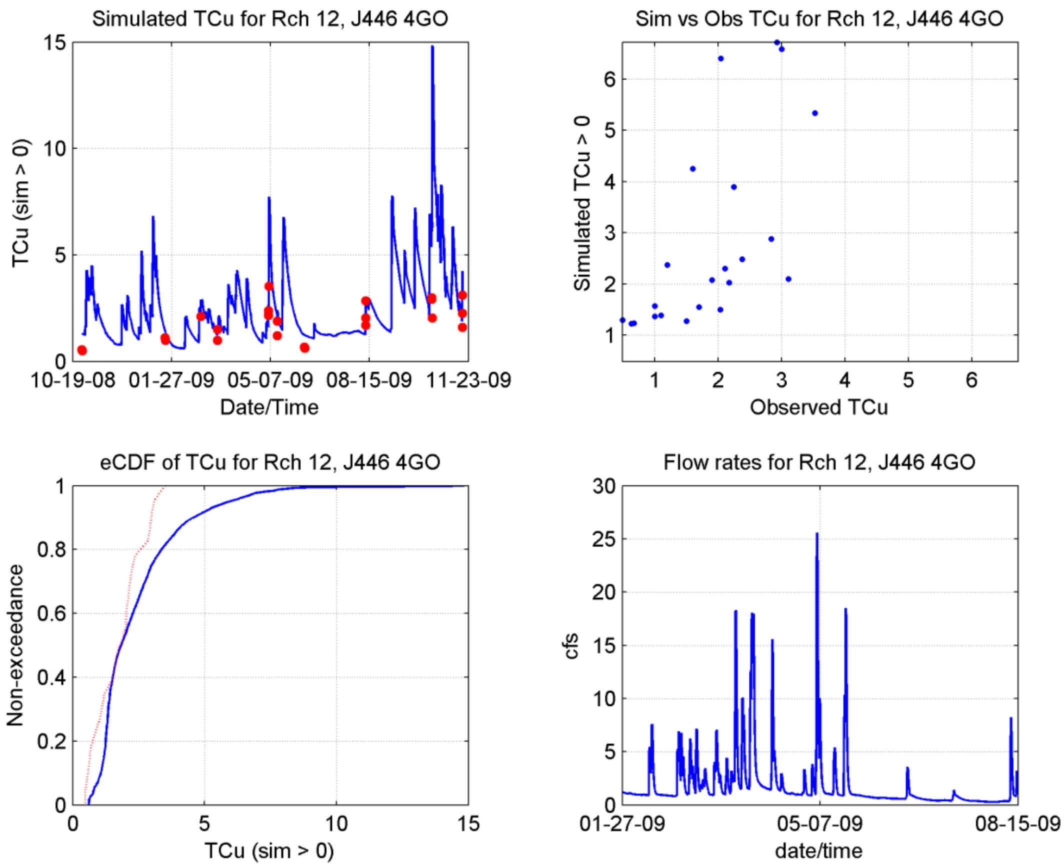

Figure 21 Time series, scatter plot, and cumulative distribution plots of calibrated total copper concentrations for the wetland outlet (4GO). Red colors are observed while blue is simulated. Stream flows were not recorded at this location.

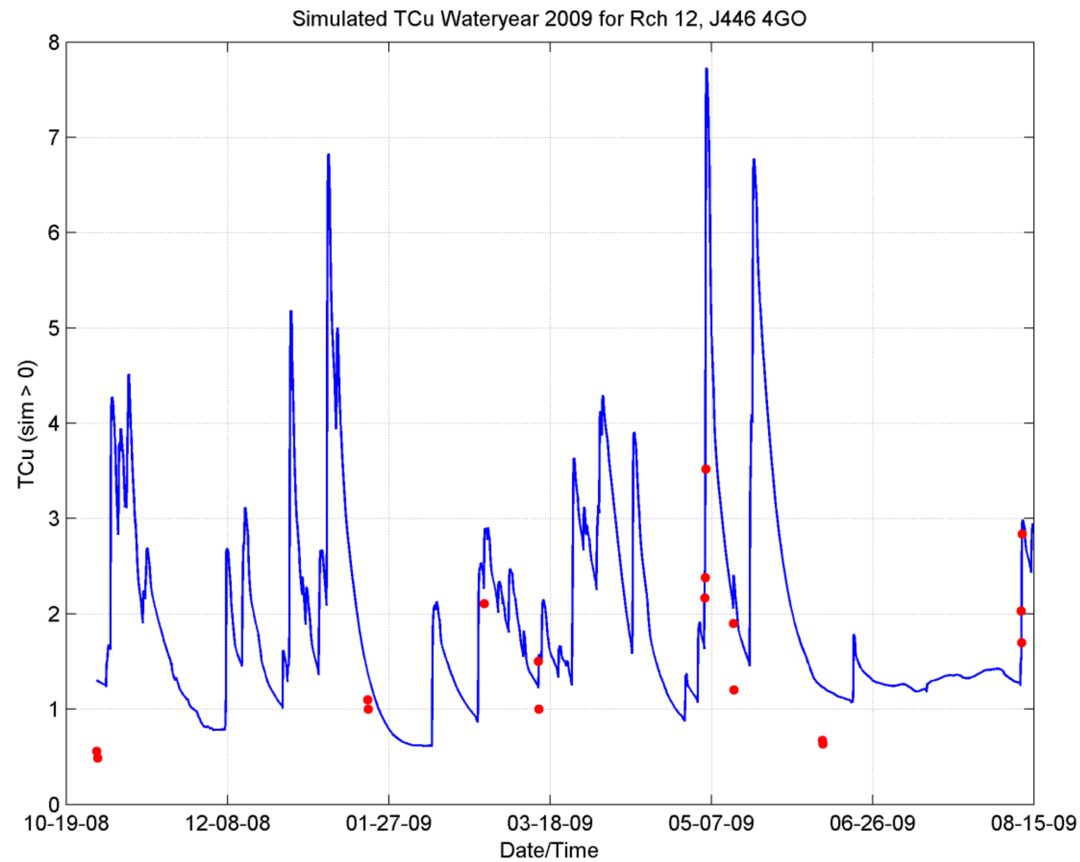

Figure 22 Time series plot of total copper concentrations for outlet of wetland (4GO). Red colors are observed, blue is simulated.

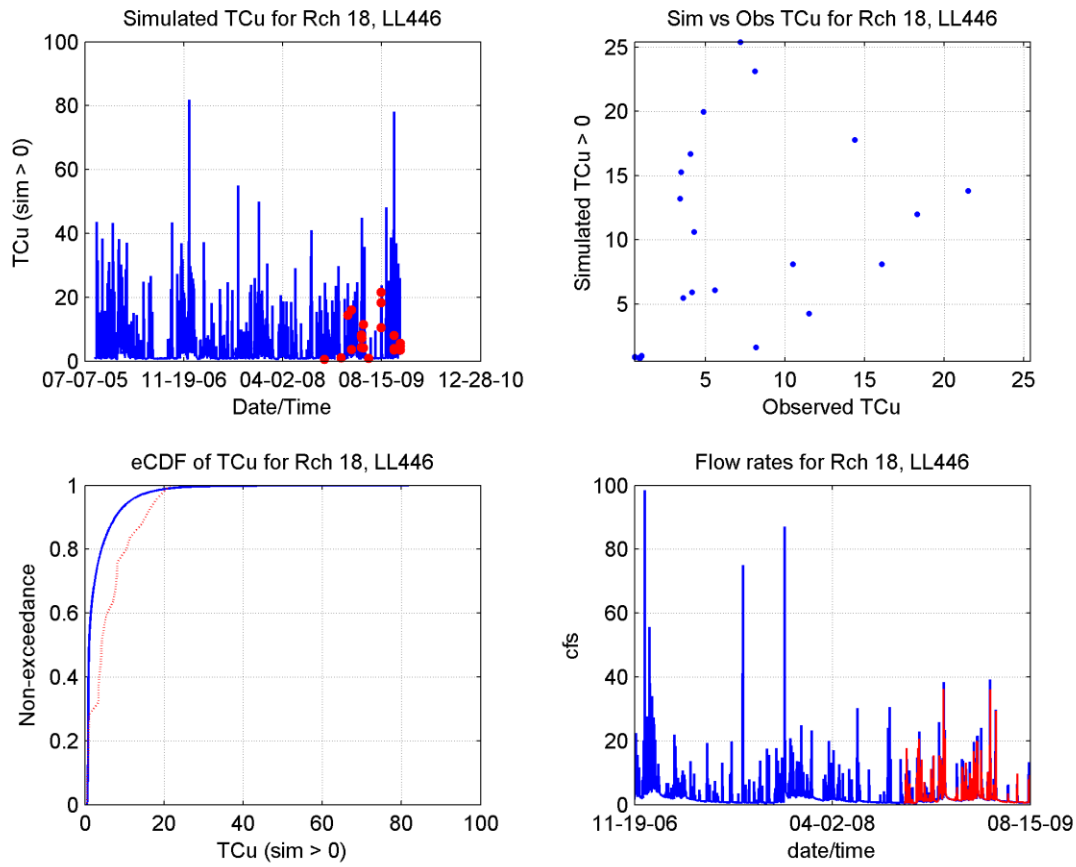

Figure 23 Time series, scatter plot, and cumulative distribution plots of calibrated total copper concentrations for the north branch (7G). Red colors are observed while blue is simulated.

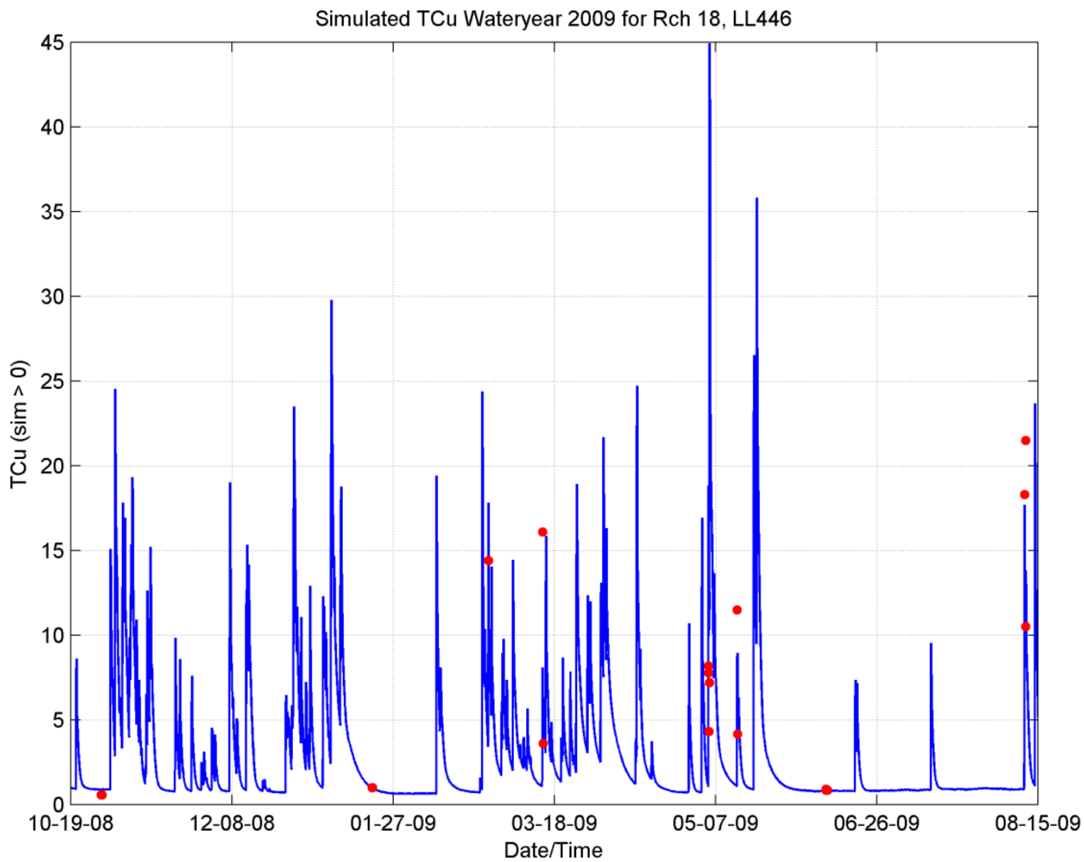

Figure 24 Time series plot of total copper concentrations for north branch tributary (7G). Red colors are observed, blue is simulated.

5.7 Dissolved Copper

The level of model accuracy is generally better modeling total copper as opposed to dissolved copper which is dependent on other time varying environmental factors such as hardness and concentration of suspended solids. Again, since metrics used to evaluate modeled scenarios relies on acute and chronic concentrations, the same higher level of statistical scrutiny is applied to simulated results on instantaneous concentrations.

Dissolved copper r-square values ranged from 0.01 to 0.49 (Table 13), where the 0.49 value here was higher than the total copper simulation for the same calibration point (0.26). Similar to the model capabilities of simulating total copper, dissolved copper has relatively poor statistical accuracy but overall reflects the variability in concentrations (Figure 25). However, visual inspection of time series plots (Figure 26 through Figure 33) illustrates comparable results between simulated and observed when allowing for some shifts in timing beyond the instantaneous data values used testing model accuracy, with one exception—wetland outflow concentrations at 4GO. Sequestering of dissolved copper in the wetland is not well characterized in the model as can be seen with persistent over simulating concentrations (Figure 29). This is likely due to sequestering from vegetative uptake not present in the fate/transport of the model design.

Table 16 Summary statistics of calibration for total and dissolved copper.

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
Dissolved Copper (ug/L)	RMSE	2.03	2.49	0.90	2.97	3.24	3.65	2.62	1.91
	ME	0.81	0.80	0.53	-0.70	2.57	0.27	-0.21	-0.46
	RPD	39%	36%	44%	-19%	189%	9%	-7%	-18%
	r-square	0.15	0.01	0.49	0.03	0.12	0.02	0.04	0.07

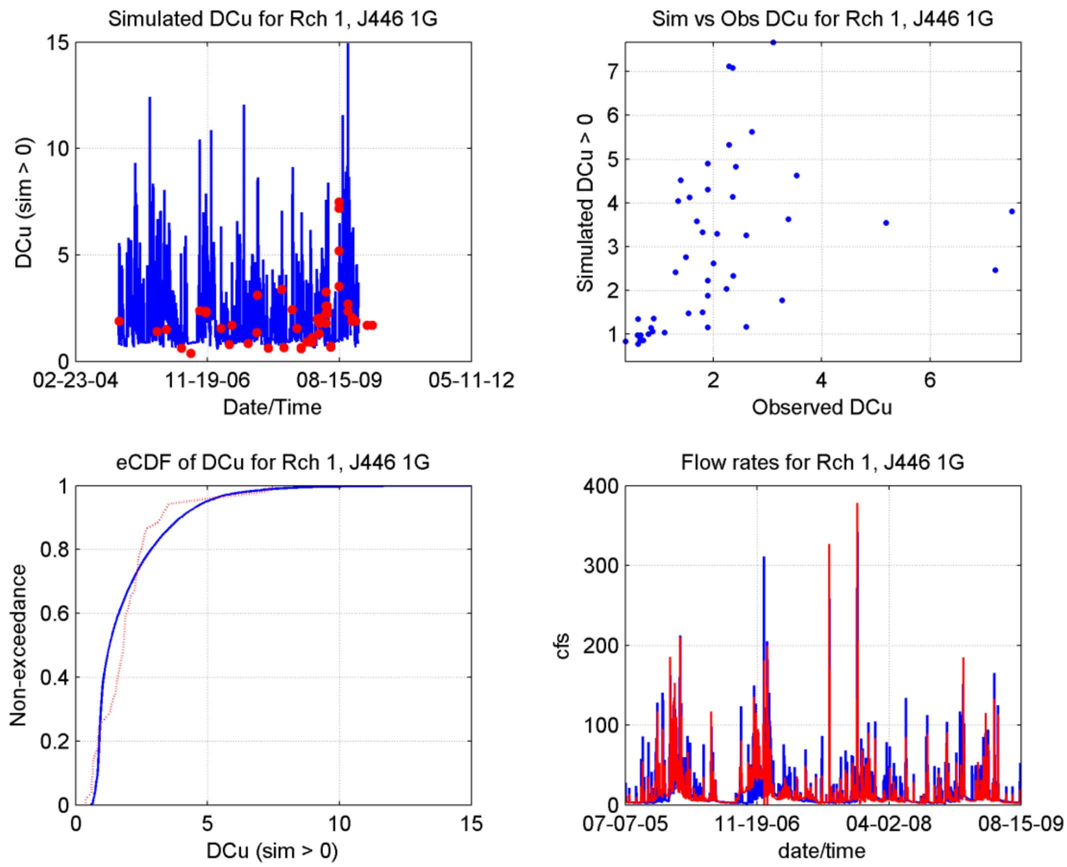

Figure 25 Time series, scatter plot, and cumulative distribution plots of calibrated for dissolved copper concentrations for the mainstem.

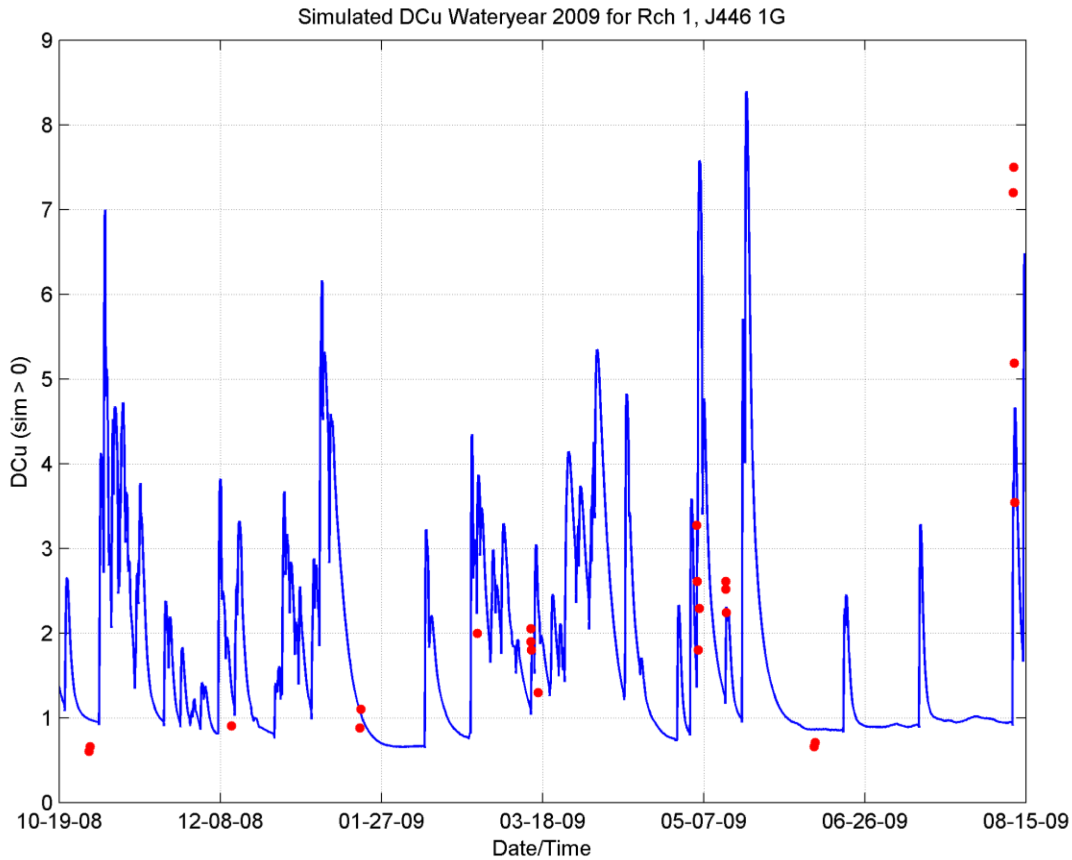

Figure 26 Time series plot of dissolved copper concentrations for mainstem (1G). Red colors are observed, blue is simulated.

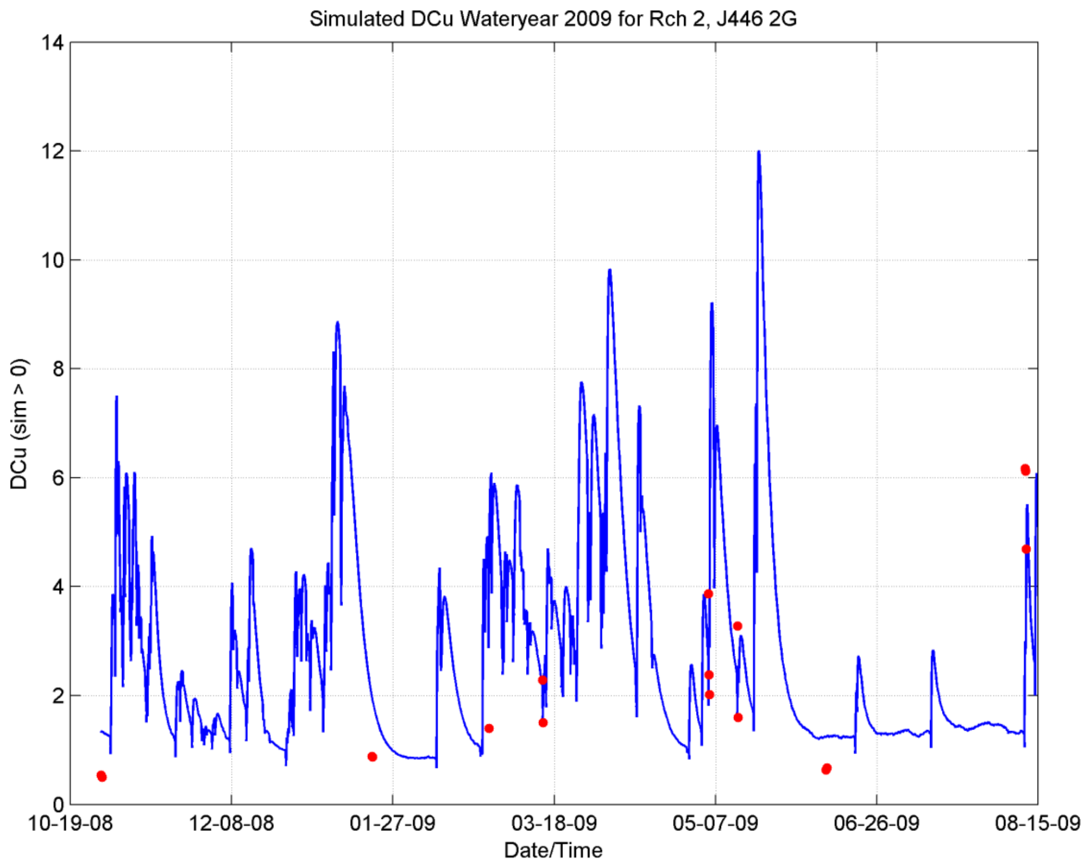

Figure 27 Time series plot of dissolved copper concentrations for Billy Creek (2G). Red colors are observed, blue is simulated.

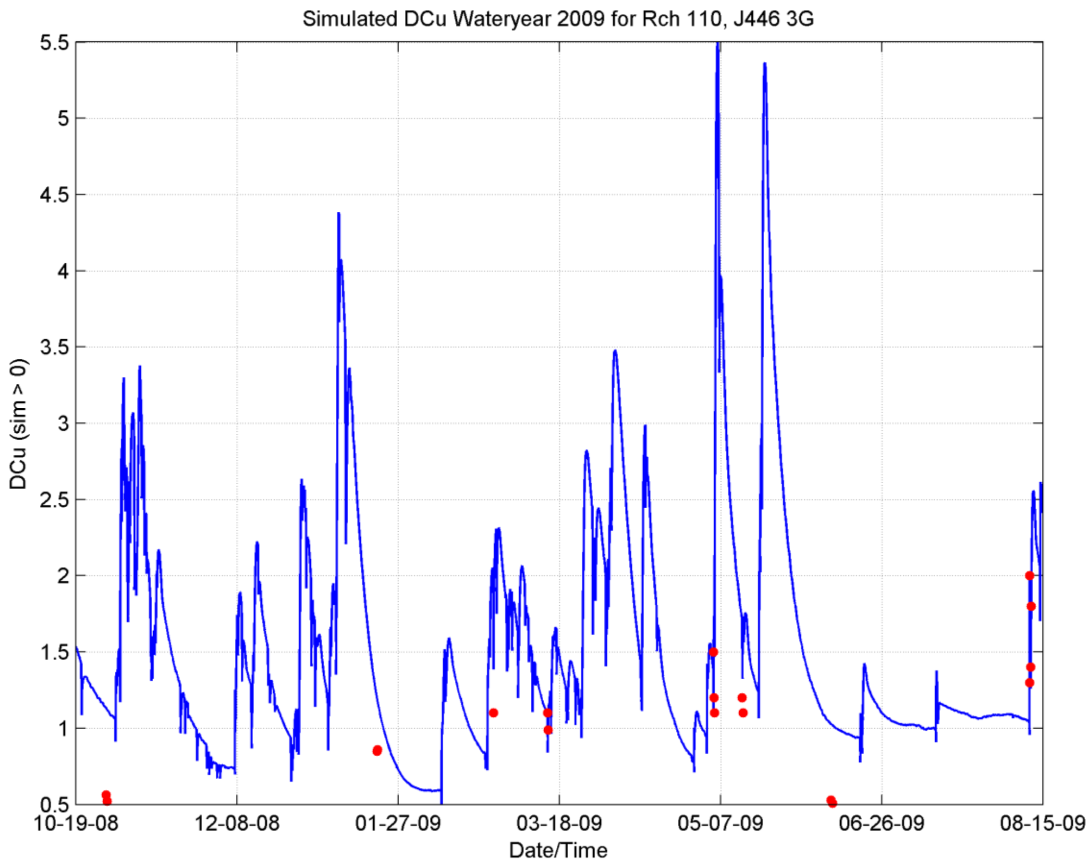

Figure 28 Time series plot of dissolved copper concentrations for Totem Lake tributary (3G). Red colors are observed, blue is simulated.

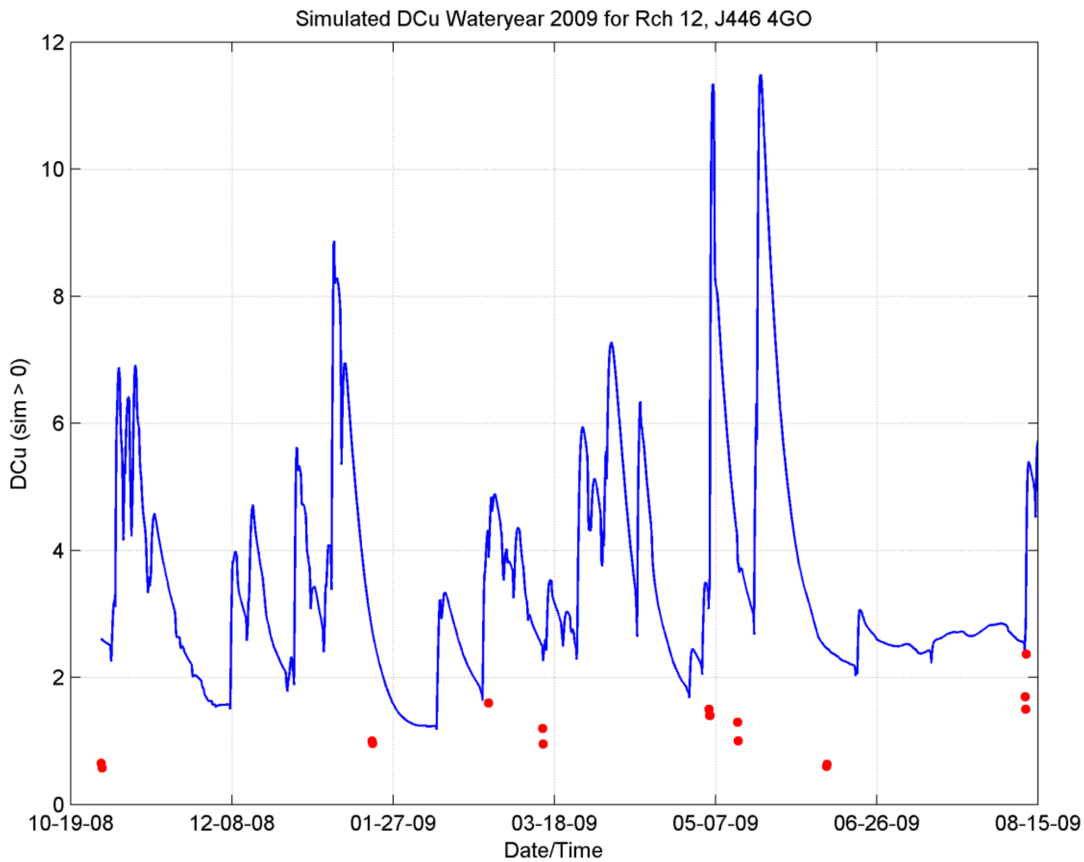

Figure 29 Time series plot of dissolved copper concentrations for outlet of wetland (4GO). Red colors are observed, blue is simulated.

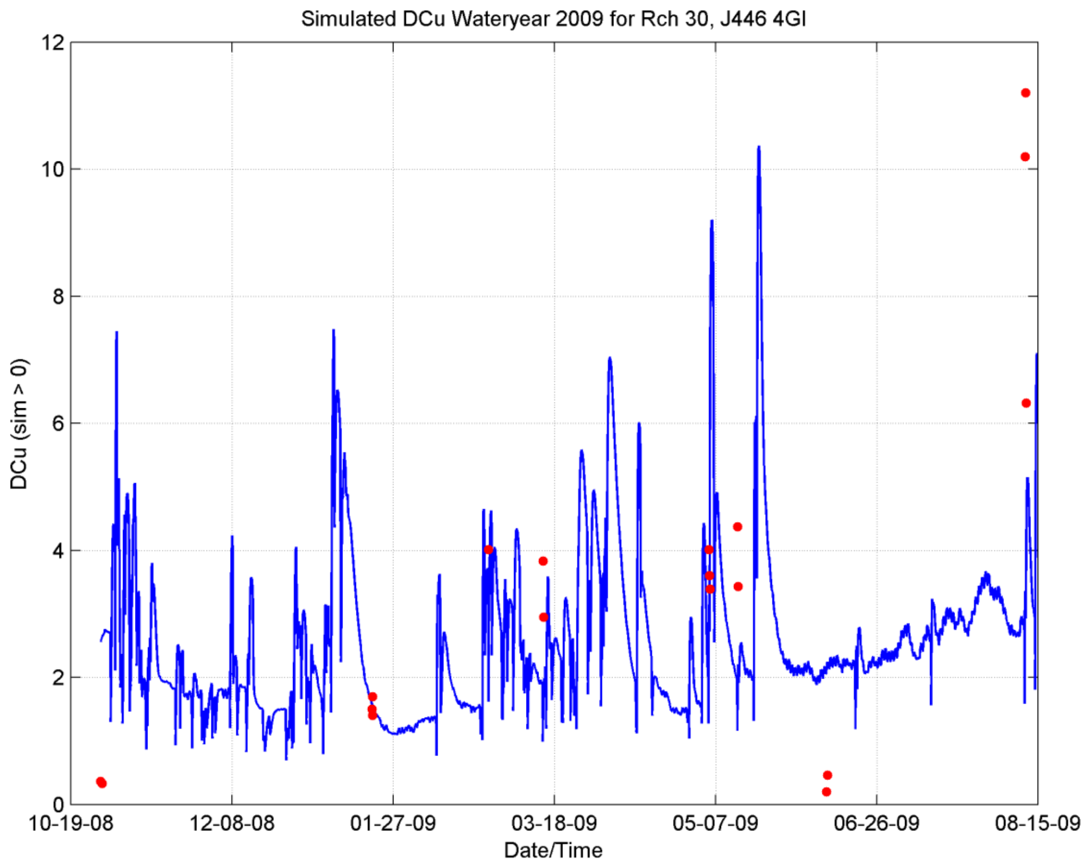

Figure 30 Time series plot of dissolved copper concentrations for inlet to wetland (4GI). Red colors are observed, blue is simulated.

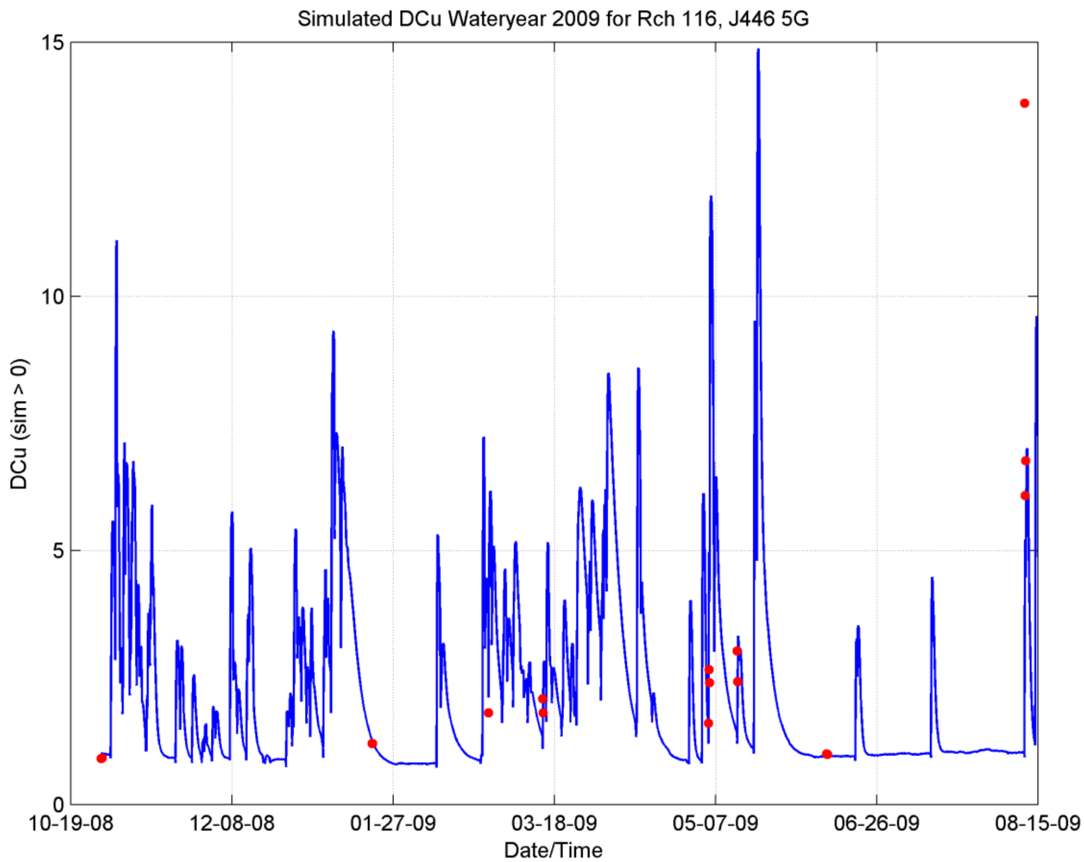

Figure 31 Time series plot of dissolved copper concentrations for west branch tributary (5G). Red colors are observed, blue is simulated.

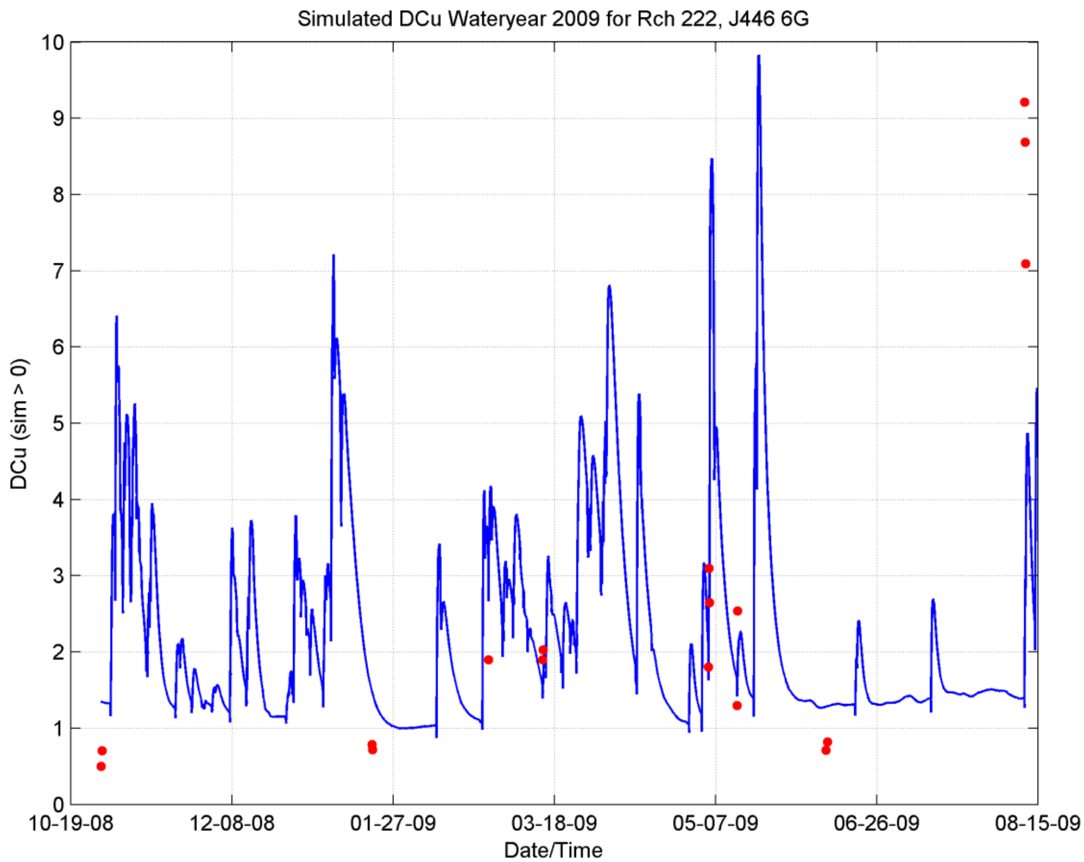

Figure 32 Time series plot of dissolved copper concentrations for east branch tributary (6G). Red colors are observed, blue is simulated.

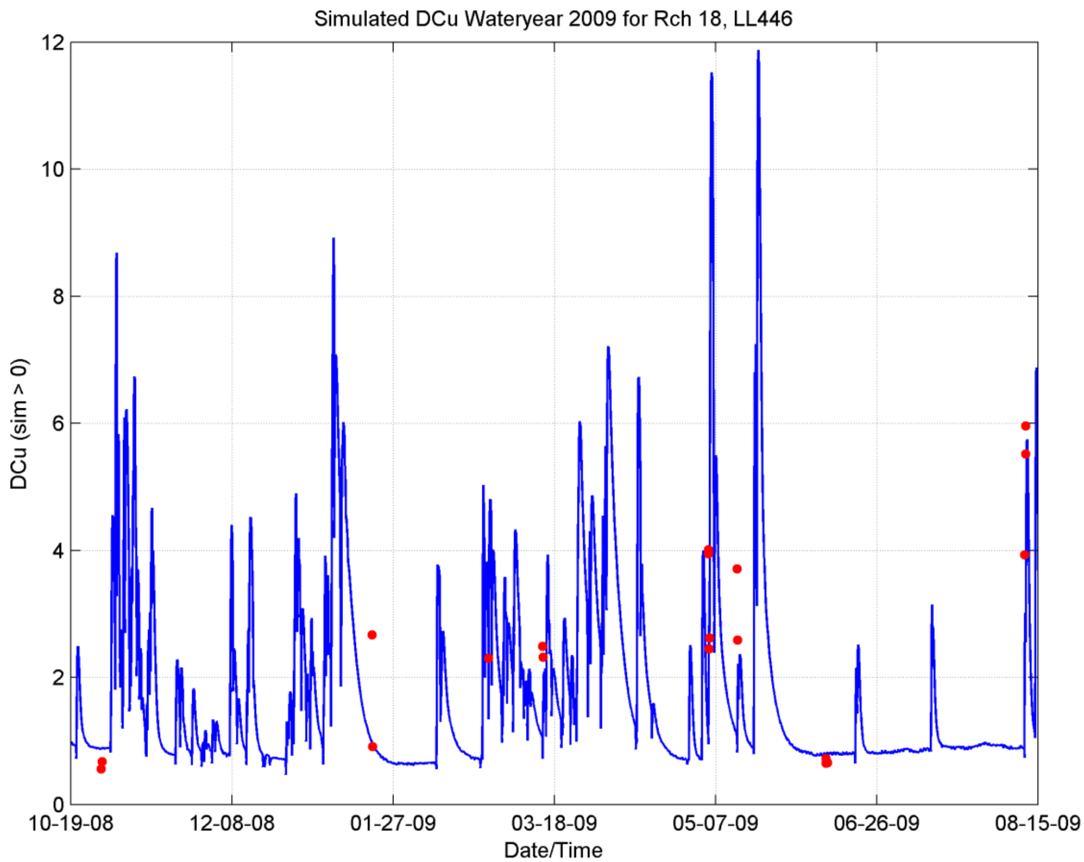

Figure 33 Time series plot of dissolved copper concentrations for north branch tributary (7G). Red colors are observed, blue is simulated.

5.8 Nutrients

Metrics used for evaluating nutrient loads in defined scenarios are based on annual loading rates for Juanita Creek mainstem. Therefore the test for model acceptance is based on performing a non-parametric test of distributions using the Mann-Whitney *U*-Test with a p-value greater than 0.10 as acceptable similarity in distributions and summarized in Table 17 below. Further detail about the calibration parameters are in the follow subsections.

Table 17 Summary of Mann-Whitney U-Test of calibration for nitrogen and phosphorus species.

Mann-Whitney U-test for Juanita Creek mainstem (1G)		
Parameter	p-value	Test
Nitrates	0.01	Fail*
Ammonia-N	0.24	Pass
Orthophosphorus	0.41	Pass
Total Phosphorus	0.31	Pass

*Passes two of the three headwater tributaries (.75, .57,.08)

<u>Ammonia-N</u>

Ammonia-N is modeled by generating nonpoint loadings from surface runoff, interflow, and groundwater. Ammonia is assumed to be exclusively in dissolved form, and not associated with sediment.

Modeled mainstem ammonia concentrations do not reflect the high ammonia concentration measured during discrete storm events. Two observed events have concentrations three times and ten times all other observed events during the five years of reported values (Figure 34). Those two events appear to have substantial duration associated with them suggesting a systemic (possibly seasonal) source. Without further understanding the causality of those, replicating them in the model would be tenuous and lead to excessively high annual loading rates (Horner, et al. 1994); thus, were ignored during parameter adjustments but not removed from the dataset used testing model accuracy. Simulated mainstem ammonia concentrations pass the U-test with a p-value of 0.24. See Table 18 at the end of this section for other statistics.

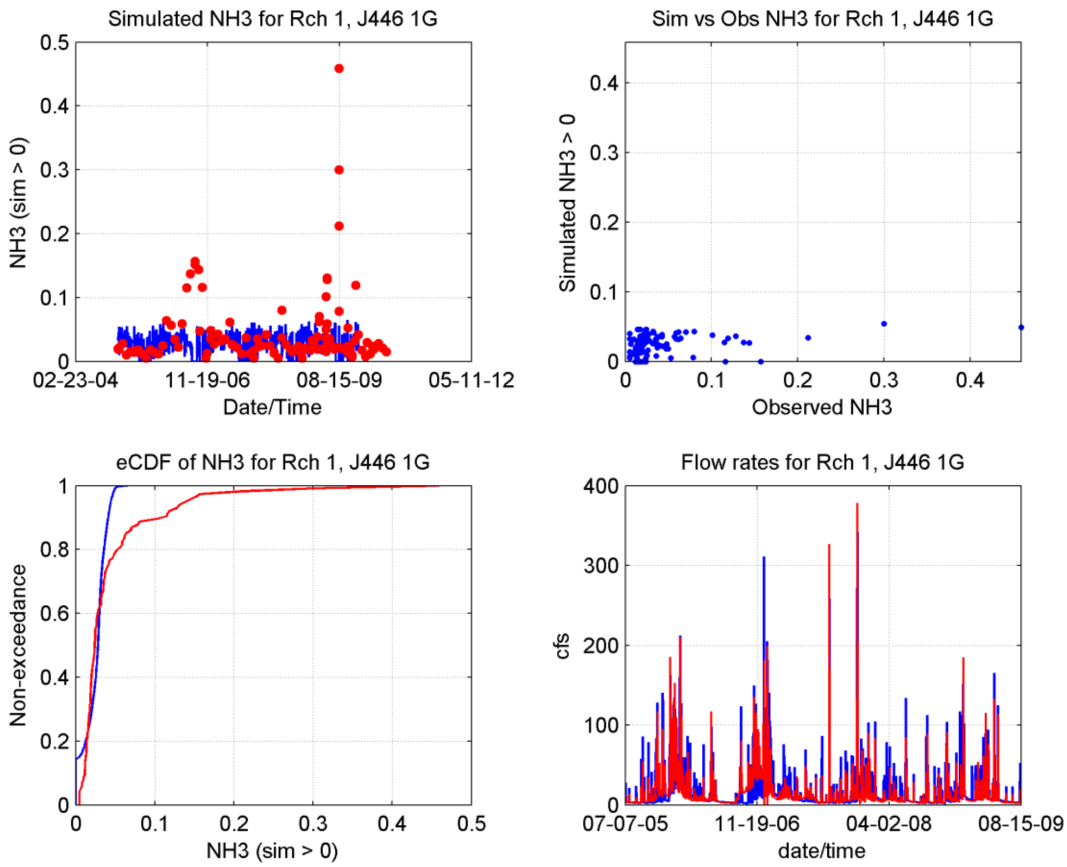

Figure 34 Time series, scatter plot, and cumulative distribution plots of calibrated for ammonia-n concentrations for the mainstem (1G). Red is observed and blue is simulated.

Nitrate-N

Nitrate-N is modeled similarly to ammonia, based on surface accumulation, washoff with surface runoff, and definition of monthly-varying interflow and groundwater concentrationsCalibration of nitrate and ammonia was largely done by adjusting the interflow and groundwater concentrations (and ammonia surface loading factors) by land use, until the errors were minimized at the eight calibration points. While the agreement was fairly good for nitrate (r-square > 0.50) at four of the eight calibration points (3G, 4GO, 6G, and 7G), the mainstem r-square value was 0.26 (Table 19). However, there is enough model error to fail the U-test (p-value approximately 0.01). This model error can be seen in the scatter plot and cumulative distribution as shown in Figure 35 below.

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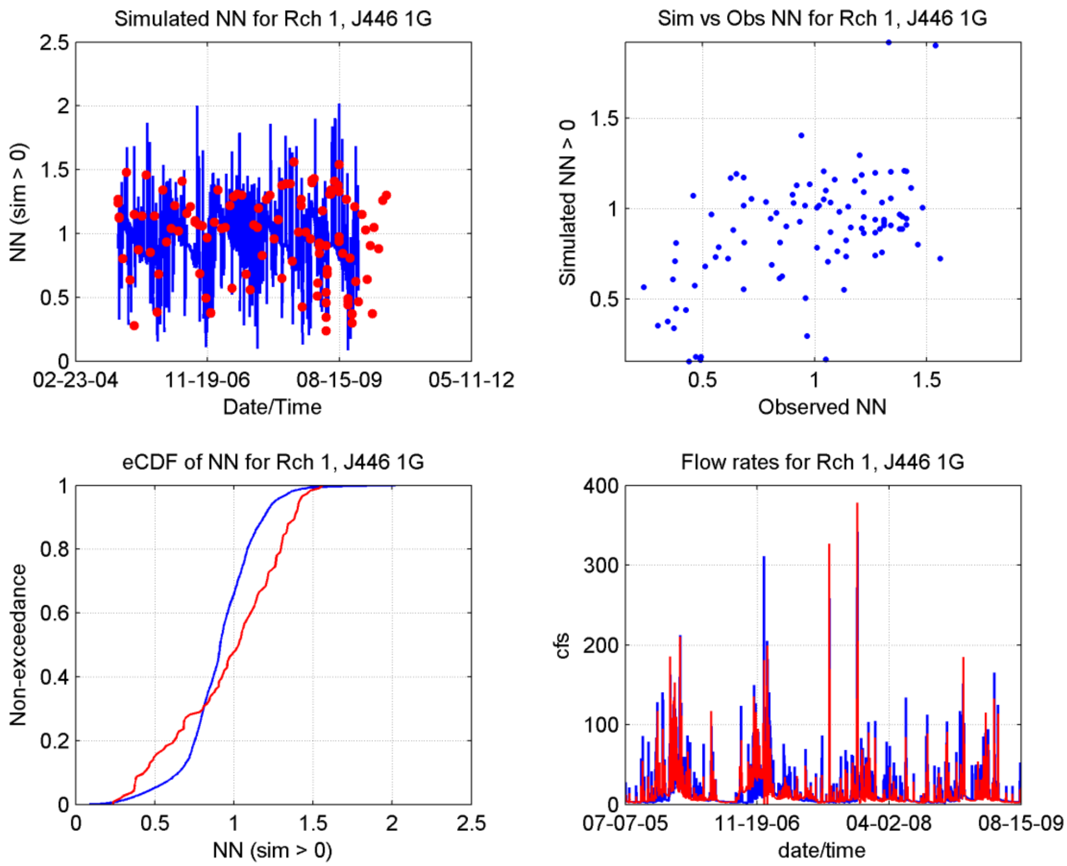

Figure 35 Time series, scatter plot, and cumulative distribution plots of calibrated for nitrate concentrations for the mainstem (1G). Red is observed and blue is simulated.

Orthophosphate-P

Orthophosphate-P is modeled similarly to ammonia with one major difference. Because of its propensity to be bound to solids, it is associated with sediment instead of surface runoff and is also associated with sediment in the stream, and loadings are calculated based on surface accumulation, wash-off in association with sediment that is transported to the stream, and definition of monthly-varying interflow and groundwater concentrations. Similar to ammonia-n, two drawn out events are observed with concentrations twice other peaks reported during a five year period. Correlations are poor, but have similar distributions (Figure 36). Without further understanding the cause of those elevated concentrations, annual loadings would be substantially higher than supported by the multiple years of observed data. Orthophosphate concentrations were calibrated by adjusting the land use-specific interflow and groundwater concentrations and the surface parameters (potency factors) seasonally to achieve a fit ignoring those two events. The statistical measures indicate the model is poorly calibrated with respect to instantaneous

values (Table 20), but passes the U-test of fitness with a p-value of 0.41. Note that storms produce spikes of PO4, which is primarily from the surface-generated particulate P.

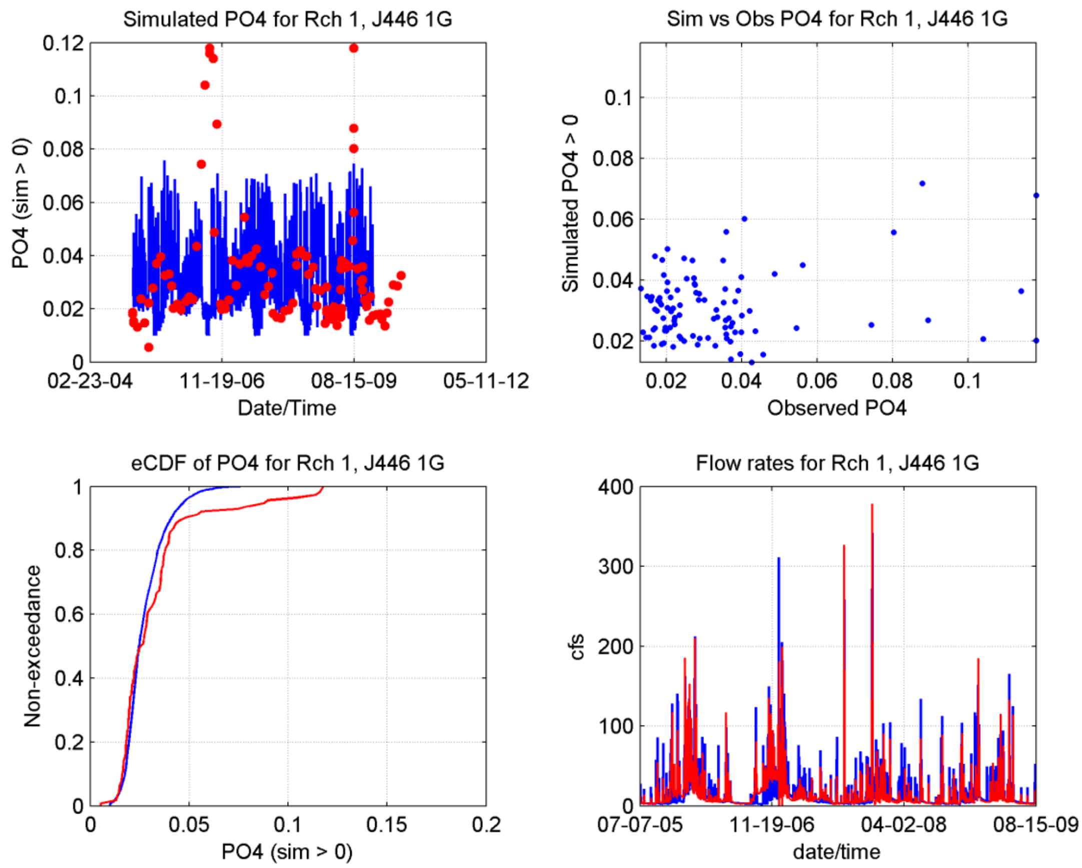

Figure 36 Time series, scatter plot, and cumulative distribution plots of calibrated for orthophosphorus concentrations for the mainstem (1G). Red is observed and blue is simulated.

Table 18 Summary statistics for calibration of ammonia-N.

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
Ammonia-N (mg/L)	RMSE	0.06	0.06	0.02	0.28	0.04	0.28	0.11	0.10
	ME	-0.02	0.03	-0.01	-0.16	-0.02	-0.07	-0.05	-0.05
	RPD	-44%	130%	-50%	-90%	-94%	-62%	-70%	-67%
	r-square	0.08	0.02	0.18	0.07	0.14	0.00	0.03	0.24

Table 19 Summary statistics for calibration of nitrates.

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
Nitrate (mg/L)	RMSE	0.34	0.78	0.63	0.94	0.60	0.53	0.40	0.33
	ME	-0.09	-0.51	0.50	0.76	0.55	-0.04	0.22	0.12
	RPD	-10%	-37%	152%	502%	500%	-4%	24%	16%
	r-square	0.26	0.27	0.54	0.53	0.27	0.17	0.51	0.64

Table 20 Summary statistics for orthophosphate.

Parameter	Statistic	Mainstem (1G)	Billy Creek (2G)	Totem Lake Trib. (3G)	Wetland outlet (4GO)	Wetland inlet (4GI)	West Branch (5G)	East Branch (6G)	North Branch (7G)
Ortho-phosphate (mg/L)	RMSE	0.02	0.04	0.03	0.03	0.02	0.06	0.03	0.02
	ME	0.00	-0.03	0.03	0.02	0.02	-0.01	0.00	0.00
	RPD	-8%	-42%	148%	121%	134%	-18%	7%	9%
	r-square	0.05	0.32	0.00	0.34	0.05	0.13	0.02	0.18

6.0. DISCUSSION

There are two approaches in calibrating a model when multiple locations of observed data are available: 1) build multiple sub models (seven to eight would be required for this study) and calibrate each one with likely unique parameterization, or 2) minimize model error among all observation locations and weight parameters with best fits focusing on Juanita Creek mainstem. Since land use in the study area is fairly uniform with the exception of Totem Lake drainage subbasin dominated by commercial land use, there was not enough variability to support multiple calibrated models generating multiple unique parameter datasets. Thus, model calibration was conducted minimizing error among the calibration points, but when necessary, giving more weight to the mainstem.

The rigor of the statistical testing of the calibrated model was one of three levels; with the most comprehensive testing include using a suite of statistics characterizing various aspects of the hydrologic regime and how well the model replicates it. Based on the suite of statistics used, the model is well calibrated at all locations except during the lowest flow conditions in Totem Lake tributary. This deficiency is not believed to substantially affect outcomes from the various stormwater management scenarios evaluated for this study.

The next level of rigor includes four statistics (root-mean-square-error, mean error, relative percent difference, and r-square) applied to simulated and observed water quality parameters requiring competency predicting instantaneous conditions throughout the period of record. These parameters include water temperature, dissolved oxygen, dissolved copper, and fecal coliforms. Because copper is sediment associated, the same level of statistical rigor was applied to suspended solids as well. Overall the model is well calibrated for water temperature, and dissolved oxygen throughout the study area. However, as complexity of a parameter increases either mechanistically and/or with uncertainty, the model accuracy diminishes. Total copper was generally simulated with acceptable accuracy among all the calibration points except near the outlet of the large wetland (4GO). As previously mentioned this is likely due to plant uptake in the wetland not represented in the model. Dissolved copper simulations were generally less accurate but more variable among calibration points. Emphasis was given to best characterizing the mainstem of Juanita Creek. However, processes simulating dissolved copper were statistically best modeled at the confluence of the Totem Lake tributary (3G). Further evaluations of copper at the various calibration points using visual inspections of time series, scatter plots, and cumulative distributions reveal that other than slight shifts in timings, or the occurrence of an “anomalous” event, simulated copper concentrations are well characterized at all but one calibration point (4GO).

Simulated fecal coliform concentrations generally follow observed concentrations except for the highest concentrations when the model is under simulating those conditions. It is believed that there are multiple cross-connections, illicit connections, and failing onsite septic systems within the Juanita Creek basin. These types of conditions can generate instantaneous loads not predictable with any consistency without substantially more modeling effort and likely a different modeling scheme or framework. Thus while the

model accuracy is relatively poor, it is believed to be usable for evaluating mitigation strategies relative to each other in the study.

Nutrients were generally less accurately modeled, but were tested for equivalency using the Mann-Whitney *U*-test—the least level of statistical rigor. All but nitrates passed the test focusing the mainstem of Juanita Creek. This model deficiency is likely a result of the before mentioned wide spread septic sewer loads as revealed in the monitoring of fecal coliform concentrations. So while the model fails the test, it still generally reflects conditions in the basin and can be used for such applications with this understanding.

Overall the model is weakest in the Totem Lake tributary system (3G, 4GO, and 4GI), but this is understandable given the greater level of uncertainty characterizing the hydrodynamics and nutrient cycle for Totem Lake (proper), a large wetland downstream, and the flat highly vegetative stream reach connecting the two. Notwithstanding these uncertainties, the model does characterize the various elements throughout the study area, but with some inconsistency. We conclude that the model is sufficiently well calibrated throughout the basin for assessing stormwater management scenario impacts on flows. We also conclude that the model is sufficiently well calibrated only in the mainstem for assessing stormwater management scenario impacts on water quality.

7.0. REFERENCES

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APPENDIX F. DETAILED COST ESTIMATE WORKSHEETS

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Level2 - Dry Pond - LULC: Future									
Basin	**POND FOOTPRINT (AC)**	**Land Cost /Acre**	**Land Cost**	**Level II - Dry Pond Det. Pond Volume (ac-ft)**	**Pond Volume (cu yds)**	**Excavation cost* (cu yds)**	****Soft Costs**	**Total Ecv. and Soft Costs**	**Total Cost for Construction**
WA3001	4.224	$1,300,000.00	$5,490,577.30	26.76	43,179.56	$259,077	$90,677.07	$349,754.43	$5,840,331.73
WA3002	0.425	$1,300,000.00	$552,581.90	2.15	3,471.15	$20,827	$7,289.41	$28,116.31	$580,698.21
WA3003	3.769	$1,300,000.00	$4,900,142.00	23.75	38,308.62	$229,852	$80,448.09	$310,299.78	$5,210,441.78
WA3004	1.485	$1,300,000.00	$1,930,992.70	8.78	14,157.54	$84,945	$29,730.84	$114,676.10	$2,045,668.80
WA3005	0.724	$1,300,000.00	$940,644.90	3.96	6,390.85	$38,345	$13,420.78	$51,765.85	$992,410.75
WA3006	1.440	$1,300,000.00	$1,872,319.80	8.49	13,690.25	$82,142	$28,749.53	$110,891.05	$1,983,210.85
WA3007	2.147	$1,300,000.00	$2,791,465.30	13.06	21,072.20	$126,433	$44,251.62	$170,684.80	$2,962,150.10
WA3008	1.291	$1,300,000.00	$1,678,779.70	7.53	12,153.74	$72,922	$25,522.85	$98,445.27	$1,777,224.97
WA3009	0.869	$1,300,000.00	$1,129,733.80	4.86	7,848.08	$47,088	$16,480.96	$63,569.42	$1,193,303.22
WA3010	2.994	$1,300,000.00	$3,892,574.40	18.62	30,035.39	$180,212	$63,074.33	$243,286.69	$4,135,861.09
WA3011	4.857	$1,300,000.00	$6,313,457.80	30.99	49,990.31	$299,942	$104,979.65	$404,921.51	$6,718,379.31
WA3012	3.928	$1,300,000.00	$5,106,884.90	18.18	29,326.91	$175,961	$61,586.52	$237,548.01	$5,344,432.91
WA3013	2.823	$1,300,000.00	$3,669,268.20	12.88	20,777.55	$124,665	$43,632.87	$168,298.19	$3,837,566.39
WA3014	24.251	$1,300,000.00	$31,525,949.00	117.55	189,642.33	$1,137,854	$398,248.89	$1,536,102.86	$33,062,051.86
WA3015	2.598	$1,300,000.00	$3,377,429.90	16.01	25,829.69	$154,978	$54,242.35	$209,220.51	$3,586,650.41
WA3016	3.376	$1,300,000.00	$4,388,822.10	21.14	34,103.24	$204,619	$71,616.80	$276,236.21	$4,665,058.31
WA3017	10.529	$1,300,000.00	$13,687,349.00	69.22	111,682.74	$670,096	$234,533.75	$904,630.19	$14,591,979.19
WA3018	3.945	$1,300,000.00	$5,128,636.50	24.91	40,191.89	$241,151	$84,402.97	$325,554.32	$5,454,190.82
WA3019	6.533	$1,300,000.00	$8,492,649.10	42.22	68,120.93	$408,726	$143,053.96	$551,779.56	$9,044,428.66
WA3020	1.256	$1,300,000.00	$1,632,853.30	7.31	11,790.34	$70,742	$24,759.72	$95,501.78	$1,728,355.08
WA3021	18.458	$1,300,000.00	$23,995,244.00	89.06	143,685.88	$862,115	$301,740.36	$1,163,855.66	$25,159,099.66
WA3022	2.373	$1,300,000.00	$3,085,336.80	83.62	134,902.88	$809,417	$283,296.05	$1,092,713.34	$4,178,050.14
WA3023	16.013	$1,300,000.00	$20,816,549.00	73.22	118,133.62	$708,802	$248,080.60	$956,882.33	$21,773,431.33
WA3024	0.892	$1,300,000.00	$1,159,057.90	3.52	5,675.57	$34,053	$11,918.70	$45,972.13	$1,205,030.03
WA3025	5.292	$1,300,000.00	$6,879,694.90	23.11	37,288.28	$223,730	$78,305.39	$302,035.06	$7,181,729.96
WA3026	1.033	$1,300,000.00	$1,343,184.70	5.90	9,510.88	$57,065	$19,972.85	$77,038.15	$1,420,222.85
WA3027	1.321	$1,300,000.00	$1,717,708.20	7.72	12,462.15	$74,773	$26,170.51	$100,943.41	$1,818,651.61
WA3028	6.222	$1,300,000.00	$8,088,265.90	26.95	43,478.59	$260,872	$91,305.04	$352,176.58	$8,440,442.48
WA3029	0.632	$1,300,000.00	$821,897.70	2.45	3,951.44	$23,709	$8,298.03	$32,006.69	$853,904.39
WA3030	15.258	$1,300,000.00	$19,835,296.00	69.05	111,403.86	$668,423	$233,948.10	$902,371.26	$20,737,667.26
					Total Project Cost =				$207,522,624

***Excavation cost = $6 per cu yd**
****Soft Costs = 35%**

LID40 - LID with 40% TIA Capture - LULC:Future						
Basin	POND FOOTPRINT (AC)	Land Cost /Acre	Land Cost	LID Pond Area (ac)	LID Pond Area (sq ft)	Construction cost**
WA3001	0	0	0	11.53	502,259.87	$10,045,197.36
WA3002	0	0	0	1.2523	54,550.19	$1,091,003.76
WA3003	0	0	0	7.2878	317,456.57	$6,349,131.36
WA3004	0	0	0	4.213	183,518.28	$3,670,365.60
WA3005	0	0	0	1.1888	51,784.13	$1,035,682.56
WA3006	0	0	0	2.357	102,670.92	$2,053,418.40
WA3007	0	0	0	4.4203	192,548.27	$3,850,965.36
WA3008	0	0	0	3.5565	154,921.14	$3,098,422.80
WA3009	0	0	0	2.4703	107,606.27	$2,152,125.36
WA3010	0	0	0	6.6825	291,089.70	$5,821,794.00
WA3011	0	0	0	7.7118	335,926.01	$6,718,520.16
WA3012	0	0	0	6.0473	263,420.39	$5,268,407.76
WA3013	0	0	0	2.673	116,435.88	$2,328,717.60
WA3014	0	0	0	27.356	1,191,605.58	$23,832,111.60
WA3015	0	0	0	6.2678	273,025.37	$5,460,507.36
WA3016	0	0	0	9.7123	423,067.79	$8,461,355.76
WA3017	0	0	0	17.713	771,578.28	$15,431,565.60
WA3018	0	0	0	4.0918	178,238.81	$3,564,776.16
WA3019	0	0	0	14.418	628,026.30	$12,560,526.00
WA3020	0	0	0	3.5255	153,570.78	$3,071,415.60
WA3021	0	0	0	12.122	528,047.39	$10,560,947.76
WA3022	0	0	0	5.0715	220,914.54	$4,418,290.80
WA3023	0	0	0	15.803	688,369.97	$13,767,399.36
WA3024	0	0	0	1.297	56,497.32	$1,129,946.40
WA3025	0	0	0	5.09	221,720.40	$4,434,408.00
WA3026	0	0	0	1.54	67,082.40	$1,341,648.00
WA3027	0	0	0	2.1023	91,576.19	$1,831,523.76
WA3028	0	0	0	6.4485	280,896.66	$5,617,933.20
WA3029	0	0	0	0.688	29,969.28	$599,385.60
WA3030	0	0	0	17.218	749,994.30	$14,999,886.00
				Total Project Cost =		$184,567,379

**LID Construction Cost = $20 per sq ft

ECY08 - Proposed Regulatory 8% - LID with 80% TIA Capture with Detention Ponds- LULC: Future														
Basin	Pond Foot print(ac)	Land Cost per Acre	Land Cost	Det. Pond Volume (ac-ft)	Det. Pond Volume (cu yds)	Wet Pond Volume (cu yds)	Total Pond Volume (cu yds)	Excavation cost*	Soft Costs***	LID Pond Area (ac)	LID Pond Area (sq ft)	LID Ponds Construction cost**	Construction Cost for Detention	Total Construction Cost
WA3001	1.47	$1,300,000.00	$1,907,989	6.46	10,424.47	20,975.24	31,399.71	$188,398	$65,939.39	23.06	1,004,519.74	$20,090,395	$2,162,327	$22,252,722
WA3002	0.05	$1,300,000.00	$61,695	0.11	184.11	2,109.56	2,293.67	$13,762	$4,816.70	2.50	109,100.38	$2,182,008	$80,274	$2,262,282
WA3003	3.22	$1,300,000.00	$4,188,431	14.79	23,857.75	14,456.13	38,313.87	$229,883	$80,459.13	14.58	634,913.14	$12,698,263	$4,498,773	$17,197,036
WA3004	0.82	$1,300,000.00	$1,068,972	3.47	5,592.53	7,132.02	12,724.55	$76,347	$26,721.55	8.43	367,036.56	$7,340,731	$1,172,041	$8,512,772
WA3005	0.30	$1,300,000.00	$390,716	1.13	1,823.72	1,992.20	3,815.92	$22,896	$8,013.44	2.38	103,568.26	$2,071,365	$421,625	$2,492,990
WA3006	0.02	$1,300,000.00	$26,949	0.03	55.84	4,717.69	4,773.53	$28,641	$10,024.40	4.71	205,341.84	$4,106,837	$65,615	$4,172,451
WA3007	1.14	$1,300,000.00	$1,479,691	4.92	7,944.50	8,171.71	16,116.20	$96,697	$33,844.02	8.84	385,096.54	$7,701,931	$1,610,232	$9,312,163
WA3008	0.04	$1,300,000.00	$54,644	0.10	155.69	6,520.81	6,676.50	$40,059	$14,020.65	7.11	309,842.28	$6,196,846	$108,724	$6,305,569
WA3009	0.05	$1,300,000.00	$61,195	0.11	182.06	4,220.28	4,402.34	$26,414	$9,244.92	4.94	215,212.54	$4,304,251	$96,854	$4,401,105
WA3010	0.14	$1,300,000.00	$178,296	0.45	723.02	11,592.39	12,315.41	$73,892	$25,862.35	13.37	582,179.40	$11,643,588	$278,051	$11,921,639
WA3011	3.51	$1,300,000.00	$4,567,518	16.19	26,112.35	13,830.85	39,943.20	$239,659	$83,880.71	15.42	671,852.02	$13,437,040	$4,891,057	$18,328,098
WA3012	3.00	$1,300,000.00	$3,902,600	13.74	22,161.15	10,248.48	32,409.63	$194,458	$68,060.22	12.09	526,840.78	$10,536,816	$4,165,118	$14,701,934
WA3013	0.08	$1,300,000.00	$106,951	0.24	381.79	5,115.61	5,497.39	$32,984	$11,544.53	5.35	232,871.76	$4,657,435	$151,480	$4,808,915
WA3014	15.71	$1,300,000.00	$20,417,137	75.55	121,893.07	49,217.56	171,110.63	$1,026,664	$359,332.33	54.71	2,383,211.16	$47,664,223	$21,803,133	$69,467,356
WA3015	0.02	$1,300,000.00	$26,859	0.03	55.56	10,432.49	10,488.05	$62,928	$22,024.91	12.54	546,050.74	$10,921,015	$111,813	$11,032,827
WA3016	1.16	$1,300,000.00	$1,512,963	5.04	8,136.30	16,781.18	24,917.48	$149,505	$52,326.70	19.42	846,135.58	$16,922,712	$1,714,795	$18,637,506
WA3017	11.07	$1,300,000.00	$14,385,319	52.84	85,247.61	40,697.66	125,945.28	$755,672	$264,485.08	35.43	1,543,156.56	$30,863,131	$15,405,476	$46,268,607
WA3018	0.52	$1,300,000.00	$680,962	2.11	3,409.56	16,764.29	20,173.85	$121,043	$42,365.08	8.18	356,477.62	$7,129,552	$844,370	$7,973,923
WA3019	3.81	$1,300,000.00	$4,950,210	17.60	28,392.41	26,434.50	54,826.90	$328,961	$115,136.50	28.84	1,256,052.60	$25,121,052	$5,394,308	$30,515,360
WA3020	0.28	$1,300,000.00	$360,034	1.03	1,660.22	5,906.35	7,566.57	$45,399	$15,889.79	7.05	307,141.56	$6,142,831	$421,323	$6,564,154
WA3021	34.92	$1,300,000.00	$45,397,833	170.15	274,507.85	41,256.27	315,764.13	$1,894,585	$663,104.66	24.24	1,056,094.78	$21,121,896	$47,955,522	$69,077,418
WA3022	0.13	$1,300,000.00	$174,396	0.44	703.78	8,470.48	9,174.26	$55,046	$19,265.94	10.14	441,829.08	$8,836,582	$248,708	$9,085,289
WA3023	32.94	$1,300,000.00	$42,820,258	160.37	258,723.00	31,963.76	290,686.76	$1,744,121	$610,442.21	31.61	1,376,739.94	$27,534,799	$45,174,821	$72,709,620
WA3024	0.10	$1,300,000.00	$123,620	0.28	459.11	2,159.28	2,618.39	$15,710	$5,498.62	2.59	112,994.64	$2,259,893	$144,829	$2,404,721
WA3025	1.87	$1,300,000.00	$2,425,743	8.34	13,447.31	10,102.97	23,550.29	$141,302	$49,455.60	10.18	443,440.80	$8,868,816	$2,616,500	$11,485,316
WA3026	1.24	$1,300,000.00	$1,606,339	5.38	8,675.42	1,191.90	9,867.32	$59,204	$20,721.37	3.08	134,164.80	$2,683,296	$1,686,264	$4,369,560
WA3027	0.25	$1,300,000.00	$328,331	0.93	1,492.51	3,660.70	5,153.21	$30,919	$10,821.75	4.20	183,152.38	$3,663,048	$370,072	$4,033,119
WA3028	4.33	$1,300,000.00	$5,631,878	20.12	32,462.38	12,621.01	45,083.38	$270,500	$94,675.11	12.90	561,793.32	$11,235,866	$5,997,054	$17,232,920
WA3029	0.20	$1,300,000.00	$260,000	0.70	1,129.33	1,150.17	2,279.50	$13,677	$4,786.95	1.38	59,938.56	$1,198,771	$278,464	$1,477,235
WA3030	21.14	$1,300,000.00	$27,483,846	102.25	164,962.04	29,527.42	194,489.46	$1,166,937	$408,427.86	34.44	1,499,988.60	$29,999,772	$29,059,211	$59,058,983
*Excavation cost = $6 per cu yd **LID Construction Cost = $20 per sq ft ***Soft Costs = 35%											Total Project Cost =			$568,063,590

Pragmatic - Pragmatic Ponds plus LID 40% - LULC: Future											
Basin	Pond Footprint(ac)	Land Cost per Acre	Land Cost	Det. Pond Volume (ac-ft)	Pond Volume (cu yds)	Excavation cost*	Soft Costs***	LID Pond Area (ac)	LID Pond Area (sq ft)	LID Ponds Construction cost**	Total Construction cost
WA3001	0	0	0	0	0	0		11.5303	502,260	$10,045,197	$10,045,197
WA3002	0	0	0	0	0	0		1.2523	54,550	$1,091,004	$1,091,004
WA3003	0	0	0	0	0	0		7.2878	317,457	$6,349,131	$6,349,131
WA3004	0	0	0	0	0	0		4.213	183,518	$3,670,366	$3,670,366
WA3005	0	0	0	0	0	0		1.1888	51,784	$1,035,683	$1,035,683
WA3006	0	0	0	0	0	0		2.357	102,671	$2,053,418	$2,053,418
WA3007	0	0	0	0	0	0		4.4203	192,548	$3,850,965	$3,850,965
WA3008	0	0	0	0	0	0		3.5565	154,921	$3,098,423	$3,098,423
WA3009	0	0	0	0	0	0		2.4703	107,606	$2,152,125	$2,152,125
WA3010	0	0	0	0	0	0		6.6825	291,090	$5,821,794	$5,821,794
WA3011	0	0	0	0	0	0		7.7118	335,926	$6,718,520	$6,718,520
WA3012	0	0	0	0	0	0		6.0473	263,420	$5,268,408	$5,268,408
WA3013	0	0	0	0	0	0		2.673	116,436	$2,328,718	$2,328,718
WA3014	0	0	0	0	0	0		27.3555	1,191,606	$23,832,112	$23,832,112
WA3015	0	0	0	0	0	0		6.2678	273,025	$5,460,507	$5,460,507
WA3016	0	0	0	15.8	25,491	$152,944	$53,530	9.7123	423,068	$8,461,356	$8,667,830
WA3017	0	0	0	8.7	14,036	$84,216	$29,476	17.713	771,578	$15,431,566	$15,545,257
WA3018	0	0	0	0	0	0		4.0918	178,239	$3,564,776	$3,564,776
WA3019	0	0	0	0	0	0		14.4175	628,026	$12,560,526	$12,560,526
WA3020	0	0	0	0	0	0		3.5255	153,571	$3,071,416	$3,071,416
WA3021	0	0	0	33.1	53,401	$320,408	$112,143	12.1223	528,047	$10,560,948	$10,993,499
WA3022	0	0	0	0	0	0		5.0715	220,915	$4,418,291	$4,418,291
WA3023	0	0	0	0	0	0		15.8028	688,370	$13,767,399	$13,767,399
WA3024	0	0	0	0	0	0		1.297	56,497	$1,129,946	$1,129,946
WA3025	0	0	0	0	0	0		5.09	221,720	$4,434,408	$4,434,408
WA3026	0	0	0	0	0	0		1.54	67,082	$1,341,648	$1,341,648
WA3027	0	0	0	0	0	0		2.1023	91,576	$1,831,524	$1,831,524
WA3028	0	0	0	0	0	0		6.4485	280,897	$5,617,933	$5,617,933
WA3029	0	0	0	0	0	0		0.688	29,969	$599,386	$599,386
WA3030	0	0	0	0	0	0		17.2175	749,994	$14,999,886	$14,999,886
									Total Project Cost =		$185,320,096

*Excavation cost = $6 per cu yd **LID Construction Cost = $20 per sq ft ***Soft Costs = 35%

LID80 - LID with 80% Capture - LULC: 2002						
Basin	**POND FOOTPRINT (AC)**	**Land Cost /Acre**	**Land Cost**	**LID Pond Area (ac)**	**LID Pond Area (sq ft)**	**LID Ponds Construction cost****
WA3001	0	0	0	23.0606	1,004,519.74	$20,090,395
WA3002	0	0	0	2.5046	109,100.38	$2,182,008
WA3003	0	0	0	14.5756	634,913.14	$12,698,263
WA3004	0	0	0	8.426	367,036.56	$7,340,731
WA3005	0	0	0	2.3776	103,568.26	$2,071,365
WA3006	0	0	0	4.714	205,341.84	$4,106,837
WA3007	0	0	0	8.8406	385,096.54	$7,701,931
WA3008	0	0	0	7.113	309,842.28	$6,196,846
WA3009	0	0	0	4.9406	215,212.54	$4,304,251
WA3010	0	0	0	13.365	582,179.40	$11,643,588
WA3011	0	0	0	15.4236	671,852.02	$13,437,040
WA3012	0	0	0	12.0946	526,840.78	$10,536,816
WA3013	0	0	0	5.346	232,871.76	$4,657,435
WA3014	0	0	0	54.711	2,383,211.16	$47,664,223
WA3015	0	0	0	12.5356	546,050.74	$10,921,015
WA3016	0	0	0	19.4246	846,135.58	$16,922,712
WA3017	0	0	0	35.426	1,543,156.56	$30,863,131
WA3018	0	0	0	8.1836	356,477.62	$7,129,552
WA3019	0	0	0	28.835	1,256,052.60	$25,121,052
WA3020	0	0	0	7.051	307,141.56	$6,142,831
WA3021	0	0	0	24.2446	1,056,094.78	$21,121,896
WA3022	0	0	0	10.143	441,829.08	$8,836,582
WA3023	0	0	0	31.6056	1,376,739.94	$27,534,799
WA3024	0	0	0	2.594	112,994.64	$2,259,893
WA3025	0	0	0	10.18	443,440.80	$8,868,816
WA3026	0	0	0	3.08	134,164.80	$2,683,296
WA3027	0	0	0	4.2046	183,152.38	$3,663,048
WA3028	0	0	0	12.897	561,793.32	$11,235,866
WA3029	0	0	0	1.376	59,938.56	$1,198,771
WA3030	0	0	0	34.435	1,499,988.60	$29,999,772
****LID Construction Cost = $20 per sq ft**				**Total Project Cost =**		$369,134,758

LVL2Wet - Level II with Wet Pond - LULC: Future										
Basin	POND FOOTPRINT (AC)	Land Cost /Acre	Land Cost	Level II - Det. Pond Volume (ac-ft)	Det. Pond Volume (cu yds)	Wet Pond Volume (cu yds)	Total Pond Volume (cu yds)	Excavation cost*	Soft Costs***	Total Construction cost
WA3001	4.224	$1,300,000.00	$5,490,577.30	26.76	43,179.56	20,975.24	64,154.79	$384,928.77	$134,725.07	$6,010,231
WA3002	0.425	$1,300,000.00	$552,581.90	2.15	3,471.15	2,109.56	5,580.71	$33,484.26	$11,719.49	$597,786
WA3003	3.769	$1,300,000.00	$4,900,142.00	23.75	38,308.62	14,456.13	52,764.74	$316,588.45	$110,805.96	$5,327,536
WA3004	1.485	$1,300,000.00	$1,930,992.70	8.78	14,157.54	7,132.02	21,289.56	$127,737.36	$44,708.08	$2,103,438
WA3005	0.724	$1,300,000.00	$940,644.90	3.96	6,390.85	1,992.20	8,383.05	$50,298.29	$17,604.40	$1,008,548
WA3006	1.440	$1,300,000.00	$1,872,319.80	8.49	13,690.25	4,717.69	18,407.94	$110,447.66	$38,656.68	$2,021,424
WA3007	2.147	$1,300,000.00	$2,791,465.30	13.06	21,072.20	8,171.71	29,243.90	$175,463.42	$61,412.20	$3,028,341
WA3008	1.291	$1,300,000.00	$1,678,779.70	7.53	12,153.74	6,520.81	18,674.55	$112,047.28	$39,216.55	$1,830,044
WA3009	0.869	$1,300,000.00	$1,129,733.80	4.86	7,848.08	4,220.28	12,068.36	$72,410.13	$25,343.55	$1,227,487
WA3010	2.994	$1,300,000.00	$3,892,574.40	18.62	30,035.39	11,592.39	41,627.78	$249,766.69	$87,418.34	$4,229,759
WA3011	4.857	$1,300,000.00	$6,313,457.80	30.99	49,990.31	13,830.85	63,821.16	$382,926.96	$134,024.43	$6,830,409
WA3012	3.928	$1,300,000.00	$5,106,884.90	18.18	29,326.91	10,248.48	39,575.39	$237,452.36	$83,108.33	$5,427,446
WA3013	2.823	$1,300,000.00	$3,669,268.20	12.88	20,777.55	5,115.61	25,893.16	$155,358.96	$54,375.64	$3,879,003
WA3014	24.251	$1,300,000.00	$31,525,949.00	117.55	189,642.33	49,217.56	238,859.89	$1,433,159.33	$501,605.76	$33,460,714
WA3015	2.598	$1,300,000.00	$3,377,429.90	16.01	25,829.69	10,432.49	36,262.19	$217,573.11	$76,150.59	$3,671,154
WA3016	3.376	$1,300,000.00	$4,388,822.10	21.14	34,103.24	16,781.18	50,884.41	$305,306.47	$106,857.26	$4,800,986
WA3017	10.529	$1,300,000.00	$13,687,349.00	69.22	111,682.74	40,697.66	152,380.40	$914,282.42	$319,998.85	$14,921,630
WA3018	3.945	$1,300,000.00	$5,128,636.50	24.91	40,191.89	16,764.29	56,956.18	$341,737.08	$119,607.98	$5,589,982
WA3019	6.533	$1,300,000.00	$8,492,649.10	42.22	68,120.93	26,434.50	94,555.43	$567,332.58	$198,566.40	$9,258,548
WA3020	1.256	$1,300,000.00	$1,632,853.30	7.31	11,790.34	5,906.35	17,696.69	$106,180.13	$37,163.05	$1,776,196
WA3021	18.458	$1,300,000.00	$23,995,244.00	89.06	143,685.88	41,256.27	184,942.15	$1,109,652.93	$388,378.52	$25,493,275
WA3022	2.373	$1,300,000.00	$3,085,336.80	83.62	134,902.88	8,470.48	143,373.36	$860,240.16	$301,084.06	$4,246,661
WA3023	16.013	$1,300,000.00	$20,816,549.00	73.22	118,133.62	31,963.76	150,097.38	$900,584.30	$315,204.51	$22,032,338
WA3024	0.892	$1,300,000.00	$1,159,057.90	3.52	5,675.57	2,159.28	7,834.85	$47,009.12	$16,453.19	$1,222,520
WA3025	5.292	$1,300,000.00	$6,879,694.90	23.11	37,288.28	10,102.97	47,391.25	$284,347.50	$99,521.63	$7,263,564
WA3026	1.033	$1,300,000.00	$1,343,184.70	5.90	9,510.88	1,191.90	10,702.78	$64,216.67	$22,475.84	$1,429,877
WA3027	1.321	$1,300,000.00	$1,717,708.20	7.72	12,462.15	3,660.70	16,122.85	$96,737.09	$33,857.98	$1,848,303
WA3028	6.222	$1,300,000.00	$8,088,265.90	26.95	43,478.59	12,621.01	56,099.60	$336,597.57	$117,809.15	$8,542,673
WA3029	0.632	$1,300,000.00	$821,897.70	2.45	3,951.44	1,150.17	5,101.61	$30,609.67	$10,713.38	$863,221
WA3030	15.258	$1,300,000.00	$19,835,296.00	69.05	111,403.86	29,527.42	140,931.28	$845,587.67	$295,955.68	$20,976,839
						419,420.93				
							Total Project Cost =			$210,919,934

*Excavation cost = $6 per cu yd ***Soft Costs = 35%

Cistern - Cistern for Roofs - LULC:Future							
Basin	Cistern Volume (ac/ft)	Precipitation Factor (ft)	Basin Roof Area (ac)	Basin Roof Area (sqft)	Number of Cisterns*	Installation cost per home**	Total Construction Cost for Cisterns
WA3001	92.57	2.5	37.03	1,612,940	1613	$4,000.00	$6,451,759
WA3002	9.16	2.5	3.66	159,560	160	$4,000.00	$638,241
WA3003	58.28	2.5	23.31	1,015,427	1015	$4,000.00	$4,061,709
WA3004	34.10	2.5	13.64	594,202	594	$4,000.00	$2,376,808
WA3005	10.08	2.5	4.03	175,590	176	$4,000.00	$702,361
WA3006	16.14	2.5	6.46	281,267	281	$4,000.00	$1,125,068
WA3007	33.13	2.5	13.25	577,170	577	$4,000.00	$2,308,680
WA3008	24.98	2.5	9.99	435,252	435	$4,000.00	$1,741,006
WA3009	17.46	2.5	6.98	304,179	304	$4,000.00	$1,216,718
WA3010	59.19	2.5	23.68	1,031,370	1031	$4,000.00	$4,125,480
WA3011	66.71	2.5	26.68	1,162,268	1162	$4,000.00	$4,649,072
WA3012	55.16	2.5	22.06	961,064	961	$4,000.00	$3,844,257
WA3013	21.44	2.5	8.57	373,483	373	$4,000.00	$1,493,934
WA3014	241.43	2.5	96.57	4,206,633	4207	$4,000.00	$16,826,531
WA3015	794.84	2.5	317.94	13,849,299	13849	$4,000.00	$55,397,197
WA3016	79.76	2.5	31.91	1,389,782	1390	$4,000.00	$5,559,127
WA3017	165.27	2.5	66.11	2,879,708	2880	$4,000.00	$11,518,832
WA3018	69.91	2.5	27.96	1,218,025	1218	$4,000.00	$4,872,099
WA3019	101.58	2.5	40.63	1,769,930	1770	$4,000.00	$7,079,720
WA3020	18.99	2.5	7.60	330,882	331	$4,000.00	$1,323,527
WA3021	187.19	2.5	74.88	3,261,555	3262	$4,000.00	$13,046,220
WA3022	28.68	2.5	11.47	499,677	500	$4,000.00	$1,998,707
WA3023	128.32	2.5	51.33	2,235,804	2236	$4,000.00	$8,943,216
WA3024	12.68	2.5	5.07	220,849	221	$4,000.00	$883,397
WA3025	40.99	2.5	16.39	714,123	714	$4,000.00	$2,856,491
WA3026	14.21	2.5	5.69	247,639	248	$4,000.00	$990,554
WA3027	9.61	2.5	3.84	167,401	167	$4,000.00	$669,604
WA3028	50.44	2.5	20.18	878,867	879	$4,000.00	$3,515,466
WA3029	6.85	2.5	2.74	119,398	119	$4,000.00	$477,592
WA3030	100.68	2.5	40.27	1,754,292	1754	$4,000.00	$7,017,168
					Total Project Cost =		$177,710,540

***Basin Roof Area was divided by 1000 sqft to obtain the equivalent number of homes that would need to have cisterns installed.**

****Cost per home = $4 per sqft of tributary area * 1000 sqft per home .**

Soft costs were not included for the cisterns based on the assumption that the cisterns would be above ground and would be sized by the method shown in the 2009 KCSWM, Appendix C, Section C.2.7.2.

Level2 - Level II with Dry Pond - LULC: Future				
Basin	Total Cost for Detention	Pond Maintenance Cost (Annual)	Pond Maint. Cost for 40 yr at PV	Total Present Value
WA3001	$5,840,331.73	$4,663	$92,301	$5,932,633
WA3002	$580,698.21	$375	$7,420	$588,118
WA3003	$5,210,441.78	$4,137	$81,889	$5,292,331
WA3004	$2,045,668.80	$1,529	$30,263	$2,075,932
WA3005	$992,410.75	$690	$13,661	$1,006,072
WA3006	$1,983,210.85	$1,479	$29,265	$2,012,475
WA3007	$2,962,150.10	$2,276	$45,044	$3,007,194
WA3008	$1,777,224.97	$1,313	$25,980	$1,803,205
WA3009	$1,193,303.22	$848	$16,776	$1,210,079
WA3010	$4,135,861.09	$3,244	$64,204	$4,200,065
WA3011	$6,718,379.31	$5,399	$106,860	$6,825,240
WA3012	$5,344,432.91	$3,167	$62,690	$5,407,123
WA3013	$3,837,566.39	$2,244	$44,414	$3,881,981
WA3014	$33,062,051.86	$20,481	$405,383	$33,467,435
WA3015	$3,586,650.41	$2,790	$55,214	$3,641,864
WA3016	$4,665,058.31	$3,683	$72,900	$4,737,958
WA3017	$14,591,979.19	$12,062	$238,735	$14,830,714
WA3018	$5,454,190.82	$4,341	$85,915	$5,540,106
WA3019	$9,044,428.66	$7,357	$145,617	$9,190,045
WA3020	$1,728,355.08	$1,273	$25,203	$1,753,558
WA3021	$25,159,099.66	$15,518	$307,146	$25,466,245
WA3022	$4,178,050.14	$14,570	$288,371	$4,466,421
WA3023	$21,773,431.33	$12,758	$252,525	$22,025,956
WA3024	$1,205,030.03	$613	$12,132	$1,217,162
WA3025	$7,181,729.96	$4,027	$79,708	$7,261,438
WA3026	$1,420,222.85	$1,027	$20,331	$1,440,553
WA3027	$1,818,651.61	$1,346	$26,639	$1,845,291
WA3028	$8,440,442.48	$4,696	$92,941	$8,533,383
WA3029	$853,904.39	$427	$8,447	$862,351
WA3030	$20,737,667.26	$12,032	$238,139	$20,975,806
		Total Present Value Cost =		$210,498,738

LID40 - LID with 40% TIA Capture - LULC:Future				
Basin	**Total Cost for LID Ponds**	**LID Pond Maintenance Cost (Annual)**	**Pond Maint. Cost for 40 yr at PV**	**Total Present Value**
WA3001	$10,045,197	$1,115,017	$22,069,273	$32,114,471
WA3002	$1,091,004	$121,101	$2,396,933	$3,487,936
WA3003	$6,349,131	$704,754	$13,949,026	$20,298,157
WA3004	$3,670,366	$407,411	$8,063,784	$11,734,150
WA3005	$1,035,683	$114,961	$2,275,392	$3,311,075
WA3006	$2,053,418	$227,929	$4,511,355	$6,564,773
WA3007	$3,850,965	$427,457	$8,460,561	$12,311,527
WA3008	$3,098,423	$343,925	$6,807,227	$9,905,650
WA3009	$2,152,125	$238,886	$4,728,214	$6,880,339
WA3010	$5,821,794	$646,219	$12,790,467	$18,612,261
WA3011	$6,718,520	$745,756	$14,760,572	$21,479,092
WA3012	$5,268,408	$584,793	$11,574,679	$16,843,086
WA3013	$2,328,718	$258,488	$5,116,187	$7,444,904
WA3014	$23,832,112	$2,645,364	$52,359,089	$76,191,200
WA3015	$5,460,507	$606,116	$11,996,721	$17,457,228
WA3016	$8,461,356	$939,210	$18,589,577	$27,050,933
WA3017	$15,431,566	$1,712,904	$33,903,111	$49,334,676
WA3018	$3,564,776	$395,690	$7,831,804	$11,396,580
WA3019	$12,560,526	$1,394,218	$27,595,444	$40,155,970
WA3020	$3,071,416	$340,927	$6,747,892	$9,819,308
WA3021	$10,560,948	$1,172,265	$23,202,376	$33,763,323
WA3022	$4,418,291	$490,430	$9,706,974	$14,125,265
WA3023	$13,767,399	$1,528,181	$30,246,942	$44,014,341
WA3024	$1,129,946	$125,424	$2,482,489	$3,612,436
WA3025	$4,434,408	$492,219	$9,742,383	$14,176,791
WA3026	$1,341,648	$148,923	$2,947,597	$4,289,245
WA3027	$1,831,524	$203,299	$4,023,853	$5,855,377
WA3028	$5,617,933	$623,591	$12,342,585	$17,960,518
WA3029	$599,386	$66,532	$1,316,849	$1,916,234
WA3030	$14,999,886	$1,664,987	$32,954,712	$47,954,598
		Total Present Value Cost =		$590,061,444

ECY08 - Proposed Regulatory 8% - LULC: Future				
Basin	**Total Construction Cost**	**Facilities Maintenance Cost (Annual)**	**Facilities Maint. Cost for 40 yr at PV**	**Total Present Value**
WA3001	$21,759,128	$2,233,425	$44,205,667	$65,964,795
WA3002	$2,342,547	$242,451	$4,798,768	$7,141,315
WA3003	$22,362,038	$1,413,645	$27,979,952	$50,341,990
WA3004	$8,674,229	$816,195	$16,154,768	$24,828,997
WA3005	$2,352,645	$230,334	$4,558,941	$6,911,586
WA3006	$4,331,696	$456,374	$9,032,914	$13,364,610
WA3007	$23,477,338	$856,655	$16,955,573	$40,432,911
WA3008	$6,304,714	$688,571	$13,628,726	$19,933,440
WA3009	$4,701,164	$478,247	$9,465,838	$14,167,002
WA3010	$12,100,228	$1,293,768	$25,607,259	$37,707,487
WA3011	$58,421,243	$1,495,825	$29,606,527	$88,027,770
WA3012	$14,701,934	$1,171,661	$23,190,418	$37,892,351
WA3013	$5,196,962	$517,569	$10,244,125	$15,441,087
WA3014	$86,318,128	$5,309,209	$105,083,947	$191,402,075
WA3015	$11,032,825	$1,213,365	$24,015,861	$35,048,687
WA3016	$17,148,816	$1,881,112	$37,232,418	$54,381,234
WA3017	$52,231,024	$3,439,410	$68,075,444	$120,306,469
WA3018	$8,815,201	$793,559	$15,706,732	$24,521,934
WA3019	$27,016,204	$2,794,358	$55,308,087	$82,324,291
WA3020	$6,564,154	$682,518	$13,508,924	$20,073,078
WA3021	$69,077,418	$2,378,633	$47,079,734	$116,157,152
WA3022	$9,461,646	$981,851	$19,433,558	$28,895,205
WA3023	$32,247,833	$3,087,757	$61,115,261	$93,363,094
WA3024	$2,404,721	$251,093	$4,969,835	$7,374,557
WA3025	$11,485,316	$986,663	$19,528,792	$31,014,108
WA3026	$4,369,560	$298,645	$5,911,005	$10,280,565
WA3027	$4,033,119	$407,028	$8,056,210	$12,089,329
WA3028	$17,232,920	$1,250,281	$24,746,527	$41,979,447
WA3029	$1,477,235	$133,310	$2,638,572	$4,115,807
WA3030	$59,058,983	$3,338,937	$66,086,815	$125,145,798
		Total Present Value Cost =		$1,420,628,169

Pragmatic - Pragmatic Ponds plus LID 40% - LULC: Future				
Basin	**Total Cost for Construction**	**Pond Maintenance Cost (Annual)**	**Pond Maint. Cost for 40 yr at PV**	**Total Present Value**
WA3001	$10,045,197	$1,115,017	$22,069,273	$32,114,471
WA3002	$1,091,004	$121,101	$2,396,933	$3,487,936
WA3003	$6,349,131	$704,754	$13,949,026	$20,298,157
WA3004	$3,670,366	$407,411	$8,063,784	$11,734,150
WA3005	$1,035,683	$114,961	$2,275,392	$3,311,075
WA3006	$2,053,418	$227,929	$4,511,355	$6,564,773
WA3007	$3,850,965	$427,457	$8,460,561	$12,311,527
WA3008	$3,098,423	$343,925	$6,807,227	$9,905,650
WA3009	$2,152,125	$238,886	$4,728,214	$6,880,339
WA3010	$5,821,794	$646,219	$12,790,467	$18,612,261
WA3011	$6,718,520	$745,756	$14,760,572	$21,479,092
WA3012	$5,268,408	$584,793	$11,574,679	$16,843,086
WA3013	$2,328,718	$258,488	$5,116,187	$7,444,904
WA3014	$23,832,112	$2,645,364	$52,359,089	$76,191,200
WA3015	$5,460,507	$606,116	$11,996,721	$17,457,228
WA3016	$8,667,830	$941,964	$18,644,067	$27,311,897
WA3017	$15,545,257	$1,714,420	$33,933,114	$49,478,371
WA3018	$3,564,776	$395,690	$7,831,804	$11,396,580
WA3019	$12,560,526	$1,394,218	$27,595,444	$40,155,970
WA3020	$3,071,416	$340,927	$6,747,892	$9,819,308
WA3021	$10,993,499	$1,178,033	$23,316,527	$34,310,025
WA3022	$4,418,291	$490,430	$9,706,974	$14,125,265
WA3023	$13,767,399	$1,528,181	$30,246,942	$44,014,341
WA3024	$1,129,946	$125,424	$2,482,489	$3,612,436
WA3025	$4,434,408	$492,219	$9,742,383	$14,176,791
WA3026	$1,341,648	$148,923	$2,947,597	$4,289,245
WA3027	$1,831,524	$203,299	$4,023,853	$5,855,377
WA3028	$5,617,933	$623,591	$12,342,585	$17,960,518
WA3029	$599,386	$66,532	$1,316,849	$1,916,234
WA3030	$14,999,886	$1,664,987	$32,954,712	$47,954,598
		Total Present Value Cost =		$591,012,805

LID80 - LID with 80% Capture - LULC: Future				
Basin	Total Cost for Construction	Pond Maintenance Cost (Annual)	Pond Maint. Cost for 40 yr at PV	Total Present Value
WA3001	$20,090,395	$2,230,034	$44,138,546	$64,228,941
WA3002	$2,182,008	$242,203	$4,793,865	$6,975,873
WA3003	$12,698,263	$1,409,507	$27,898,051	$40,596,314
WA3004	$7,340,731	$814,821	$16,127,568	$23,468,299
WA3005	$2,071,365	$229,922	$4,550,784	$6,622,149
WA3006	$4,106,837	$455,859	$9,022,710	$13,129,547
WA3007	$7,701,931	$854,914	$16,921,122	$24,623,053
WA3008	$6,196,846	$687,850	$13,614,454	$19,811,300
WA3009	$4,304,251	$477,772	$9,456,428	$13,760,679
WA3010	$11,643,588	$1,292,438	$25,580,933	$37,224,521
WA3011	$13,437,040	$1,491,511	$29,521,144	$42,958,184
WA3012	$10,536,816	$1,169,587	$23,149,357	$33,686,173
WA3013	$4,657,435	$516,975	$10,232,373	$14,889,809
WA3014	$47,664,223	$5,290,729	$104,718,178	$152,382,401
WA3015	$10,921,015	$1,212,233	$23,993,442	$34,914,456
WA3016	$16,922,712	$1,878,421	$37,179,154	$54,101,866
WA3017	$30,863,131	$3,425,808	$67,806,221	$98,669,352
WA3018	$7,129,552	$791,380	$15,663,608	$22,793,161
WA3019	$25,121,052	$2,788,437	$55,190,888	$80,311,940
WA3020	$6,142,831	$681,854	$13,495,785	$19,638,616
WA3021	$21,121,896	$2,344,530	$46,404,751	$67,526,647
WA3022	$8,836,582	$980,861	$19,413,947	$28,250,529
WA3023	$27,534,799	$3,056,363	$60,493,883	$88,028,682
WA3024	$2,259,893	$250,848	$4,964,979	$7,224,872
WA3025	$8,868,816	$984,439	$19,484,766	$28,353,582
WA3026	$2,683,296	$297,846	$5,895,195	$8,578,491
WA3027	$3,663,048	$406,598	$8,047,706	$11,710,754
WA3028	$11,235,866	$1,247,181	$24,685,170	$35,921,036
WA3029	$1,198,771	$133,064	$2,633,697	$3,832,468
WA3030	$29,999,772	$3,329,975	$65,909,423	$95,909,195
		Total Present Value Cost =		$1,180,122,888

LVL2Wet - Level II with Wet Pond - LULC: Future				
Basin	**Total Cost for Construction**	**Pond Maintenance Cost (Annual)**	**Pond Maint. Cost for 40 yr at PV**	**Total Present Value**
WA3001	$6,010,231	$6,929	$137,139	$6,147,370
WA3002	$597,786	$603	$11,929	$609,715
WA3003	$5,327,536	$5,699	$112,791	$5,440,327
WA3004	$2,103,438	$2,299	$45,509	$2,148,947
WA3005	$1,008,548	$905	$17,920	$1,026,467
WA3006	$2,021,424	$1,988	$39,349	$2,060,773
WA3007	$3,028,341	$3,158	$62,512	$3,090,853
WA3008	$1,830,044	$2,017	$39,919	$1,869,963
WA3009	$1,227,487	$1,303	$25,798	$1,253,285
WA3010	$4,229,759	$4,496	$88,984	$4,318,744
WA3011	$6,830,409	$6,893	$136,425	$6,966,835
WA3012	$5,427,446	$4,274	$84,597	$5,512,043
WA3013	$3,879,003	$2,796	$55,350	$3,934,353
WA3014	$33,460,714	$25,797	$510,591	$33,971,306
WA3015	$3,671,154	$3,916	$77,515	$3,748,668
WA3016	$4,800,986	$5,496	$108,771	$4,909,757
WA3017	$14,921,630	$16,457	$325,731	$15,247,362
WA3018	$5,589,982	$6,151	$121,751	$5,711,732
WA3019	$9,258,548	$10,212	$202,123	$9,460,672
WA3020	$1,776,196	$1,911	$37,829	$1,814,025
WA3021	$25,493,275	$19,974	$395,336	$25,888,611
WA3022	$4,246,661	$15,484	$306,478	$4,553,139
WA3023	$22,032,338	$16,211	$320,851	$22,353,189
WA3024	$1,222,520	$846	$16,748	$1,239,268
WA3025	$7,263,564	$5,118	$101,304	$7,364,868
WA3026	$1,429,877	$1,156	$22,878	$1,452,756
WA3027	$1,848,303	$1,741	$34,465	$1,882,768
WA3028	$8,542,673	$6,059	$119,920	$8,662,592
WA3029	$863,221	$551	$10,905	$874,126
WA3030	$20,976,839	$15,221	$301,257	$21,278,097
		Total Present Value Cost =		$214,792,610

Cistern - Cistern for Roofs - LULC:Future				
Basin	Total Cost for Construction	Annual Inspection Cost per basin	Inspection Cost for 40 yr at PV	Total Present Value
WA3001	$6,451,759	$145,745	$2,884,702	$9,336,461
WA3002	$638,241	$14,418	$285,370	$923,611
WA3003	$4,061,709	$91,754	$1,816,066	$5,877,774
WA3004	$2,376,808	$53,692	$1,062,715	$3,439,523
WA3005	$702,361	$15,866	$314,039	$1,016,400
WA3006	$1,125,068	$25,415	$503,039	$1,628,106
WA3007	$2,308,680	$52,153	$1,032,254	$3,340,934
WA3008	$1,741,006	$39,329	$778,436	$2,519,442
WA3009	$1,216,718	$27,486	$544,017	$1,760,735
WA3010	$4,125,480	$93,195	$1,844,579	$5,970,060
WA3011	$4,649,072	$105,023	$2,078,687	$6,727,758
WA3012	$3,844,257	$86,842	$1,718,839	$5,563,096
WA3013	$1,493,934	$33,748	$667,966	$2,161,899
WA3014	$16,826,531	$380,111	$7,523,456	$24,349,987
WA3015	$55,397,197	$1,251,423	$24,769,121	$80,166,318
WA3016	$5,559,127	$125,581	$2,485,590	$8,044,717
WA3017	$11,518,832	$260,210	$5,150,285	$16,669,117
WA3018	$4,872,099	$110,061	$2,178,406	$7,050,505
WA3019	$7,079,720	$159,931	$3,165,475	$10,245,195
WA3020	$1,323,527	$29,898	$591,774	$1,915,301
WA3021	$13,046,220	$294,714	$5,833,209	$18,879,429
WA3022	$1,998,707	$45,151	$893,659	$2,892,366
WA3023	$8,943,216	$202,027	$3,998,679	$12,941,896
WA3024	$883,397	$19,956	$394,983	$1,278,380
WA3025	$2,856,491	$64,528	$1,277,190	$4,133,681
WA3026	$990,554	$22,377	$442,895	$1,433,450
WA3027	$669,604	$15,126	$299,393	$968,997
WA3028	$3,515,466	$79,414	$1,571,831	$5,087,297
WA3029	$477,592	$10,789	$213,540	$691,132
WA3030	$7,017,168	$158,518	$3,137,507	$10,154,674
		Total Present Value Cost =		$257,168,241

Level2 - Level II with Dry Pond - LULC: Future				
Basin	**Total Cost for Detention**	**Pond Maintenance Cost (Annual)**	**Pond Maint. Cost for 40 yr at PV**	**Total Present Value**
WA3001	$5,840,331.73	$4,663	$92,301	$5,932,633
WA3002	$580,698.21	$375	$7,420	$588,118
WA3003	$5,210,441.78	$4,137	$81,889	$5,292,331
WA3004	$2,045,668.80	$1,529	$30,263	$2,075,932
WA3005	$992,410.75	$690	$13,661	$1,006,072
WA3006	$1,983,210.85	$1,479	$29,265	$2,012,475
WA3007	$2,962,150.10	$2,276	$45,044	$3,007,194
WA3008	$1,777,224.97	$1,313	$25,980	$1,803,205
WA3009	$1,193,303.22	$848	$16,776	$1,210,079
WA3010	$4,135,861.09	$3,244	$64,204	$4,200,065
WA3011	$6,718,379.31	$5,399	$106,860	$6,825,240
WA3012	$5,344,432.91	$3,167	$62,690	$5,407,123
WA3013	$3,837,566.39	$2,244	$44,414	$3,881,981
WA3014	$33,062,051.86	$20,481	$405,383	$33,467,435
WA3015	$3,586,650.41	$2,790	$55,214	$3,641,864
WA3016	$4,665,058.31	$3,683	$72,900	$4,737,958
WA3017	$14,591,979.19	$12,062	$238,735	$14,830,714
WA3018	$5,454,190.82	$4,341	$85,915	$5,540,106
WA3019	$9,044,428.66	$7,357	$145,617	$9,190,045
WA3020	$1,728,355.08	$1,273	$25,203	$1,753,558
WA3021	$25,159,099.66	$15,518	$307,146	$25,466,245
WA3022	$4,178,050.14	$14,570	$288,371	$4,466,421
WA3023	$21,773,431.33	$12,758	$252,525	$22,025,956
WA3024	$1,205,030.03	$613	$12,132	$1,217,162
WA3025	$7,181,729.96	$4,027	$79,708	$7,261,438
WA3026	$1,420,222.85	$1,027	$20,331	$1,440,553
WA3027	$1,818,651.61	$1,346	$26,639	$1,845,291
WA3028	$8,440,442.48	$4,696	$92,941	$8,533,383
WA3029	$853,904.39	$427	$8,447	$862,351
WA3030	$20,737,667.26	$12,032	$238,139	$20,975,806
		Total Present Value Cost =		$210,498,738

LID40 - LID with 40% TIA Capture - LULC:Future				
Basin	**Total Cost for LID Ponds**	**LID Pond Maintenance Cost (Annual)**	**Pond Maint. Cost for 40 yr at PV**	**Total Present Value**
WA3001	$10,045,197	$180,813.55	$3,578,801	$13,623,998
WA3002	$1,091,004	$19,638.07	$388,692	$1,479,696
WA3003	$6,349,131	$114,284.36	$2,262,004	$8,611,136
WA3004	$3,670,366	$66,066.58	$1,307,641	$4,978,006
WA3005	$1,035,683	$18,642.29	$368,982	$1,404,665
WA3006	$2,053,418	$36,961.53	$731,571	$2,784,989
WA3007	$3,850,965	$69,317.38	$1,371,983	$5,222,948
WA3008	$3,098,423	$55,771.61	$1,103,875	$4,202,297
WA3009	$2,152,125	$38,738.26	$766,737	$2,918,863
WA3010	$5,821,794	$104,792.29	$2,074,130	$7,895,924
WA3011	$6,718,520	$120,933.36	$2,393,606	$9,112,126
WA3012	$5,268,408	$94,831.34	$1,876,975	$7,145,383
WA3013	$2,328,718	$41,916.92	$829,652	$3,158,369
WA3014	$23,832,112	$428,978.01	$8,490,663	$32,322,775
WA3015	$5,460,507	$98,289.13	$1,945,414	$7,405,922
WA3016	$8,461,356	$152,304.40	$3,014,526	$11,475,882
WA3017	$15,431,566	$277,768.18	$5,497,802	$20,929,367
WA3018	$3,564,776	$64,165.97	$1,270,022	$4,834,798
WA3019	$12,560,526	$226,089.47	$4,474,937	$17,035,463
WA3020	$3,071,416	$55,285.48	$1,094,253	$4,165,668
WA3021	$10,560,948	$190,097.06	$3,762,547	$14,323,495
WA3022	$4,418,291	$79,529.23	$1,574,104	$5,992,395
WA3023	$13,767,399	$247,813.19	$4,904,909	$18,672,309
WA3024	$1,129,946	$20,339.04	$402,566	$1,532,512
WA3025	$4,434,408	$79,819.34	$1,579,846	$6,014,254
WA3026	$1,341,648	$24,149.66	$477,989	$1,819,637
WA3027	$1,831,524	$32,967.43	$652,517	$2,484,040
WA3028	$5,617,933	$101,122.80	$2,001,500	$7,619,433
WA3029	$599,386	$10,788.94	$213,543	$812,929
WA3030	$14,999,886	$269,997.95	$5,344,007	$20,343,893
		Total Present Value Cost =		$250,323,173

ECY08 - Proposed Regulatory 8% - LULC: Future				
Basin	**Total Construction Cost**	**Pond Maintenance Cost (Annual)**	**Pond Maint. Cost for 40 yr at PV**	**Total Present Value**
WA3001	$22,252,722	$365,018	$7,224,723	$29,477,444
WA3002	$2,262,282	$39,524	$782,287	$3,044,568
WA3003	$17,197,036	$232,707	$4,605,909	$21,802,945
WA3004	$8,512,772	$133,507	$2,642,482	$11,155,253
WA3005	$2,492,990	$37,697	$746,122	$3,239,112
WA3006	$4,172,451	$74,439	$1,473,346	$5,645,798
WA3007	$9,312,163	$140,375	$2,778,416	$12,090,579
WA3008	$6,305,569	$112,264	$2,222,021	$8,527,591
WA3009	$4,401,105	$77,952	$1,542,885	$5,943,990
WA3010	$11,921,639	$210,915	$4,174,585	$16,096,224
WA3011	$18,328,098	$246,181	$4,872,596	$23,200,694
WA3012	$12,347,956	$193,163	$3,823,229	$16,171,185
WA3013	$4,808,915	$84,428	$1,671,055	$6,479,970
WA3014	$69,467,356	$876,436	$17,347,095	$86,814,452
WA3015	$11,032,827	$197,711	$3,913,248	$14,946,075
WA3016	$18,637,506	$307,300	$6,082,316	$24,719,823
WA3017	$46,268,607	$569,138	$11,264,826	$57,533,433
WA3018	$7,973,923	$130,511	$2,583,169	$10,557,091
WA3019	$30,515,360	$458,100	$9,067,073	$39,582,433
WA3020	$6,267,694	$111,388	$2,204,680	$8,472,374
WA3021	$69,077,418	$414,297	$8,200,078	$77,277,496
WA3022	$9,085,289	$160,049	$3,167,819	$12,253,108
WA3023	$72,709,620	$527,021	$10,431,196	$83,140,816
WA3024	$2,321,747	$40,961	$810,729	$3,132,475
WA3025	$10,955,331	$162,182	$3,210,033	$14,165,365
WA3026	$3,919,589	$49,365	$977,070	$4,896,659
WA3027	$3,788,040	$66,491	$1,316,049	$5,104,089
WA3028	$14,342,695	$207,115	$4,099,372	$18,442,067
WA3029	$1,476,134	$21,824	$431,957	$1,908,092
WA3030	$39,800,102	$561,001	$11,103,759	$50,903,861
		Total Present Value Cost =		$676,725,062

Pragmatic - Pragmatic Ponds plus LID 40% - LULC: Future				
Basin	Total Cost for Construction	Pond Maintenance Cost (Annual)	Pond Maint. Cost for 40 yr at PV	Total Present Value
WA3001	$10,045,197	$180,814	$3,578,801	$13,623,998
WA3002	$1,091,004	$19,638	$388,692	$1,479,696
WA3003	$6,349,131	$114,284	$2,262,004	$8,611,136
WA3004	$3,670,366	$66,067	$1,307,641	$4,978,006
WA3005	$1,035,683	$18,642	$368,982	$1,404,665
WA3006	$2,053,418	$36,962	$731,571	$2,784,989
WA3007	$3,850,965	$69,317	$1,371,983	$5,222,948
WA3008	$3,098,423	$55,772	$1,103,875	$4,202,297
WA3009	$2,152,125	$38,738	$766,737	$2,918,863
WA3010	$5,821,794	$104,792	$2,074,130	$7,895,924
WA3011	$6,718,520	$120,933	$2,393,606	$9,112,126
WA3012	$5,268,408	$94,831	$1,876,975	$7,145,383
WA3013	$2,328,718	$41,917	$829,652	$3,158,369
WA3014	$23,832,112	$428,978	$8,490,663	$32,322,775
WA3015	$5,460,507	$98,289	$1,945,414	$7,405,922
WA3016	$8,667,830	$155,057	$3,069,016	$11,736,846
WA3017	$15,545,257	$279,284	$5,527,805	$21,073,063
WA3018	$3,564,776	$64,166	$1,270,022	$4,834,798
WA3019	$12,560,526	$226,089	$4,474,937	$17,035,463
WA3020	$3,071,416	$55,285	$1,094,253	$4,165,668
WA3021	$10,993,499	$195,864	$3,876,698	$14,870,197
WA3022	$4,418,291	$79,529	$1,574,104	$5,992,395
WA3023	$13,767,399	$247,813	$4,904,909	$18,672,309
WA3024	$1,129,946	$20,339	$402,566	$1,532,512
WA3025	$4,434,408	$79,819	$1,579,846	$6,014,254
WA3026	$1,341,648	$24,150	$477,989	$1,819,637
WA3027	$1,831,524	$32,967	$652,517	$2,484,040
WA3028	$5,617,933	$101,123	$2,001,500	$7,619,433
WA3029	$599,386	$10,789	$213,543	$812,929
WA3030	$14,999,886	$269,998	$5,344,007	$20,343,893
		Total Present Value Cost =		$251,274,535

LID80 - LID with 80% Capture - LULC: Future				
Basin	Total Cost for Construction	Pond Maintenance Cost (Annual)	Pond Maint. Cost for 40 yr at PV	Total Present Value
WA3001	$20,090,395	$361,627	$7,157,602	$27,247,997
WA3002	$2,182,008	$39,276	$777,384	$2,959,391
WA3003	$12,698,263	$228,569	$4,524,008	$17,222,271
WA3004	$7,340,731	$132,133	$2,615,281	$9,956,012
WA3005	$2,071,365	$37,285	$737,965	$2,809,330
WA3006	$4,106,837	$73,923	$1,463,142	$5,569,979
WA3007	$7,701,931	$138,635	$2,743,966	$10,445,896
WA3008	$6,196,846	$111,543	$2,207,749	$8,404,595
WA3009	$4,304,251	$77,477	$1,533,475	$5,837,726
WA3010	$11,643,588	$209,585	$4,148,259	$15,791,847
WA3011	$13,437,040	$241,867	$4,787,212	$18,224,253
WA3012	$10,536,816	$189,663	$3,753,950	$14,290,765
WA3013	$4,657,435	$83,834	$1,659,304	$6,316,739
WA3014	$47,664,223	$857,956	$16,981,326	$64,645,549
WA3015	$10,921,015	$196,578	$3,890,828	$14,811,843
WA3016	$16,922,712	$304,609	$6,029,052	$22,951,764
WA3017	$30,863,131	$555,536	$10,995,603	$41,858,735
WA3018	$7,129,552	$128,332	$2,540,045	$9,669,597
WA3019	$25,121,052	$452,179	$8,949,874	$34,070,926
WA3020	$6,142,831	$110,571	$2,188,506	$8,331,337
WA3021	$21,121,896	$380,194	$7,525,095	$28,646,990
WA3022	$8,836,582	$159,058	$3,148,208	$11,984,789
WA3023	$27,534,799	$495,626	$9,809,819	$37,344,618
WA3024	$2,259,893	$40,678	$805,132	$3,065,024
WA3025	$8,868,816	$159,639	$3,159,692	$12,028,508
WA3026	$2,683,296	$48,299	$955,977	$3,639,273
WA3027	$3,663,048	$65,935	$1,305,033	$4,968,081
WA3028	$11,235,866	$202,246	$4,003,001	$15,238,867
WA3029	$1,198,771	$21,578	$427,086	$1,625,857
WA3030	$29,999,772	$539,996	$10,688,015	$40,687,787
		Total Present Value Cost =		$500,646,347

LVL2Wet - Level II with Wet Pond - LULC: Future				
Basin	**Total Cost for Construction**	**Pond Maintenance Cost (Annual)**	**Pond Maint. Cost for 40 yr at PV**	**Total Present Value**
WA3001	$6,010,231	$6,929	$137,139	$6,147,370
WA3002	$597,786	$603	$11,929	$609,715
WA3003	$5,327,536	$5,699	$112,791	$5,440,327
WA3004	$2,103,438	$2,299	$45,509	$2,148,947
WA3005	$1,008,548	$905	$17,920	$1,026,467
WA3006	$2,021,424	$1,988	$39,349	$2,060,773
WA3007	$3,028,341	$3,158	$62,512	$3,090,853
WA3008	$1,830,044	$2,017	$39,919	$1,869,963
WA3009	$1,227,487	$1,303	$25,798	$1,253,285
WA3010	$4,229,759	$4,496	$88,984	$4,318,744
WA3011	$6,830,409	$6,893	$136,425	$6,966,835
WA3012	$5,427,446	$4,274	$84,597	$5,512,043
WA3013	$3,879,003	$2,796	$55,350	$3,934,353
WA3014	$33,460,714	$25,797	$510,591	$33,971,306
WA3015	$3,671,154	$3,916	$77,515	$3,748,668
WA3016	$4,800,986	$5,496	$108,771	$4,909,757
WA3017	$14,921,630	$16,457	$325,731	$15,247,362
WA3018	$5,589,982	$6,151	$121,751	$5,711,732
WA3019	$9,258,548	$10,212	$202,123	$9,460,672
WA3020	$1,776,196	$1,911	$37,829	$1,814,025
WA3021	$25,493,275	$19,974	$395,336	$25,888,611
WA3022	$4,246,661	$15,484	$306,478	$4,553,139
WA3023	$22,032,338	$16,211	$320,851	$22,353,189
WA3024	$1,222,520	$846	$16,748	$1,239,268
WA3025	$7,263,564	$5,118	$101,304	$7,364,868
WA3026	$1,429,877	$1,156	$22,878	$1,452,756
WA3027	$1,848,303	$1,741	$34,465	$1,882,768
WA3028	$8,542,673	$6,059	$119,920	$8,662,592
WA3029	$863,221	$551	$10,905	$874,126
WA3030	$20,976,839	$15,221	$301,257	$21,278,097
		Total Present Value Cost =		$214,792,610

Cistern - Cistern for Roofs - LULC:Future				
Basin	Total Cost for Construction	Annual Inspection Cost per basin	Inspection Cost for 40 yr at PV	Total Present Value
WA3001	$6,451,759	$145,745	$2,884,702	$9,336,461
WA3002	$638,241	$14,418	$285,370	$923,611
WA3003	$4,061,709	$91,754	$1,816,066	$5,877,774
WA3004	$2,376,808	$53,692	$1,062,715	$3,439,523
WA3005	$702,361	$15,866	$314,039	$1,016,400
WA3006	$1,125,068	$25,415	$503,039	$1,628,106
WA3007	$2,308,680	$52,153	$1,032,254	$3,340,934
WA3008	$1,741,006	$39,329	$778,436	$2,519,442
WA3009	$1,216,718	$27,486	$544,017	$1,760,735
WA3010	$4,125,480	$93,195	$1,844,579	$5,970,060
WA3011	$4,649,072	$105,023	$2,078,687	$6,727,758
WA3012	$3,844,257	$86,842	$1,718,839	$5,563,096
WA3013	$1,493,934	$33,748	$667,966	$2,161,899
WA3014	$16,826,531	$380,111	$7,523,456	$24,349,987
WA3015	$55,397,197	$1,251,423	$24,769,121	$80,166,318
WA3016	$5,559,127	$125,581	$2,485,590	$8,044,717
WA3017	$11,518,832	$260,210	$5,150,285	$16,669,117
WA3018	$4,872,099	$110,061	$2,178,406	$7,050,505
WA3019	$7,079,720	$159,931	$3,165,475	$10,245,195
WA3020	$1,323,527	$29,898	$591,774	$1,915,301
WA3021	$13,046,220	$294,714	$5,833,209	$18,879,429
WA3022	$1,998,707	$45,151	$893,659	$2,892,366
WA3023	$8,943,216	$202,027	$3,998,679	$12,941,896
WA3024	$883,397	$19,956	$394,983	$1,278,380
WA3025	$2,856,491	$64,528	$1,277,190	$4,133,681
WA3026	$990,554	$22,377	$442,895	$1,433,450
WA3027	$669,604	$15,126	$299,393	$968,997
WA3028	$3,515,466	$79,414	$1,571,831	$5,087,297
WA3029	$477,592	$10,789	$213,540	$691,132
WA3030	$7,017,168	$158,518	$3,137,507	$10,154,674
		Total Present Value Cost =		$257,168,241

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