Performance of Sedimentation Basins and Oil/Grit Separators at Anchorage, Alaska

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EXECUTIVE SUMMARY

The Municipality of Anchorage (MOA) and the State of Alaska Department of Transportation and Public Facilities (DOT) are required under their joint Alaska Pollution Discharge Elimination System (APDES) storm water permit to evaluate the performance of oil/grit separators (OGS) and sedimentation basins within the Anchorage municipal separate storm sewer system (MS4) and to report results in the third year of the permit term (AKS-052558, IV.A.8., p. 39). Through the MOA's Watershed Management Services Section (WMS), the permittees completed work in compliance with this permit requirement during the period 2011-2012. This report summarizes the results and recommendations of that work, referenced here as the Project.

This report is organized into an Executive Summary, briefly outlining principle findings and recommendations, and a series of technical appendices, summarizing in more detail the findings, discussions and recommendations of core elements of Project work. Appendix A, "Project Technical Summary", provides a description of the project approach and history. Appendix B, "Performance Analyses", provides discussion of the results and findings of performance analyses made of select Anchorage OGS and sedimentation basins in 2012, and a brief conceptual outline of recommended changes to Anchorage design and maintenance practices for these types of water quality control devices. Appendix C, "Project Data Analyses", includes discussions of the results of various data collection, review, and analyses efforts performed to achieve overall Project goals. Appendix D, "Project Data Records", describes content, record structures, and approaches used in managing and archiving quality control records, hardcopy field and laboratory records, and all digital data collected or compiled and used to complete Project work. Appendix E, "Project Stations and Equipment Inventory", describes field locations and equipment installations, characteristics, maintenance practices, and storage locations that supported Project work. Appendix F, "Data References", describes physical hardcopy locations and computer digital archival locations of all data, calculation sheets, worksheets, and other project records. Appendix G, "Project Team", lists project team members, their primary expertise, their affiliations, and their primary responsibilities and roles in performing Project work. References cited in all sections of this report are compiled in a single reference list in the Reference section.

Project Goals and History:

Part IV.A.8. of the APDES storm water discharge permit AKS-052558 requires permittees to:

> *'..select and evaluate the effectiveness' of four oil and grit separators and three sedimentation basins in the treatment of water quality parameters described in Table IV.A.'*

Parameters listed at Table IV.A (IV.A.7, p 39) include dissolved oxygen (DO), biochemical oxygen demand (BOD), pH, temperature, fecal coliform, total aromatic hydrocarbons (TAH), total aqueous hydrocarbons (TaqH), flow, turbidity, and total suspended solids. However, OGS and sedimentation basins are primarily intended for density separation of solids. Therefore this Project focuses principally on the performance of these devices in removing particulates from storm water.

This Project is intended to provide for compliance with Part IV.A.8. and to provide useful information to the permittees in applying oil and grit separators and sedimentation basins to storm water quality treatment at Anchorage. Project goals include:

(1) Completion of performance evaluation of these types of devices in controlling the listed Table IV.A pollutants to the maximum extent practicable in context with environmental conditions and the water quality treatment train typical of the Anchorage MS4.

(2) Compilation of conceptual guidance for application, design and maintenance of these controls to optimize their use and performance as elements in the water quality 'treatment train' typical of the Anchorage MS4.

Field work and data analysis for the Project was completed by WMS and HDR Alaska staff (Appendix G) in 2011 and 2012, and included monitoring and sampling at three Anchorage sedimentation basins and four OGS (Appendix E). Six field monitoring stations at the three sedimentation basins and a cellular weather telemetry station were installed and tested in fall 2011. Continuous monitoring instrumentation was installed at the six stations in early spring 2012 and operated into September 2012. Storm water grab sampling at the six sedimentation stations was completed throughout both the snowmelt and rainfall runoff seasons in 2012. Additional grab sampling at four Anchorage OGS was also completed in 2012. Finally sediments were collected and mixed from Anchorage streets and used to complete full-scale laboratory performance testing of an OGS model commonly used in Anchorage installations.

All Project data was compiled, validated, and archived for analysis. Data analyses were performed throughout the sampling period as needed to adjust sampling schedules. Final data analyses, performance analyses and design methods development were completed in the fall and early winter of 2012. Field sampling and data records, laboratory reports, worksheets and summary report documents are compiled and archived as hardcopy and digital records (Appendices D, E and F).

Project Approach

The approach WMS took in evaluating OGS and sedimentation basin performance at Anchorage was driven by a number of considerations, including:

- The time available to complete Project work;
- The Anchorage conditions under which these devices must perform;
- The intrinsic and system characteristics limiting performance of the devices, and;
- The nationally-accepted standard practices used in water quality treatment design.

The timeframe in which Project work was to be completed was severely constrained by scheduling mandated in the permittees' storm water permit. The permit schedule limited time for field monitoring to a single water year (October to October, 2011-2012). The city's sub-arctic transitional maritime location also drove Project strategy. Winter-long average daily maximum temperatures below freezing and low solar insolation result in springtime accumulated street dirt loads at Anchorage that are unusually high relative to most United States communities. Similarly, cyclonic-type storm events combined with orographic shadowing result in semi-arid conditions for most of urban Anchorage, and summer rainfall events with unusually low peak intensities. Also OGS and sedimentation basins are intrinsically density separation devices typically located at or near the end of piped storm drainage networks. Consequently they are primarily suited for particle removal with their performance limited by the size and character of the particles that actually reach them. Finally, this Project also explicitly recognizes the fundamental national and international strategy in design of storm water quality treatment devices that reflects the common reality of a predominantly gamma distribution in rainfall. That is, almost all (usually around 90%) rainfall events result in relatively low-intensity, low-volume runoff, with only a very few generating large flows, leading to a common treatment strategy: treat all small flows (under which conditions treatment processes are very effective), and bypass the few flows that are larger.

To most effectively address these constraints, WMS selected a sum-of-loads approach to assess device water quality treatment performance, with monitoring data collected in 2012 normalized to reflect performance under mean seasonal conditions. This approach assumes a narrow variance in seasonal street sediment loading and particle size distribution characteristics at Anchorage. It also assumes particulate transport and loading at OGS or sedimentation basins is closely related to flow, and the presence and known treatment efficiencies of other headwater controls is reasonably known. These assumptions were tested in this Project through review of national research and a number of detailed Anchorage street sediment loading studies. Flow and particulate transport relationships at Anchorage were measured directly through sampling for suspended sediment concentrations, with these relationships then correlated to continuous measurements made of turbidity (Nephelometric Turbidity Units, NTUs) for use as a surrogate measure of settleable solids. The limited sampling period (one water year) was optimized by capturing correlative data of the widest range possible in independent rising limb flows and analyzing them as a single population. Seasonal particulate removal was measured through use of these relationships to compare matched influent/effluent pairs or through direct measure of the entire seasonally captured particulate loads for tested devices. The Project also compiled and synoptically analyzed all historic precipitation data through 2011 from the National Weather Service (NWS) station at Anchorage International Airport to provide norms against which Project results were compared and adjusted.

This Project assesses OGS and sedimentation basins separately to address the significant intrinsic differences in treatment (primarily through availability of storage) of these two types of devices. OGS devices currently in use at Anchorage include baffle box and hydrodynamic designs. Hydrodynamic separators are separators that incorporate a dynamic element, typically through introduction of some designed angular or vertical flow component to the influent to help separate particulates. Baffle box designs used in Anchorage typically incorporate two or more offset vertical plates set directly across the flow path along the length of rectangular or cylindrical boxes. Baffle boxes were assessed in work performed by WMS in the late ’90s. That earlier assessment assumed these OGS design types performed ideally (no short circuiting or scouring). However typically bypasses for these earlier devices are absent or otherwise conducive to increased velocities and scouring at larger flows. Nevertheless, despite its optimistic assumptions, this earlier study estimated cost-marginal performance to be poor. As a result of these study results and advances in manufacture of hydrodynamic devices, the baffle boxes are now infrequently used at Anchorage and hydrodynamic separator types are more commonly applied. Based on these earlier results, this Project evaluation addresses only hydrodynamic separators.

Recent studies suggest hydrodynamic separators lend themselves well to design scaling (through use of a Péclet number—a ratio of settling process to turbulence), of known performance of one device to different sized models within the same family of devices. Scaling for design application is supported by the availability of standardized test results of intrinsic performance that many manufacturers complete of their proprietary devices. Because all OGS provide little treatment storage (with treatment occurring only in direct response to, and during, storm water runoff) device performance is then reasonably predictable based only on knowledge of the range in the storm flows and particulate loading at a specific site. To assess applicability of this type of performance assessment and design scaling to Anchorage conditions, WMS commissioned a full-scale laboratory test using Anchorage street dirt of an OGS_h model commonly used here. Given the relatively small size of these devices and their simple dynamic treatment response, performance should also be able to be assessed by measuring the seasonal volume of captured wastes. WMS completed inspection and sampling of four OGS_h in 2012 to provide additional exploratory assessment of in-place performance at Anchorage.

The approach to assessment of sedimentation basins was considerably more complex than that used in evaluating OGS. Converse to OGS, optimum designs for sedimentation basins include incorporation of significant storage and/or surface treatment area. Best particulate removal performance takes place when treatment occurs both during a storm event (dynamic treatment) and as the result of capture and detention of some portion of the runoff event (quiescent treatment). Similarly best performance occurs when the total surface and volume available for treatment is utilized (i.e., short circuiting, or a concentrated preferential flow, does not occur across the treatment basin). To test performance in this context, three sedimentation basin installations at Anchorage were selected for continuous influent/effluent monitoring. Basins were selected to the extent they reflected a range of characteristics critical to performance, including: length:width ratio, treatment basin volume: runoff volume ratio, storm flow: surface area ratio, inlet/outlet character and aspect and other short-circuiting factors, and constructed wetlands use and geometry.

Sedimentation basin sampling was conducted throughout 2012 at matched influent/effluent stations established at each of the Project test basins. Weirs and instrumentation were set up at a total of six stations in the fall of 2011 and operated from the beginning of the snowmelt season in the spring of 2012 through late fall of that same year. Sensors and dataloggers were used to continuously monitor flow, temperature, conductivity, and Nephelometric Turbidity Units (NTU) at paired influent/effluent stations for each sedimentation basin. In addition, passive cumulative devices (pcd's) were installed to measure average concentrations of petroleum hydrocarbons (diesel and gasoline range, and total aqueous hydrocarbons) at each station over the project period. A remote telemetry weather station using cellular communications was installed at one of the sedimentation basins and, along with National Weather Service (NWS) web-based radar and GOES satellite data, was used to trigger grab sampling at all stations. A standard daily storm watch window was established from 5am to 11pm seven days a week. Once a sampling event was triggered, sampling continued until completion. Rising limb peak flow targets were established as sampling triggers, with an overall Project objective to collect samples over the broadest possible range in peak flow magnitudes. To achieve this, the Project sampling coordinator identified current target threshold flows for all sampling crews. For sampling events triggered after 8a and before 5pm Monday through Friday, sample collection was scheduled for all Table IV.A parameters. During all non-business hours sampling events, only TSS grab samples and field measurements were collected.

Project 2012 sedimentation basin data was analyzed: 1) to establish correlation between NTU, TSS and flow data; 2) to identify and establish influent/effluent storm hydrograph and pollutograph pairs; 3), to estimate seasonal influent and effluent particulate loadings, and 4), to normalize 2012 storm event data and seasonal pollutant loadings to the historic mean at each of the sedimentation basins. Suspended sediment concentration values of grab samples (obtained using a modified TSS laboratory method) were used to establish correlation with continuous NTU data and flow. Historic NWS precipitation data for the Anchorage International Airport weather station was compiled through the year 2010 and synoptically analyzed using the EPA module SYNOP to statistically identify and characterize meteorologically independent storm events over the period of record. Similar analysis was performed for the 2012 Project weather dataset, and statistics for the Project year normalized to the historic record. The beginning of Project storm runoff events were established for each of the three Project influent stations based on inspection of station flow records and the 2012 synoptic storm analysis. Storm hydrographs for paired influent/effluent stations were then developed using station flow records, applying influent station start point data as hydrograph start times for both influent and effluent stations, and seasonal trend analysis of base flow data of influent stations to establish best fit end points of effluent station hydrographs.

Once matched, hydrographs at influent/effluent station pairs were established, and pollutographs for particulate loading were developed by applying the TSS-correlated NTU data to the hydrographs. Gross error was checked using mass balance methods for flow and particulates between the paired stations. Treatment performance for particulates and basin hydraulic efficiency were estimated through analysis of storm by storm and base flow particulate removals summed over each seasonal period using the NTU surrogate measure, and through calculation of a number of hydraulic efficiency parameters based on storm by storm analysis. Treatment for other parameters listed at Table IV.A was estimated through inspection.

Finally, collected test data and observations were used to test, calibrate, normalize and set standard parameters for simple particulate pollutant transport models and several design methodologies either used or proposed for use at Anchorage. Annualized analysis of all complete seasonal records of NWS precipitation data established a median 90 percentile rainfall intensity, and synoptic storm analyses identified mean and variation in rainfall storm intensities for use in device designs. Inspection of previous inventories and street sediment sampling studies made along Anchorage's streets and parking lots provided estimates of particulate loading and particle size distributions on these surfaces. Application of simple shear stress models to a typical distribution of sediments observed on Anchorage streets provided estimates of washoff loadings to headwater controls (primarily catchbasins). Adjustment of observed treatment performance of catchbasins made in various national studies to match the geometry common to most Anchorage catchbasins provided estimates of loading and characteristics of particulates transmitted beyond these devices to any OGS and sedimentation basins further along the storm drainage system. Project data of the characteristics of sediments seasonally captured by OGS_h were compared to modeled results to calibrate the models and refine estimates of overall 'treatment train' performance. Finally, sedimentation basins Project data was used to calibrate current and proposed design models, and to assess relative effects of the various basin geometric design parameters on individual performance of Anchorage basins.

Project Results

Because of this Projects systemic and sum-of-loads (seasonal performance) approach to controls evaluation, findings are grouped first, as they relate to the effects of Anchorage headwater (system) conditions on down-line controls, and, second, to performance of OGS and sedimentation basins in context with these headwater systems.

Headwater Factors

The findings of this Project well illustrate the fundamental concept that all storm water controls work in context with the entire storm drainage system and the series of controls in place along its length (i.e., as part of a whole 'treatment train'). For OGS and sedimentation basins, typically located nearer the end of storm drainage systems as they are, this is particularly true. As a result, local performance of these devices clearly can only be evaluated in context with system characteristics, and most particularly characteristics of the 'headwater' portions of these systems.

Critical headwater characteristics addressed in this evaluation include weather, particularly precipitation, street sediment loading and distribution, runoff and washoff characteristics, and the presence and efficiency of headwater water quality controls. For this study, precipitation and associated runoff is stratified into two seasonal sum-of-load events, including a spring snowmelt runoff event and a summer/fall rainfall runoff event. Mid-winter runoff events do occur at Anchorage but they remain relatively rare as a result of below-freezing average maximum daily winter temperatures. This Project also concentrated only on characterization of sediment (and associated pollutants) loading and washoff from streets and larger parking lots. Though other national studies report these sources as only two of many, at Anchorage the assumption of streets and parking as a primary source may be a reasonable first estimate given the unusually large application of traction sand made to Anchorage streets each winter. The seasonal washoff models applied in this study also differed from past Anchorage analytical efforts in their simplicity. However, significant re-distribution, concentration and fining of winter street dirt along Anchorage gutter lines as a result of spring sweeping (commonly observed nationwide as well) and some uncertainty in actual street sediment loading is believed to make such simple models adequate until further study can more finely resolve variations across each of the runoff seasons. Finally, for this performance analysis, Project analysts assume headwater controls consist predominantly of catchbasins and that these perform similarly to those studied in other national investigations. Project findings of the effects of each of these critical headwater factors at Anchorage are briefly summarized below.

Climate and Precipitation

Statistical and synoptic analysis of Anchorage historic precipitation data, as well as 2012 Project records, reflects Anchorage sub-arctic and semi-arid climatic conditions. Snow accumulates all winter long and snowmelt runoff occurs in a single seasonal event three to six weeks in length. Snowmelt runoff is diurnal early in the season, becoming continuous towards the end of the snowmelt event. End-of-winter sediment loading is large (50,000 to 115,000 pounds per curb mile, 16/cmile) at the beginning of the spring runoff event and is relatively uniformly distributed across street and parking surfaces. However ice, common along these surfaces throughout much of the snowmelt event, along with larger particles not yet removed with sweeping tend to protect the accumulated street dirt from mobilization with melt water. Ice cover is still present on sedimentation basins during urban snowmelt as well, which is suspected to affect performance of these devices where basin depths are shallow. In any event, as a result spring snowmelt washoff loads may typically be an order of magnitude or more smaller than that those mobilized during the summer rainfall season, despite much smaller street sediment loading in the summer. These observations are supported by 2012 Project data.

Rainfall runoff occurs at Anchorage from about May through October, with storm events rising in frequency towards the fall, and represents the summer seasonal runoff event. This seasonal event in urban Anchorage is driven solely by rainfall. Synoptic analysis of the historic rainfall record reveals Anchorage summer storm events have very small mean volumes and intensities but also relatively short separation times (Table 0.1). Low rainfall intensities are particularly prominent. Analysis of annualized rainfall records show that even at the median annual 90th percentile of hourly intensity (representing approximately 90% of total seasonal rainfall volumes), rainfall intensities remain low at about 0.12 inches per hour (in/hr).

Table 0.1: SYNOP rainfall storm statistics for Anchorage (Historic and 2012)

	Historic (1963-2010) Rainfall Statistics	**2012 Rainfall Statistics**
Mean Storm Volume inches	0.24	0.34
Mean Storm Intensity inches/hour	0.026	0.028
Mean Storm Duration hours (start to end of rainfall)	13.17	24.48
Mean Storm Inter-event Time hours (centroid to centroid)	90	110
Separation time (dry hours between storms)	79	88
90 percentile annualized intensity inches/hour	0.12	0.08
Annual number of storms volume >.02 inches total rainfall	40	29

These local rainfall characteristics are driven predominantly by the cyclonic nature of Anchorage storms and the orographic effects the mountain ranges surrounding the city have on them. Weather systems are variously addressed and classified at the synoptic scale, the meso-scale, or the micro- or storm-scale. At the synoptic scale, atmospheric features having horizontal scales on the order of 600 miles and more are addressed. Near Anchorage, synoptic events include cyclonic storms centered along the polar jet stream. Mesoscale and micro-scale meteorology addresses smaller scale atmospheric features and effects. At Anchorage these include meteorologic features within cyclonic events, orographic effects and, more rarely, thunderstorm and other cumulus rain events.

Anchorage rainfall precipitation is strongly affected at all of these scales in a complex fashion. At the largest scale, a single cyclone complex can take 24 to 48 hours or more to completely transit the Anchorage peninsula. At the mesoscale, however, individual rainbands within a single cyclonic event can result in periods (1 to 4 hours or more) of steady rainfall interspersed with periods of variable length (2 to 3 hours to more than 6 hours) of no rainfall at all. In addition, at both the mesoscale and the micro-scale, rainfall intensity from a single cyclonic rainband can vary significantly due to its relatively small scale or to local orographic (mountain shadowing) effects.

The cyclonic process typical of Anchorage is well represented in a mid-June storm event sampled during the 2012 project year (Figure 0.1and Figure 0.2). The storm began June 12th at 3:05am and ended June 13th at 6:05am for a duration of about 28 hours, more than twice the historic average, and at 0.66 inches, about three times the average storm volume (Figure 0.2). Nevertheless, maximum rainfall intensity during this storm did not exceed 0.05 in/hr (or about 40% of the 90th percentile intensity) and averaged about 0.029 in/hr (just slightly greater than the historic mean storm intensity). The strange disparity between overall storm size and average and maximum intensities is explained by the fact that the total rain volumes released by a single meteorological event (one cyclone) passing over the Anchorage area is related to the size of the cyclone (i.e., to the size of the entire feature at the synoptic scale). Rainfall intensity, however, is more related to separation distances between rainbands and the effects the terrain has on the release of rain from these features, which is limited by the micro-scale of these features within the cyclone (Figure 0.1). This intermittent—rainband—type of cyclonic rainfall is also expressed in the June 12 storm in the multimodal (series of) rainfall peaks so characteristic of Anchorage rainfall storms. That is, rainfall in an Anchorage storm is not typically expressed in one large (SCS) type of peak but rather in many much smaller peaks spread over the entire storm.

These characteristics have very important implications for Anchorage storm water quality treatment design. Practically speaking, low intensity rainfalls represent optimum conditions for application of density separation types of devices like OGS and sedimentation basins. Recognition and use of synoptic Anchorage rainfall distribution characteristics (rather than the non-representational and inappropriately peaky SCS rainfall distribution currently used) allows selection of a practicable design storm specifically for water quality purposes. This in turn means that future designs can identify practicably-sized devices that will still easily be able to meet a goal of treating 90% of all rainfall runoff—a standard treatment objective.

Street Sediments Loading and Washoff

However, despite the relatively low intensities of Anchorage rainfall runoff events, 2012 Project data suggests, as noted earlier, that a significantly larger sediment load washes off Anchorage streets during the summer rainfall runoff seasonal period than during the snowmelt runoff seasonal period. Both local and national data suggests this is primarily due to the fining and redistribution effects of street sweeping, rather than the presence of a larger sediment loading (reasoned as follows: finer particles are more easily mobilized

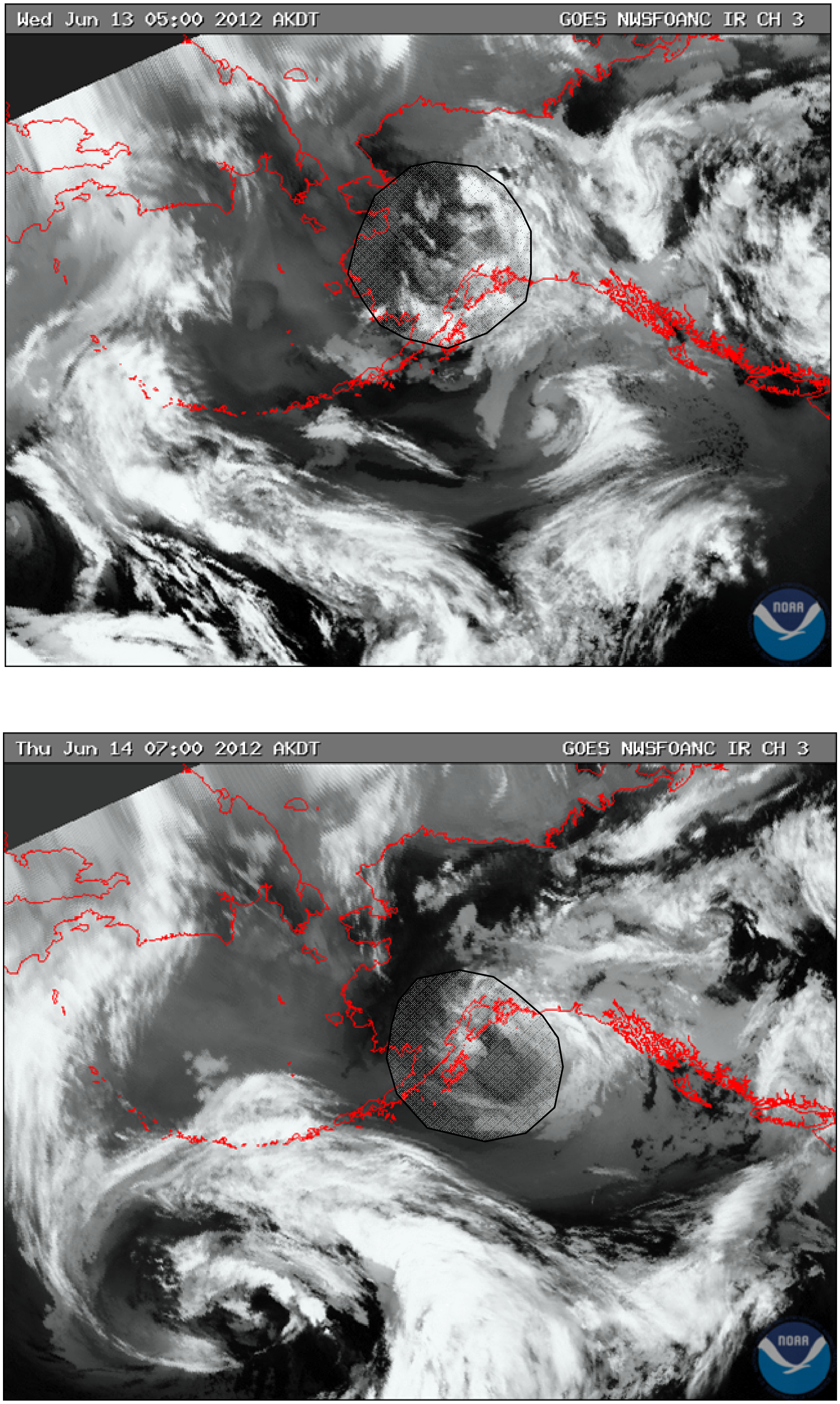

Figure 0.1: Movement of June 12th Cyclonic Event over Anchorage

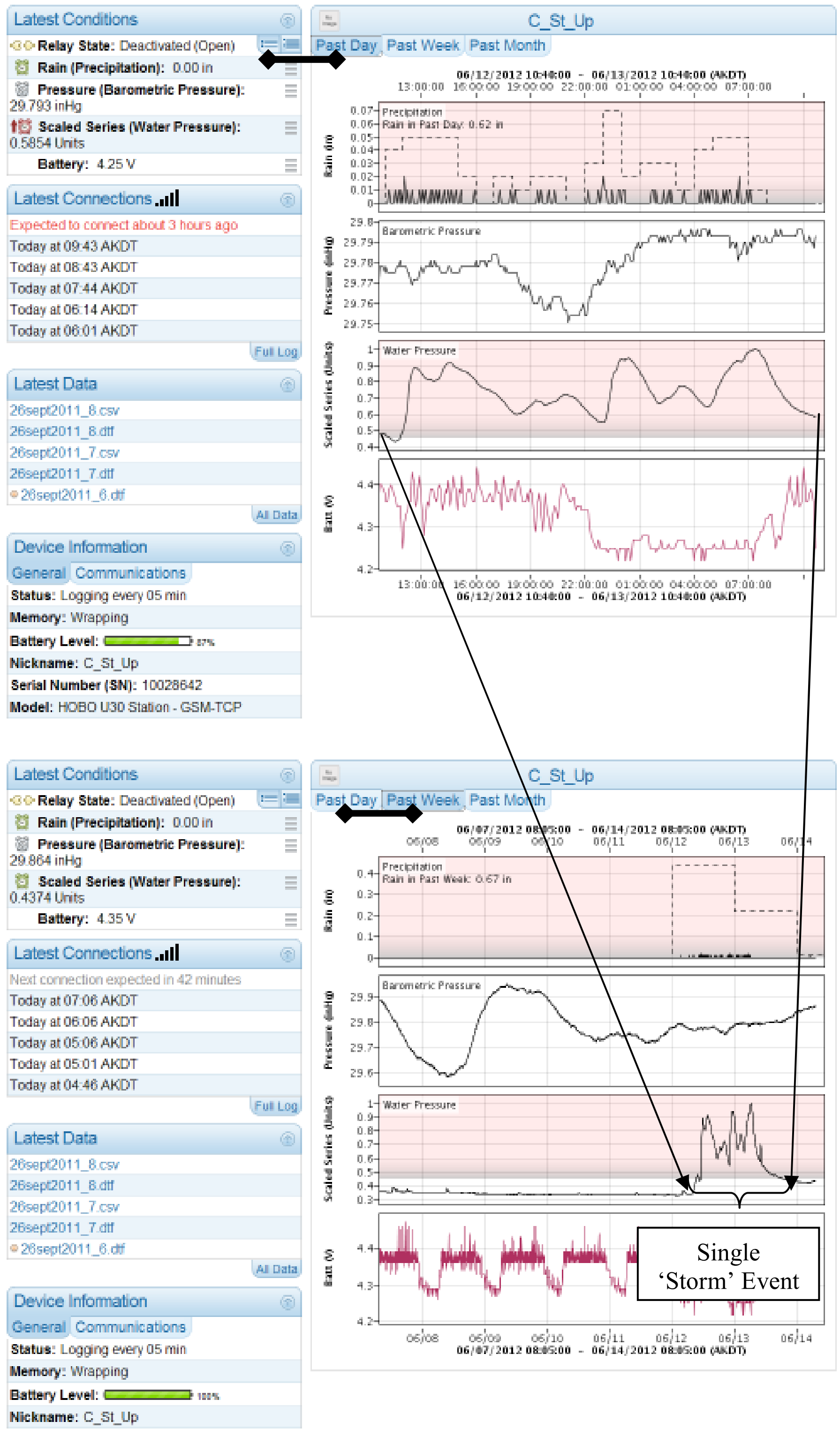

Figure 0.2: June 12th Storm Telemetry; Peak Rainfall Intensity <0.05 in/hr

by runoff, preferential removal of coarser particles removes the ‘armoring’ the larger particles can provide, and concentration of street sediment along the gutter pan makes them more subject to mobilization by concentrated flow). In fact, earlier WMS studies suggest the large winter sanding loads left on Anchorage streets and parking lots following the spring snowmelt event are significantly removed through street sweeping, with an estimated late spring residual load of about 1000 lb/cmile remaining after spring sweeping. Other WMS studies suggests that over the summer this winter residual is likely supplemented with normal summer particulate buildup yielding another 3000 lb/cmile over the 120 day summer period, for a total seasonal estimated washoff load of about 4000 lb/cmile. Based on earlier commercial parking lot sediment loading studies completed by WMS, this Project estimates an additional 4000 lb/cmile per seasonal period is contributed by larger, parking lots along arterial streets.

Though these estimates are based on substantial sampling and analysis, recent street sweeping inventories made by the permittees’ street maintenance agencies imply a summer load available for washoff much higher than this. Based on Street maintenance records, about 15,000 lb/cmile of sweeping wastes are swept up at the end of each of two 60-day summer sweeping intervals, suggesting a summer build up rate closer to about 250 lb/cmile/day. Some of the discrepancy in estimates may be due to high bias in sweeping wastes inventory estimates combined with analytical underestimation of summer buildup rates. Nevertheless differences remain significant and are scheduled to be tested and resolved in focused sweeping performance analyses required to be completed by the permittees in 2013. These Project’s findings and recommendations should be adjusted appropriately by the findings of these future studies.

For the purposes of this Project, the lower WMS estimate of 4000 #/cmile with an additional 4000 lb/cmile estimated to be contributed by larger, parking lots is used. This summer street sediment load is concentrated along the gutter pan and has a particle size distribution (PSD) finer than the original winter load as a result of spring and summer sweeping. Specifically, differences in particle size distributions between winter residual street sediments and post-spring sweep residual street sediments shows a notable reduction in coarser particle sizes and an overall increase in relative mass of fine particle sizes. However, PSD differences between sediments on the street in summer and that of particulates mobilized in storm water show little change, reflecting this Project’s conclusion that rainfall runoff is capable of mobilizing the entire particle distribution (Figure 0.3). A simple shear stress model prepared by this Project also suggests the entire summer street sediment load can be readily mobilized into curb and gutter storm drain inlets under normal frequency rainfall intensities. Based on these findings, the Project estimates 100% of street sediments not swept up are washed off into the storm drain system over the summer season.

Both earlier characterizations and 2012 Project data for street sediments reveal other qualities that have implications for OGS and sedimentation basin treatment of these wastes as well. Street sediment sampling and characterization performed by WMS in 2000, 2010 and 2011 showed significant vegetable organics loading, apparently mostly in the form of fallen leaves and grass clippings. As would be expected for such a source, loading was spatially and seasonally variable and associated mostly with residential streets in late fall. Based on the WMS studies, leaves can form as much as about 90% of street particulates by volume, or about 30% by weight. Leaves on the road surface are readily comminuted by some street sweeper types and the comminuted leaves are not easily trapped by smaller headwater controls (e.g., catchbasins). On the other hand, OGS_h may be effective at trapping a significant fraction of the comminuted organics.

Project sampling in 2012 at an OGS_h serving a largely residential neighborhood showed a 20% by weight organic content, assumed to be substantially from fall leaf load. Inspections in 2012 suggested that buildup of these fine fibrous organics from comminuted fall leaves may also create some clogging problems on screened-types of OGS_h.

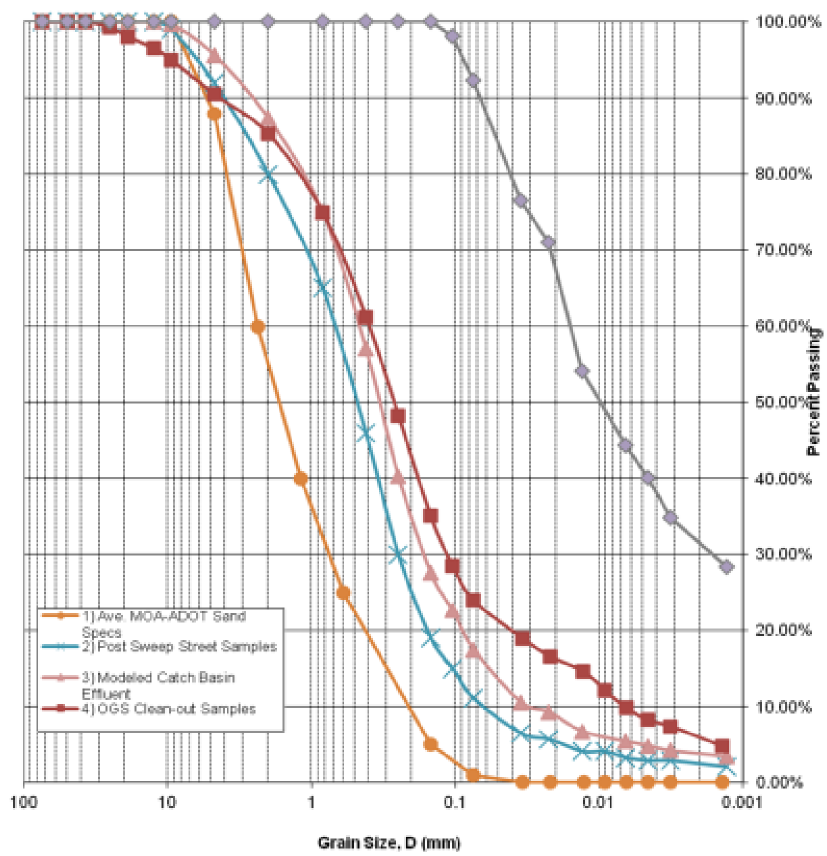

Figure 0.3: Anchorage relative particulate size distributions (PSDs)

Headwater Controls

The last critical headwater factors addressed in this Project are those water quality controls commonly in series with and up-gradient of OGS and sedimentation basin devices. At Anchorage these controls include low-impact-development (LID) practices (including use of well-designed and managed open channel drainages), on-site controls (including on-site catch basins, OGS, and small water quality/detention basins), and all off-line street inlet catch basins (on-line catch basins, including ‘manhole’ catch basins, are ineffective). Note that street and parking management practices, including, for example, winter sanding, deicing, snow plowing, sweeping etc., can certainly be included in this category. However, this Project has implicitly addressed these in earlier discussion of street sediment loading. They will be addressed in more detail in assessment of these issues in the sweeping performance analysis due under the permittees’ storm water permit schedule in 2013.

Given particulates as the primary pollutant target for treatment by OGS and sedimentation basin type-controls, any headwater controls that remove from, or limit mobilization of particulates in, storm water should be considered in the application, design and performance evaluation of down-line OGS and sedimentation basins. This can, and should, encompass a wide selection of practices and controls. For example, any substantively-applied LID practice certainly should be included, as these reduce mobilizing runoff flows and incorporate on-site controls that focus on low-flow particulate capture. Similarly, open channel drainage systems, public and private, should also be included to the extent that they are designed to optimize water quality control. Curb-less, broad vegetated shoulders encouraging sheet flow runoff entrance from street and parking surfaces into appropriately vegetated or lined ditches again minimize runoff tractive forces and promote best opportunities of particulate capture at low flows. Small headwater flow breaks and water quality detention pools or ponds installed either separately or along open channel systems serve similar purposes. Because these are placed where flow energies are smallest and only small efforts are required to prevent or treat pollutant mobilization—a sort of 'water quality judo'—they, along with good maintenance practices, have very large effect on the character and amount of particulates that are actually carried by storm water to down-line controls.

The effects of headwater controls should be considered in detail in applying and designing OGS or sedimentation basins at each specific site. However, presence of many of these headwater elements is highly variable (similar to the high spatial variability in sediment loading recognized by this study and others nationwide). Given such high variability, this Project has solely considered the effects of inlet catchbasins placed along public streets, a much more consistent element in public piped storm drainage design. Approaches and guidance in considering other headwater controls is only touched on in this current study, and should be addressed in more detail in development of formal design guidance documents for the Anchorage area.

Because of the low-intensity rainfall at Anchorage, properly designed and maintained inlet catch basins can be very effective at treating headwater-mobilized particulates. However Project observations and national research emphasize that performance of these devices is directly related to their design geometry and maintenance practices. To perform optimally, catch basins must meet certain minimums including: minimum spacing at off-line locations, outlet invert-sump geometries, sediment storage capacities, and maintenance schedules. Off-line position of catch basins is essential—at seasonal scales on-line devices are subject to scour and remobilization of any captured sediments and are not effective as water quality control devices (and in fact may be sources of problems from pollutant transformations and re-transport). Because of their headward-most positions, off-line catchbasins can remain effective at particulate capture but require design and maintenance of a minimum separation between outlet invert and the top of captured wastes to minimize turbulence effects. Similarly, waste storage sumps are subject to scour when captured sediments exceed about 50% of the sump capacity and are ineffective when 60% capacity is reached. Given this, design and optimum maintenance schedules are obviously strongly related, with designed sizes and locations in significant part driving the frequency with which a catchbasin must be cleaned to remain effective.

Nevertheless, catchbasins can predictably result in as much as a 40% reduction in the total transmitted storm water mineral particulate load, given optimum design standards and maintenance practices. This project used estimated average street sediment loading and washoff characteristics along with Anchorage standards for catchbasin location and design geometry to estimate average treatment by these headwater controls. Based on these Project data and assumptions, under average system conditions Anchorage catch basin design standards appear to be marginally adequate to achieve optimum performance assuming an annual cleaning schedule (Appendix B.1). That is, optimum treatment performance for Anchorage catchbasins is assumed as an initial condition in the evaluations and design recommendations presented in this Project for OGS and sedimentation basins. Assumptions include preferential removal of coarser particulates and mobilization of a smaller total storm water particulate load—but one with a significantly finer particle size distribution than the original street sediments (Figure 0.3: Anchorage relative particulate size distributions (PSDs)). However given the uncertainties in street sediment loading and the marginal geometries of Anchorage catchbasin standards relative to optimum performance, additional adjustments may be required based on further study or modification of headwater system conditions and standards.

OGS$_h$ 2012 Performance

As described earlier, this Project analyzed and evaluated Anchorage hydrodynamic oil/grit separators, OGS$_h$, in context with headwater systems through field inspections of in-place devices and through full-scale laboratory testing of an OGS$_h$ commonly used at Anchorage. Project staff made field inspections of a number of Anchorage OGS$_h$ and assessed the performance of four devices through volumetric measurements, sampling and laboratory testing (Appendix B.1). Volumetric measurement of the total content of waste sediments captured in the four tested devices was related to the last cleaning time for each device to provide an estimate of annual capture rates and to provide system insight for application to Project efforts in development of street sediment loading and washoff models.

Though the sample population number is obviously exploratory, OGS field observations and sampling results (Figure 0.4) reflected, on the one hand, the wide spatial variability in headwater sediment loading characteristics already indicated by other Project data collection efforts, and on the other, the significant effect headwater controls have on the fining of particulates transmitted further down storm water pipe systems. Differences in headwater loading characteristics were marked particularly by effects of vegetable organic loading, believed to originate primarily from fallen leaves. OGS serving residential contributing basins showed very high organic loading rates, at 9% and 20.7% by weight for the two basins sampled, compared to organic loading of about 4% for the two basins serving non-residential areas. Particle size distribution (PSD) analysis of the residential OGS also showed a significant increase in fine particle sizes (‘passing #200 sieve’) captured by the device, which Project analysts believe may reflect a local positive bias in the fraction of fine particulate loading from organics (leaves). On the other hand, the PSD of the particulates captured by the non-residential devices matches quite closely that of average Anchorage storm water, suggesting very high removal efficiencies, even for finer particulates, for the sampled devices. This removal efficiency does in fact approximate the ideal performance rate for the family of devices to which the tested devices belong, but such ideal performance in the field is suspiciously anomalous. Rather the ideal performance implied for the two non-residential OGS$_h$ sampled in 2012 may more likely reflect a probability that Anchorage’s current outsized design storm yields significantly oversized devices.

OGS Basin [CM1]	Basin Area (sq. ft)	Total Curb miles in basin	Basin Type	OGS Unit Model	Time since last cleaning (years)	Estimated Volume of accumulated sediments (cubic feet)*	Estimated dry weight of sediments (lb)	% passing #200 sieve (75 micron)	Organic Content of sediments
Old Seward and 74th Ave	770,000	.82	Arterial	STC 3600	0.70	11	1656 lb	10%	3.9%
Juneau Street N. End	4,568,000	7.17	Residential	STC 13000	1	60	9034 lb	33%	20.7%
Tudor Rd West of Lake Otis	400,000	.57	6 lane arterial with divider	STC900	1.85	6.75	1016 lb	17.2	4.4%
Mears Middle School 100th Ave and Bayshore Dr.	447,600	1.07	School Parking area	UK. "T" baffled tank	0.85	4	602 lb	34.9	9%

Figure 0.4 2012 OGS Field Sampling Results

In fact, though, OGS_h, even designed at the smaller—and more appropriate—local 90th percentile flow rate as identified in this Project, are expected to perform very well at Anchorage. This Project commissioned a specialty laboratory to perform full-scale testing under local street dirt washoff and runoff conditions (as specified by this Project) of an OGS_h model commonly used in Anchorage (Appendix B.1). Particle removal performance at various flow rates and particle sizes for the select model were calculated and scaled to the specific device using a Péclet number (a ratio of settling process to turbulence as reflected in flow and settling rates and device geometry). Recent national studies suggest hydrodynamic OGS lend themselves well to scaling the known performance of a single device to different sized devices within the same family of devices, using this approach. Such scaling is useful as a design tool where, of course, many different sizes of devices may be selected but only a few have been tested. The availability of standardized device performance tests that many manufacturers have completed of one or more of their devices may then be related to a Péclet number, which in turn may be suitable for application to the manufacturer's entire family of devices.

Laboratory testing of the select OGS_h for this Project suggested very high removal rates for the select device under Anchorage conditions of flow and street sediment character is possible (Figure 0.5). Results of this test demonstrate that removal of 40% of all particles equal to or larger than 20 microns is attainable at flow rates generated at or below the median of the annualized 90th percentiles of Anchorage rainfall intensities (i.e. for approximately 90% of all annual rainfall runoff volumes) using the family of OGS_h devices represented by the tested device, at least under ideal laboratory conditions. Other OGS manufacturers may provide devices having better removal rates, but based on the testing performed by this Project the removal rate specified above appears practicable.

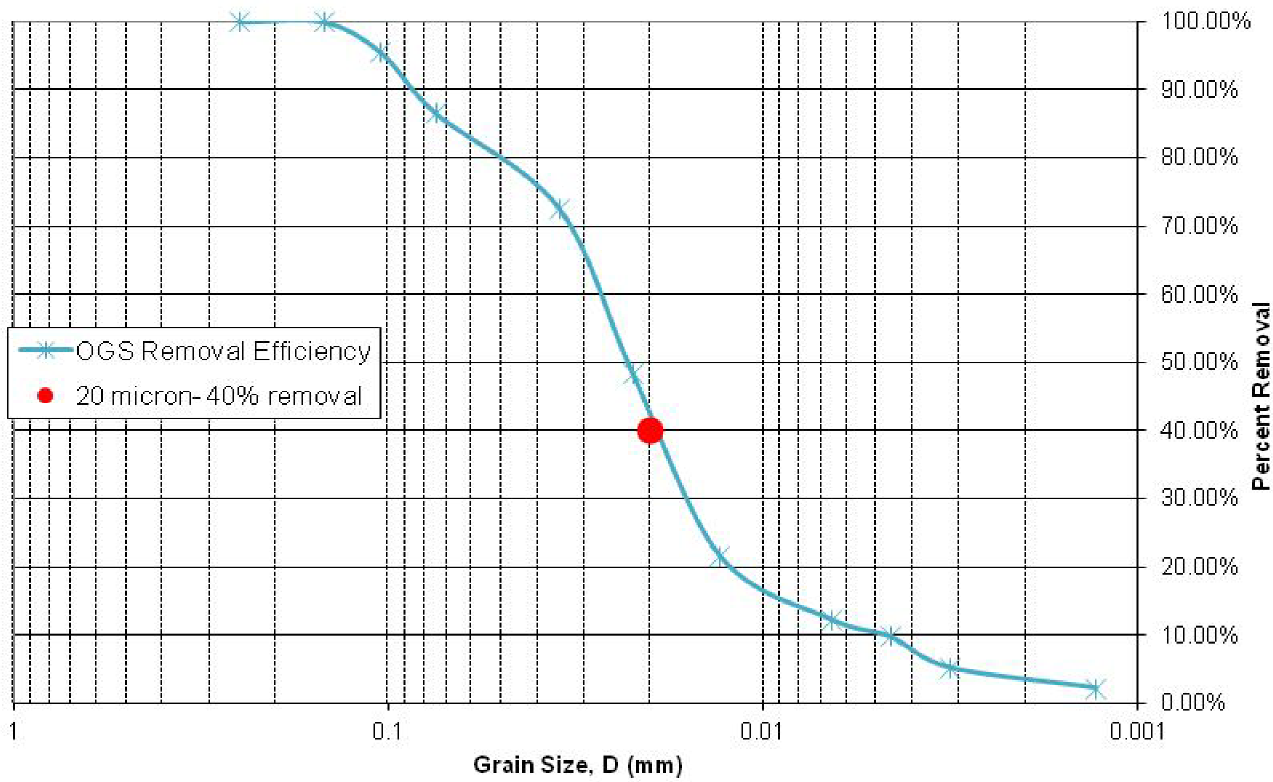

Figure 0.5: Anchorage design performance curve for select OGS_h

Field observations carried out in 2012 suggest other factors in addition to particulate removal design may play important roles in OGS performance. Interestingly one of these again reflects on the organic loading present in some Anchorage basins, especially from fine fibrous organics for which comminuted leaves and grass are suspected to be the primary sources. Though comminuted, some significant fraction of these organics retains a fine fibrous form. In several instances these fibrous materials have been observed to promote plugging of screen-type devices which could result in significant loss of efficiency. Flood flow bypass presence and design was also observed to play a prominent role in OGS performance. Optimally OGS should operate like off-line devices, similar to catchbasins. Complete absence of an effective bypass results in scour at flood flows, no matter the type of OGS.

Sedimentation Basin 2012 Performance

This project evaluated Anchorage sedimentation basin performance from a sum-of-loads pollutant removal perspective and then related that seasonal performance to a range of design factors through separate storm-by-storm analysis of basins' hydraulic efficiencies. Both analyses depended upon monitoring and comparison of paired influent and effluent flow and particulate flux using continuous NTU measurements as a surrogate for total suspended solids. Analysis of NTU and grab samples of suspended sediment concentration yielded R^2 values of about 0.7 (suggesting about 70% of the predicted relationship between NTUs and suspended sediment is due to a correlation between these parameters—i.e., the Project correlation of these variables can be considered 'true' with an error of about ±30%). This suggests a probable error of about 30% in Project estimates may result from the correlation approach used to obtain those estimates.

Data completeness may also affect Project results. Estimates of influent loading to each sedimentation basin were dependent upon data from a single influent station so that these estimates were affected only by the data completeness and other error introduced at that station alone. Data completeness at all influent stations was exceptionally good (about 90%) for both seasonal periods so that seasonal influent loading estimates are expected to have probable error that approximates that of the correlation error. However for paired station comparisons (sum-of-load treatment and hydraulic efficiencies), storm-synchronous data from two stations (from each influent and effluent pair) had to be available and valid. Percent completeness for use in performance estimates was reduced as a result. Nevertheless, when data is stratified and assessed more narrowly for those periods during which mass flux through the basins was more likely to occur, measures of completeness improve. Data representativeness—the degree to which measured relationships (NTU, TSS) values actually reflect real conditions also add uncertainty to performance estimates. For influent stations, representativeness is believed to be high. Sampling efforts, on which NTU/TSS correlations rely, were triggered by conditions at influent stations, typically on the rising limb of a storm. For effluent stations, however, correlations are likely to be biased by sampling data reflecting predominantly low-flow/low-TSS conditions. Despite these uncertainties, investigators believe that the removal efficiencies tabulated in Table NN present useful planning-level estimates of overall sum-of-load particulate treatment efficiencies for the 2012 spring and summer seasons and provide helpful insight to the relative performance of the three test basins.

Summary inspection of the 2012 performance results for the three Project sedimentation basins provide important insight into Anchorage's storm water quality treatment train (Table 0.2). The first of these is the size of measured seasonal influent loads measured at each of the sedimentation basins relative to their respective estimated total seasonal washoff loads. The influent mass measured for the 2012 summer season at each of the three test basins was approximately equivalent to 70% of the estimated total seasonal washoff load for their contributing basins. This seems reasonable, given that during the 2012 summer season none of the contributing basins had operating headwater OGS controls in place and their total respective washoff loads would therefore be available for transport to the sedimentation basins. Even given the probable error in Project measurements, these findings suggest several important system and design conclusions. First, the large fractional loadings observed at the three test basins in 2012 clearly illustrate the importance of designing downline controls in context with the entire headwater system as it exists in-place, or, conversely, planning and designing multiple controls in series. Project observations of Anchorage OGS_h in 2012 suggest that had similar properly designed and maintained OGS been in place above any of the test sedimentation basins, loadings at the test basins would likely have been reduced to a very small fraction of what was actually measured in 2012. Secondly, the 2012 data also suggests that this Project's estimates of seasonal washoff loading from Anchorage urban basins may approximate actual conditions, at least for 2012. Nevertheless, the apparent conflict between these findings and the several different street sediment loading measurements made by different permittee agencies further highlight a need to better resolve actual street sediment loading and washoff characteristics for Anchorage basins.

Loading differences between summer and winter seasonal periods are also prominent in the 2012 data, with spring seasonal influent significantly less than that of the summer seasonal period for all basins. These differences, occurring despite much larger sediment loads present on streets and parking lots during snowmelt runoff, emphasize the important effect that sweeping has on dirt mobilization as a result of fining and concentration of street dirt along gutter pans. The particular role that streets play in loadings at treatment basins is further underscored when influent loads for the Project basins are compared to total area, impervious area, and total number of curb and gutter miles within each associated contributing basin (Table 0.2). The fact that influent loadings at the Minnesota sedimentation basin equals or exceeds that of the C Street basin despite its smaller total area, is better understood after noting that the total curb and gutter miles and percent impervious area contained within its associated contributing area are the same as or exceed that of the C Street contributing area.

Table 0.2: 2012 particulate loading and treatment at Project sedimentation basins

2012 SEDIMENTATION BASIN LOADING									
	C StreetBasin			Minnesota Basin			Meadow Street Basin		
	C st In	C st Out	% capture	Minn In	Minn Out	% capture	Meadw In	Meadw Out	% capture
Measured INFLUENT Load all valid influent data, cyds (lbs)									
Spring 3/21/12-5/23/12	8			12			2.5		
% complete	99%			100%			95%		
Summer/Fall 5/24/12-10/10/12	22			25			10		
% complete	100%			90%			100%		
Measured EFFLUENT Load paired data only, cyds (lbs)									
Spring	5.3	2.9	**45**	8.6	5.7	**68%**	1.4	1.2	**16%**
% completeness- paired records/total records	65%			80%			65%		
Summer/Fall	13.5	4.6	**66%**	6.8	3.8	**45%**	2.9	2.3	**20%**
% completeness- paired records/total records	81%			70%			34%		
% completeness-rainfall captured/rainfall total	93%			80%			45%		

* - % capture represents estimated captured mass; value shown is estimated fraction of total seasonal 2012 INFLUENT load captured

Table 0.3: 2012 contributing basins landuse characteristics

	C Street	Minnesota	Meadow
% Impervious	50	66	59
% Pervious	50	34	41
Total Area, acres	843	579	377
Curb & Gutter, miles	12.6	12.3	5.5

Treatment removal rates shown in Table 0.2 reveal the importance of particular relationships between treatment basins characteristics and their associated contributing areas, as well as the effects of key design factors known to significantly influence the performance of these types of water quality controls. The three Project sedimentation basins were selected specifically to evaluate the effects of a range in values of key factors including: the shapes and layouts of the treatment basins; the aspect and location of inlets and outlets; the location and character of distributary features within the basin including weirs, islands and constructed wetlands; and the ratios of length to width (L:W), total treatment basin volume to total runoff volume (V_{basin}:V_{runoff}), and the total surface area of the treatment basin to the mean storm flow rate (Q_{runoff} :V_{area}or the hydraulic loading rate, HLR).

A number of investigators have shown that the cumulative effect of all of these factors can be approximated by a hydraulic efficiency measure, λ, based on the combined measures of a treatment basin's effective treatment volume and the degree of mixing taking place within the basin. The hydraulic efficiency, λ, can be approximately equated to a single measure of a basin's overall mixing or turbulence, 'N' that has been related through modeling and empirical investigations to overall particulate removal performance for different configurations of key factors within basins. Data collected for this Project was used to derive the hydraulic efficiencies of the 2012 test basins through analysis of individual storm hydrographs and pollutographs. The derived λ values were then converted to N values and these compared to literature-identified basin configurations that had similar N values. These comparisons provided this Project's investigators a means to assess 2012 measured performance against Anchorage test basin configurations. Results from these assessments in turn provide the basis for recommendations for future modifications to design and configuration criteria that will optimize Anchorage systems. The hydraulic efficiencies factors derived from 2012 storm data and the basin parameters measured for each Project sedimentation basin, as well as measured and calculated (based on a calibrated probabilistic design method) performance rates for 2012 are listed in Table 0.4: 2012 Project basins' parameters and basin performance ratings.

The performance ratings shown in the table reflect the effects of a range in basin configurations and design parameters implied by the individual test sedimentation basins. For ease of discussion design parameters expressed in the individual basins can be grouped into hydraulic, volumetric, and geometric factors. Hydraulic factors reflect primarily dynamic treatment, or particulate removal taking place in immediate response to various flow rates. This factor is reflected in the hydraulic loading rate (HLR), or the ratio of the mean storm flow rate to the total surface area of the sedimentation basin. Not surprisingly the HLR of all three basins is similar, as hydraulic loading rate has been the primary basis upon which all Anchorage sedimentation basins have been designed.

Table 0.4: 2012 Project basins' parameters and basin performance ratings

Variable Name	Symbol	Units	C street	Minnesota	Meadows
Basin Hydraulic Measurements					
Length to Width Ratio	**L:W**	-	3.25:1	2.5:1	4:01
Average Basin Depth	**D**	ft	5.5	3.4	5
Geometric Surface Area	A_{QR}	ft^2	157,726	49,059	25,959
Effective Sed. Basin Surface Area	AE_{QR}	ft^2	41,403	17,662	2,142
Geometric Total Basin Volume	V_B	ft^3	537,734	116,720	63,734
Effective Total Basin Volume	V_{EB}	ft^3	181,485	70,908	16,252
Basin Hydraulic Calibrations (adjustment factor)					
Turbulence Factor	**N**	-	4.29	1.77	1.4325
2012 Basin Hydraulic Calculations					
Mean Runoff Event Stage	E_{QR}	ft	82.178**	36.377	112.691
Mean Storm Runoff Volume	V_R	ft^3	296,082.70	271,174.20	151,298.20
Dynamic Sed. Basin Storage	V_S	ft^3	28,184.00	17,969.00	9,878.00
Dynamic Volume Ratio	V_S/V_R	-	0.095	0.066	0.065
Runoff Flow In	Q_R	ft^3/s	11.4	9.6	5.8
Peak Flow Out	Q_o	ft^3/s	8.8	8.6	5.2
Hydraulic loading rate	**HLR**	ft/s	7.20E-05	1.95E-04	2.20E-04
Effective Hydraulic Loading Rate	**HLR**	ft/s	2.70E-04	5.40E-04	2.69E-03
Effective Sed. Basin Surface Area	A_{QR}	ft^2	41,403	17,662	2,142
Effective Total Basin Volume	V_B	ft^3	181,485	70,908	16,252
Quiescent Volume Ratio	V_B/V_R	-	0.613	0.261	0.107
Basin Performance					
2012 Calculated Dynamic Removal	**-**	%	58.69%	45.74%	24.03%
2012 Calculated Quiescent Removal	**-**	%	30.50%	14.27%	6.43%
2012 Calculated Total Removal	**-**	%	71.29%	53.48%	28.92%
2012 Sum of Loads Measured Removal	**-**	%	66%	45%	20%

- C street mean runoff event stage variable is based on the lower pond elevation
- All parameters reflect calculations based on 2012 measurements and storm records
- All calculated basin removal estimates based on probabilistic method (Driscoll, 1986) applied to 2012 rainfall season

However, though a design approach based on HLR analysis is useful as a basic methodology, it does not address all critical design factors and certainly does not appear to explain differences in sedimentation basin performance observed in the 2012 Project data. Volumetric design parameters significantly influence quiescent—non-storm flow—treatment, which particularly influences effectiveness in removal of smaller particles. Addition of volumetric design approaches can improve on the use of the HLR approach alone, particularly in the case of

Anchorage where intensities and storm volumes are relatively small. The quiescent volume ratio or the ratio of the volume of the entire treatment basin to the volume of total runoff of the mean storm is a key design factor for quiescent treatment. It's likely that the notably good relative performance of the C St basin is due in significant part to that basin's large storage volume relative to the volume of the average storm runoff event entering it.

Basin geometry is the last principal set of factors used in this Project as a basis to assess sedimentation basin performance. In the last decade, researchers have shown a strong correlation between basin shape and inlet/outlet configurations and overall basin performance. Optimum configurations encourage flow distribution across the full surface and volume of the basin, and discourage concentrated, channelized flow. As outlined earlier, investigators have related various basin configurations to a surrogate measure of hydraulic efficiency: the mixing or turbulence factor, N. This factor is qualitatively related to overall basin performance with N values of 3.3 and larger representing basins having good hydraulic efficiencies and values of less than 1.5 representing basins having poor hydraulic efficiencies (Figure 0.6).

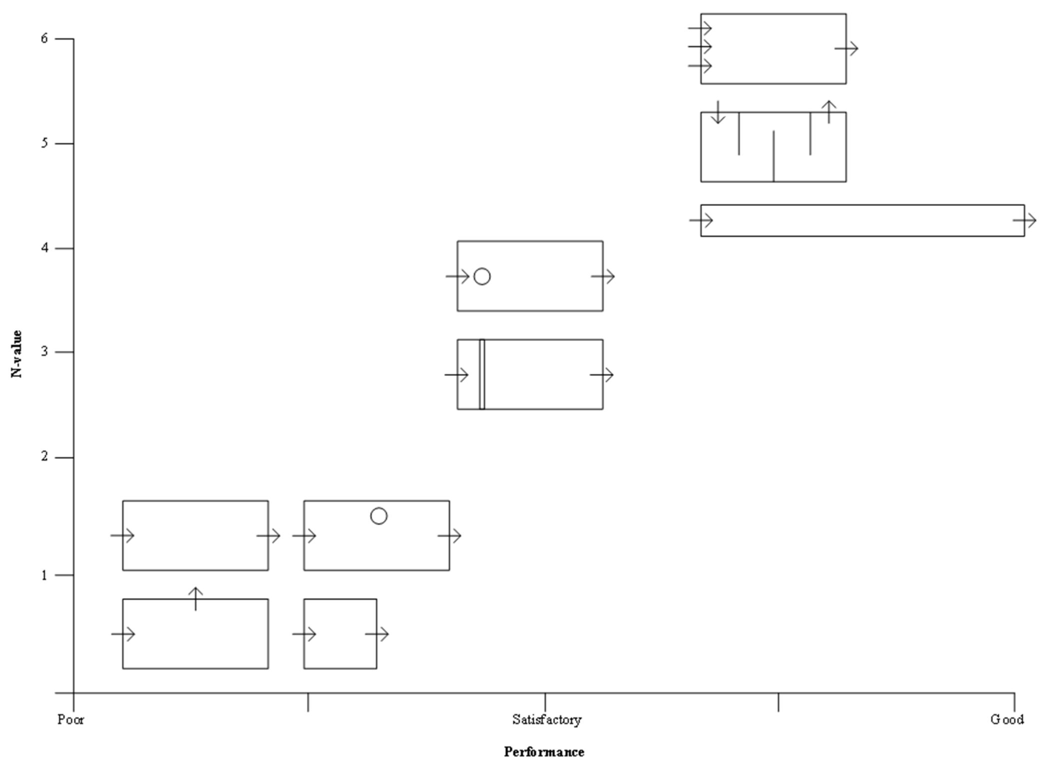

Figure 0.6: Idealized basin configurations and associated 'N' values

Using 2012 storm and pollutograph data, N factors were derived from calculated hydraulic efficiencies for each of the Project test sedimentation basins. Both 2012 data-based and empirical estimates of performance are reported in Table 0.4. Based on classification schemes reported in the technical literature, the calculated turbulence factor for the C Street basin reflects good hydraulic efficiency while those of both the Minnesota and Meadow Street basins reflect moderate and poor hydraulic efficiencies respectively. The differences in the measured 2012 (based on Project data) and empirically-derived (based on N valuations) performance between the three test sedimentations basins find clear expression both in the measured design factors as well as in the configurations of each of the basins (Figure 0.7 through Figure 0.9).

The overall poorer performance of both the Minnesota and Meadow Street basins revealed by 2012 storm measurements is reflected in these basins' basic design factors as well as their estimated N values. The HLR for both basins predicts a design capability of removing particles only to about 60 microns at the mean annual rainfall runoff event. Their low quiescent volume ratios of 0.2 also anticipate poor performance for small particle capture. Still, the actual level of performance measured for these basins in 2012 does not seem to be completely predicted by these design factors alone, particularly for the Meadow Street basin. The remainder of observed performance may be predicted by overall basin configurations, though different characteristics are believed to drive the observed performances of each basin.

The Meadow Street basin has an off-center inlet/outlet aspect with a deep central channel and a fringe marsh zone that may result in significant 'dead zones', channeling, and short circuiting (Figure 0.8). In effect, such short circuiting significantly reduces already relatively small length:width ratio and treatment surface area. Project investigators believe the notably poor spring 2012 performance at Meadow Street is particularly due to significant loss of treatment surface area and volume as a result of ice cover.

Project data suggests performance at Minnesota was moderate to poor over the 2012 water year, though still significantly better than that of the Meadow Street basin. The hydraulic loading rates and volumetric measures for these two basins are approximately the same but basin configuration is significantly improved at the Minnesota basin. The Minnesota basin includes multi-pool design, banded (full-width) constructed wetlands and submerged weirs to promote uniform flow distribution and plug flow, all of which have clearly improved treatment at the Minnesota basin relative to that of Meadow Street. However the calculated turbulence factor, N, for this basin suggests treatment is sharply reduced as a result of basin geometric factors. Observation of performance of this basin in the field in 2012 as well as its surface expression (Figure 0.9) suggests a number of current conditions that lead to this basin's low N value and poor 2012 performance.

This sedimentation basin has a much lower average depth than the other two test basins. It also includes a submerged weir of coarse washed rock across its full width at the lower end of the upper settling pond. The weir surface is less than a foot below the normal pond stage, and the pond is suspected of freezing to the weir surface along most of its length during winters. Over the course of the basin's operation, pond ice extending from the water surface to the top of the weir may have obstructed flow over most of the weir's length and promoted development of a preferential low-flow path across the weir at its eastern end. Spring thaw would tend to open pond ice initially along the warmer winter flow path, promoting further development of a preferential low flow channel through the constructed wetlands. Infilling would tend to occur preferentially away from the channel and in fact has occurred along much of the length of the west side of the wetlands. As a result, a pronounced low-flow channel has developed along the entire east side of the basin that appears to direct most low flows, winter and summer, preferentially along its east bank. Short circuiting at this basin is sufficiently pronounced to be visible as surface water flow along the entire channel length as it passes through the constructed wetlands. It does appear that some treatment is still provided by the wetlands, however, as a result of deep lateral open-water channels interfingering from the low-flow channel into the constructed wetlands and redistributing flows. Differences in performance between summer and winter at this basin may be due to a combination of loss of overall treatment volume and surface area from pond ice cover as well as reduction in treatment provided by the wetlands from ice blocking the lateral distributary channels. In any event, it appears that not only is the valuable distributary function of the weir lost with development of winter ice cover, but it also appears that the weir may have helped create the low flow channel that now seriously reduces the effectiveness of this basin both summer and winter. Conceptually, a solution may lie in creation of a deep, full-width open-water distributary channel on the wetland side of the rock weir with multiple fixed conveyances sized to more evenly distribute winter and spring snowmelt flows from the primary settling pond across or through the rock weir to the distributary channel on the other side. Use of a number of lateral, deep-water distributary and/or low-flow channels is common in constructed wetlands designs but may need to be modified to meet cold-region conditions.

Project 2012 data for the C Street sedimentation basin suggests moderate to good performance for this basin , but with spring snowmelt runoff performance noticeably reduced from its summer performance, similar to the Minnesota basin. The C Street basin's geometry is improved over that of the other two test basins. Its hydraulic loading rate suggests this basin should be able to remove particles to about 20 microns during a mean annual rainfall runoff event, substantially improved over that of the other two basins. The C Street basin also has a fair quiescent volume ratio of a little over 1, which, given Anchorage's mean storm separation time of about 78 hours, can provide for significantly improved removal of finer particles (compare calculated estimates of quiescent removal for the three basins in Table 0.4: 2012 Project basins' parameters and basin performance ratings). These improvements in the basic design characteristics of the C Street basin are clearly reflected in a significantly improved performance for both the snowmelt and rainfall events relative to the other two test basins.

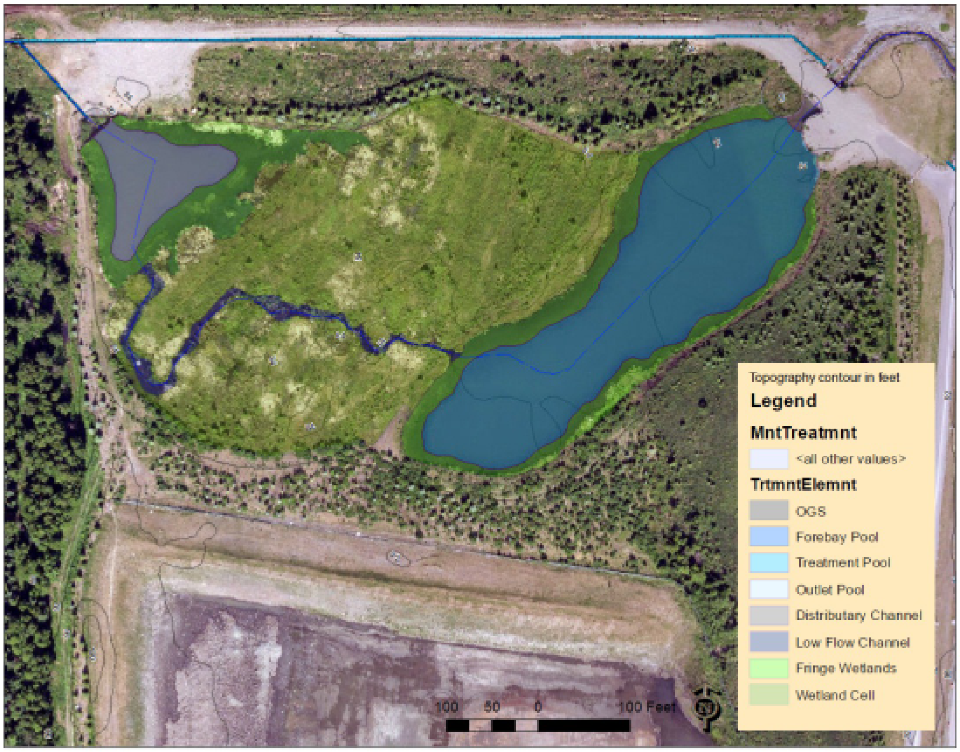

Figure 0.7: C Street Sedimentation Basin

Figure 0.8: Meadow Street Sedimentation Basin

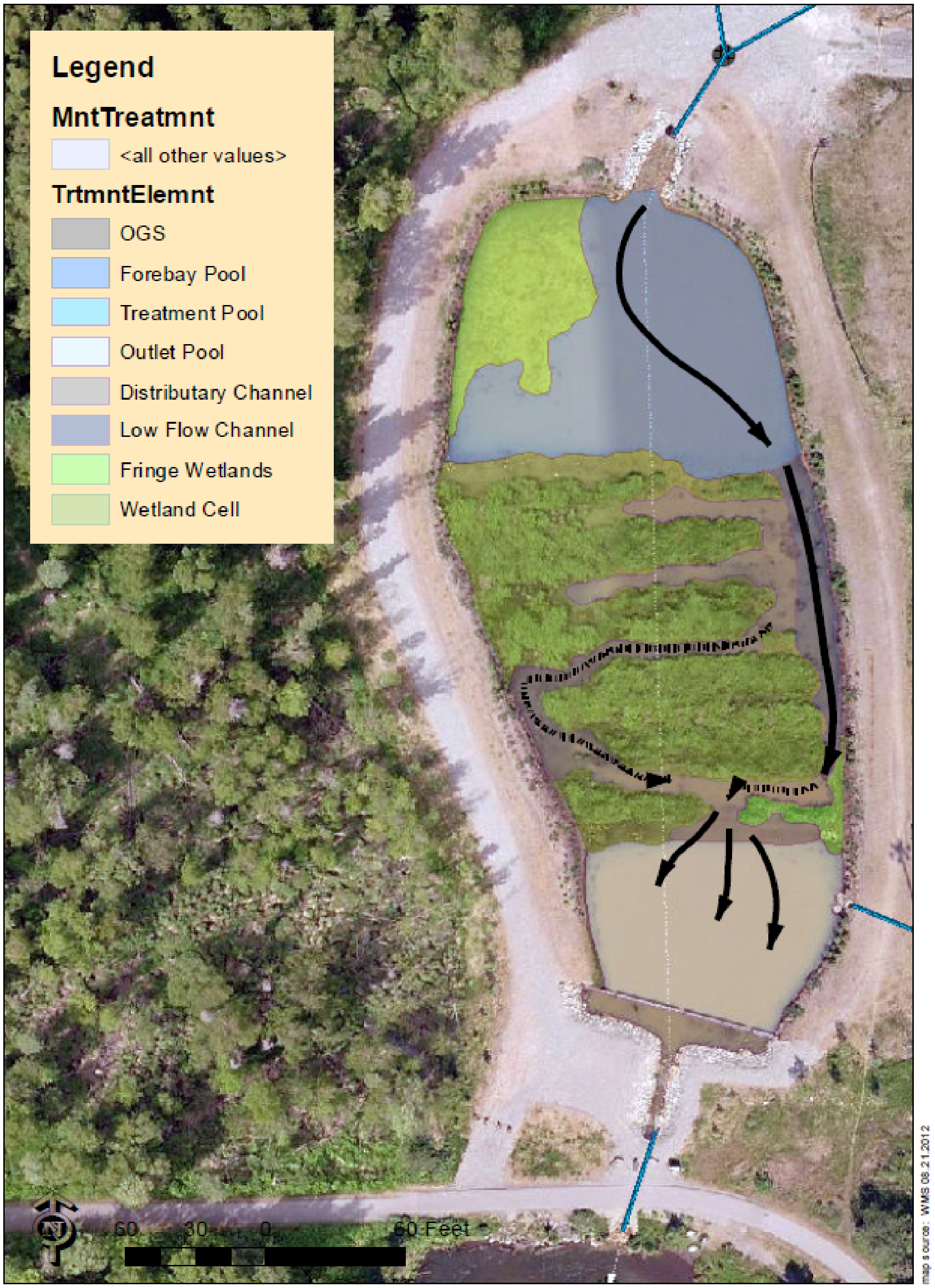

Figure 0.9: Minnesota Sedimentation Basin

However similar relative relationships between spring and summer performances within each basin for both C Street and Minnesota suggest that these two basins share some design flaws. Like the Minnesota basin, the C Street basin incorporates a low emergent gravel weir along the lower end of its upper settling pool to distribute flows from the pool to its constructed wetlands. Since initial construction, a low-flow channel has become established across the weir and over the wetland surface (Figure 0.7: C Street Sedimentation Basin), probably through similar ice-cover influences as those suspected of having occurred at the Minnesota basin. Similar to the Minnesota basin, shallow distributary channels initially constructed in the C Street wetlands have also infilled so that spring snowmelt runoff to the basin is now primarily short-circuited through the low-flow channel. For the C street basin this significantly reduces the treatment function available from the constructed wetlands during the spring runoff season. During summer, the sinuous character of the C Street low-flow channel helps to redistribute flows across the wetland surface through over-bank flows. However even then full functional treatment from the weltands probably is not available except during the largest flow events.

As discussed earlier, these observed weir, low-flow channel, and distributary, effects on winter and summer performance at the Minnesota and C Street sedimentation basins have significant implication for Anchorage design criteria for these types of devices. With the initial steep design slope of the C Street constructed wetlands, and the channelization and infilling that has occurred across the wetlands since its start of operation, as much as one third of the designed effective treatment surface area of this sedimentation basin may by now have been lost. This is born out by the Effective Hydraulic loading rate as calculated in table NN. Interestingly, if this is the case, the basin's effective HLR is closer to that of the other two test basins, and its observed superior performance may be a product much more of its better volumetric character and its excellent inlet and outlet aspect than of its large constructed wetland surface. In any event, like the Minnesota basin, improvement in the C Street sedimentation basin would require stabilization of the gravel weir separating the upper pool and the wetlands (the existing low flow channel position is simply the happenstance location at which weir gravels were finally eroded through). However in the case of the C Street wetlands, due to the steepness of the constructed wetlands, substantial reconstruction would have to be done to establish more effective low-flow and distributary channels. In construction of such a system at either the C Street or Minnesota systems, designers will have to carefully balance low-flow channel sinuousity and length against losses in treatment surface area (due to elevated separating berms) or system stability (due to potential for breakthrough and re-creation of a steepened and shortened channel). Any solution must also consider the ice cover problems that will be inherent in any cold-region design.

Finally, 2012 performance assessment of Anchorage sedimentation basins included (in addition to flow, suspended sediment concentrations and NTUs) sampling for a range of water quality chemical and physical parameters including temperature, pH, conductivity, DO, BOD, fecal coliform, and petroleum hydrocarbons. Fecal coliform, DO and BOD samples were not collected systematically and are included to provide only a general indication of system conditions during Project data collection efforts. Petroleum hydrocarbon loading at each of the six stations was estimated though measurement of cumulative loading using Gore Sorber passive cumulative devices (pcd's). These devices were installed for prolonged periods of time as part of station sensor packages, collected periodically, and submitted for laboratory testing. Based on accumulated mass, average concentrations of select petroleum hydrocarbon species were calculated. Results from all pcd's suggest hydrocarbon loading in Anchorage storm water runoff is not significant at any current regulatory threshold (Table 0.5). Additional summaries and discussion of all sampled water quality parameters are available in Appendix C.4.

Table 0.5: Table IV.A results for 2012 test basins

		C st in	C st out	% change	MINN up	MINN down	% change	MDW up	MDW down	% change
Gore Sorber Results µg/l										
	DRO Spring	1.16	1.25	-7.33%	1.24	1.11	10.53%	1.29	1.33	-3.50%
	DROSummer	1.33	1.01	23.77%	0.70	0.69	1.43%	1.61	0.92	42.99%
	GRO Spring	0.80	0.86	-8.18%	bdl	0.44	NA	0.64	0.57	11.02%
	GRO Summer	0.19	0.48	-150.00%	0.28	0.14	50.00%	0.15	0.14	6.67%
	TPH Spring	1.32	1.43	-8.37%	1.27	1.15	9.09%	1.38	1.39	-1.09%
	TPH Summer	0.66	1.07	-61.36%	0.77	0.74	3.92%	1.63	0.93	43.08%
	BTEX Spring	2.57	1.30	49.42%	2.69	2.27	15.64%	1.59	0.48	70.13%
	BTEX Summer	0.56	0.42	24.32%	0.74	0.57	23.13%	0.71	0.15	78.72%
pH										
	Spring	7.03	6.56	6.68%	6.07	6.74	-10.98%	5.57	5.92	-6.35%
	Summer	13.68	14.61	-6.79%	6.91	14.89	-115.38%	13.53	14.20	-4.94%
DO (%)										
	Spring	90.70	84.27	7.09%	91.10	82.73	9.18%	83.33	78.47	5.84%
	Summer	78.60	76.17	3.09%	82.37	77.63	5.76%	77.24	75.10	2.77%
BOD, mg/l										
	Spring	6.16	6.02	2.27%	7.47	6.21	16.87%	4.75	6.40	-34.74%
	Summer	6.38	U	NA	7.14	3.52	50.77%	6.82	5.39	20.91%
Fecal Coliform, FC col/ml										
	Spring	240.00	42.00	82.50%	310.00	246.00	20.65%	18.00	96.00	-433.33%
	Summer	562.00	248.00	55.87%	2996.00	1608.50	46.31%	17089.00	2547.00	85.10%

Conclusions and Recommendations

Results from the 2012 Project evaluation of Anchorage OGS and sedimentation basins suggest the following core recommendations for modification to planning and design strategies for application of these types of devices within the Municipality:

- Plan and design all water quality controls within a treatment train context.
- Apply water quality design storms appropriate to Anchorage.
- Identify and implement practicable maintenance SOPs to support designs.
- Apply 90th percentile rainfall intensity and waste storage criteria to OGS design.
- Apply probabilistic and synergistic design criteria to sedimentation basin design.
- Design for and assess performance using seasonal sum-of-loads methods.

Each of these is briefly addressed below.

System Water Quality Treatment Strategy

This Project strategically addresses evaluation of performance of OGS and sedimentation basins at Anchorage from a system perspective. Planning and design for modern storm water management typically views water quality treatment in terms of practices and controls applied in series along a drainage system network. Such a series of water quality management practices has been usefully conceptualized as a 'treatment train'. Performance relationships between the serial elements are complex and models have been developed that provide some limited opportunity to estimate overall treatment performance of all related elements. However, the basic underlying principle of a treatment train is that not only does each individual element capture and treat specific pollutant types but also transmits a modified pollutant load to the next treatment element in the series. Effective and efficient design (and overall performance analysis) requires a sound understanding of this pollutant cascade.

This context also explicitly recognizes that all storm water treatment controls work best at low flows. The size of runoff flows anywhere is primarily determined by the amount of precipitation and the size and character of the contributing area. At semi-arid Anchorage, cyclonic storms yield prolonged rainfall with multimodal peaks of relatively low-intensity, so that most runoff is expressed as relatively low flows. This is particularly true at 'headwater' locations, i.e., where the smallest contributing surface areas first discharge runoff to the storm drainage system through many distributed inlet points. Given that best efficiency is achieved at lowest flows, controls applied at the inlets and across the headwater contributing surfaces will optimize water quality treatment, and particularly so at Anchorage. This Project specifically recommends:

- <u>Prioritize headwater control applications</u> to optimize practicable efficiency of overall system performance. Both public and private water quality controls in headwater positions maximize treatment by leveraging the low-flow conditions that these devices typically experience.
- <u>Design OGS and sedimentation basins in context with headwater conditions</u>. Headwater conditions include seasonal storm design, pollutant loading, runoff and washoff, and numbers and efficiencies of headwater controls in place. Effective selection and design of practicable down-line devices like OGS and sedimentation basins require consideration of the effects headwater conditions have on them.

Headwater Factors

Planning and design of water quality (WQ) controls at Anchorage should be done in context with their position in the treatment train series. Storm drainage ‘headwater’ devices (i.e., controls at or near inlets, including particularly catch basins for urban piped systems) provide efficient treatment of pollutants when properly designed and maintained. The quantity and quality of influent to ‘down-line’ devices such as OGS and sedimentation basins are affected by the performance of the headwater controls present. Adequate design and performance must account for site-specific headwater conditions. Primary headwater conditions to be addressed include:

- Seasonal street and parking sediment loading and variability
- Headwater controls placement, geometry and performance efficiencies
- Headwater controls maintenance practices and schedules

This Project has estimated and reported these basic headwater conditions. However Project results clearly indicate additional work is required to adequately resolve disparity in street sediment loading estimates and to characterize geometry of catchbasins in terms of performance efficiencies at Anchorage. Specifically this Project recommends:

- Resolve Anchorage street and parking sediment loading relative to season, street types and conditions, and common street maintenance practices, including street sweeping.
- Resolve optimum catchbasin geometries and distribution frequency for Anchorage including sump and invert geometry based on local street and parking sediment loading and associated seasonal runoff conditions. Catchbasin and private headwater OGS should have sump storage capacities about 1.2 times the anticipated seasonal loading to support an annual inspection and cleaning schedule.
- Resolve optimum maintenance SOPs for headwater controls based on optimized catchbasin geometries and distributions for Anchorage.
- Design all headwater devices, including catchbasins, off-line. Small on-line headwater devices are particularly subject to scour and would require maintenance schedules tied to storm interevent times to be effective. For Anchorage conditions this would suggest a catchbasin inspection and cleaning schedule on the order of two weeks and is considered impracticable.
- Address fibrous organic loading through best management practices: comminuted fibrous organic loading from leaves and urban lawns appears to represent a large local loading to Anchorage piped storm drain systems. This pollutant loading is poorly treated by catchbasins and marginally treated by density settling devices like OGS and sedimentation basins. Some evidence exists that the fine fibrous organics also create performance problems for screened types of OGS_h, making annual screen cleaning imperative. Use of downline density settlement controls may also require modifications to achieve adequate removal rates. Most efficient removal may be achieved by source controls and is recommended as a primary treatment practice.

Water Quality Treatment Design Storms

Appropriate design storm characteristics are critical to selection and application of practicable controls, or accurate estimation of control performance. Most water quality treatment processes perform most efficiently and practicably (effective treatment/low cost) at low flows. Thus standard water quality treatment design strategies focus on treatment of smaller runoff flows (lower intensity rainfall) while bypassing larger flows, with the treatment rate threshold typically set near the point of asymptotic increase in ranked rainfall intensities. Rainfall generally follows a gamma distribution function with most storm volume generated for many geographic areas at rainfall intensities at or below the mean annual 90th percentile (which is also often at or near the point in asymptotic increase in rainfall intensities). As a result, treatment to the 90th percentile is frequently selected as a performance standard for flow-rated water quality designs. That is, flows below the annual 90th percentile intensity are treated and those larger are bypassed. This standard is appropriate for water quality design of OGS and other devices that have no significant storage, and is recommended by this Project for use in Anchorage designs.

For devices with substantial storage (as for sedimentation basins), designs based solely on flow rates do not adequately address opportunities for 'quiescent' treatment that storage types of devices offer. Consideration of quiescent treatment during no-flow or very low flow conditions is important for efficient design of storage-type water quality controls at Anchorage for a number of reasons. These include very low average rainfall intensities (with the mean at 0.03 inches/hour), relatively long storm separation times (with the mean at 78 hours), and relatively fine particulate loads commonly transmitted to Anchorage end-of-pipe treatment systems where storage type devices (like sedimentation basins) are typically located.

Designing to achieve treatment goals based on dynamic and quiescent treatment is difficult, however, because of the high degree of variability in distribution of storms, and of rainfall intensities within each storm. To address these issues EPA sponsored work during early National Urban Runoff Program efforts that treats the "..variable nature of storm runoff..by specifying the rainfall and the runoff it produces in probabilistic terms, established by an appropriate analysis of a long-term precipitation record for an area." (US EPA, September 1986) This work provided detailed conceptual design approaches that characterized variable rainfall/runoff rates, volumes, durations and intensities in terms of their annual means and variations and used these values as a basis for design and estimation of long-term storm water pollutant removal, particularly addressing quiescent treatment. The design approach described in this method for treatment of particulates incorporates a short circuiting factor (referenced as the 'turbulence factor', N, described in this Project), thus making it robust for consideration of both storm and basin geometry design factors. Its use of annualized rainfall characteristics also makes it highly suitable for the annual sum-of-loads design and performance assessment approaches recommended by this Project.

It should be clearly noted that design or assessment based on peak (large) flows (or intensities) is neither feasible nor justifiable for water quality controls. This is intuitively clear when considering that as design addresses rainfall past the point of asymptotic increase in annual intensities, a similar asymptotic increase in device size must result (with very little dividend in the increased volume of runoff treated). Similarly overall device performance is not appropriately based on that observed for any one storm, but rather should be based on the range of storms as they regularly recur and vary over an annual seasonal basis. This is because, given the stochastic nature of rainfall (i.e., although high precipitation intensities do occur infrequently, they occur randomly and therefore can happen during even otherwise quite small storms), good design calls for bypass of incremental large peak flows, while capturing and treating most of the remaining smaller flows. Therefore, water quality designs are more appropriately set to treat the rainfall of an average annual storm (adjusted to account for variability) for design of devices with significant storage (e.g., sedimentation basins), or to treat at a flow rate equivalent to that generated at the median annualized 90th percentile of precipitation intensity for devices with no storage (e.g., OGS). This Project has performed statistical and synoptic analysis of the historic Anchorage precipitation record to identify these water quality design storms, and recommends their use in future design of OGS and sedimentation basins. Specific recommendations include:

- <u>Develop water quality-appropriate design storms</u> based on statistical and synoptic analysis of Anchorage precipitation and runoff patterns.
- <u>Adjust design storm volumes locally</u> through application of orographic factors.
- <u>Apply the median annualized 90th percentile rainfall rate</u> through use of tested or approved performance curves for design of flow-through (dynamic treatment only) treatment devices
- <u>Apply the mean- and the variation of the mean-annual rainfall rate</u> through use of sum-of-loads probabilistic methods for design of storage-based systems (both dynamic and quiescent treatment).

Anchorage OGS$_h$ Design and Performance

Given that headwater storm water loading and design storm conditions are adequately addressed, analysis completed during this Project suggests hydrodynamic OGS can be efficiently applied to Anchorage conditions. Hydrodynamic separators—OGS$_h$—are intrinsically more effective at capturing finer particle sizes than are the typical headwater devices installed along the Anchorage treatment train. Project field observations and laboratory assessment of a full-scale OGS$_h$ device type often installed in Anchorage systems suggest that these devices as installed can efficiently capture a significant fraction of the storm water particulate load. Design of OGS$_h$ to treat 40% of the 20 micron particulate storm water load at an Anchorage median 90th percentile rainfall intensity should be practicable. Performance of the an OGS$_h$ at this threshold will be dependent upon proper design and maintenance. Specific recommendations include:

- Design to remove 40% of 20 micron particles at the median 90th percentile of annualized Anchorage rainfall intensities.
- Use nationally-tested hydrodynamic separators with demonstrated capability to achieve the design treatment goal under Anchorage conditions. National testing made available by manufacturers for one device can be used to asses performance capacity of different-sized devices within the same family through application and scaling from a Péclet number derived from results for the tested device. Based on the manufacturer's standardized testing results, construct a removal efficiency versus Péclet number curve, and select an appropriate device size based on a minimum Péclet number of 3.
- Install controls off-line only. To achieve specified performance, flood-flow bypasses must effectively prevent any significant increase in head or flow-through velocity greater than that incurred by the treatment flow rate.
- Set sump storage at 1.2 times the annual headwater-transmitted load. Site-specific headwater loading and storm water mobilization can be estimated from results from this study, as adjusted to reflect treatment by in-place headwater controls. Sump recommendations are based on minimum annual cleaning.
- Design to maintenance schedules and needs. Sump sizing recommendations above are based on an assumption that all OGS will be inspected at least annually and cleaned when

sump storage reaches a threshold of 0.2 times sump capacity. Design should also address maintenance constraints including at minimum gated bypass controls to divert base flows for cleaning, incorporation of sloped sumps with low-friction surface coatings where cleaning access is constrained, and adequate space and utilities for maneuvering to pull and clean screens where any screened devices are specified.

Anchorage Sedimentation Basin Design and Performance

Project results suggest sedimentation basins, like OGS, can be effective at removal of particulates from Anchorage storm water, though their use may be constrained by available space in urbanized Anchorage and by costs to implement the large volumetric designs required for effective small particle treatment. In any event, current Anchorage design criteria for these devices do not adequately direct design. Particularly at issue is use of appropriate design storms, and identification and application of appropriate quiescent treatment and geometric design principles. Project results suggest implementation of the following changes to Anchorage design standards:

- <u>Design to remove ≥90% of 20 micron and ≥75% of 5 micron</u> spherical, non-charged mineral particles over the variation of the mean annual storm event. Perform designs using probabilistic analysis procedures and SYNOP storm statistics for Anchorage (detailed below and in this Project's documents).
- <u>Design for dynamic and quiescent treatment</u> with a V_B/V_R goal of >1. Quiescent treatment is dependent upon the relative volume available within the treatment basin. Project data and national research suggests quiescent treatment performance is considerably reduced at V_B/V_R ratios below 1.
- <u>Design basin geometry to achieve N > 2</u>. Research by others indicates that the Project 'turbulence' factor, N, can be approximately estimated for use in design applications through use of basin shape and aspect diagrams showing plan relationships of inlets/outlets to other design elements. Project documents reference several modeled and experimentally resolved diagram sets which can be used to standardize application of this factor to Anchorage designs.
- <u>Design tiered full-width treatment elements</u>. Include as primary design elements in sedimentation basins an initial settling pool, intermediate banded (full-width) wetlands, and a smaller outlet pool. Design the inlet pool as the primary settling pond and incorporate intermediate wetlands at minimum as a distributary device intended to minimize overall short circuiting. Design fixed-elevation full-width distributary weirs at each element transition. Design all pools with depths sufficient to prevent scour under ice cover at spring snowmelt runoff design velocities and to optimize spring snowmelt particulate removal. Avoid the summer short circuiting effects of fringe wetlands and the reduced HLR effects of ice cover along shallow-sloping pond margins.
- <u>Design low-flow winter channel(s)</u> across banded wetlands. Design low-flow channel(s) with connection to lateral distributary channels across wetland width. Wetland weirs and low-flow channels should mitigate for spring frozen wetland conditions.
- <u>Design all devices off-line</u> and/or set outlet/bypass to match quiescent design. On-line sedimentation basins may scour or develop preferential flow paths leading to short circuiting and poor long-term performance.

- Design sedimentation basin sites as public-use spaces. Sedimentation basin complexes, at locations throughout the country and worldwide, are commonly used as public open spaces. Of the three Project test basins, the two basins developed as multi-use park spaces were observed during Project activities to be actively policed by watchful nearby residents, park users, and law enforcement officers. The one Project basin isolated from public use by fencing was subject to repeated vandalism and property damage, including to Project instrumentation. Public safety at multi-use sites can be adequately provided by focused use of localized fencing or landscaping to limit access along potentially more hazardous basin elements.

Water Quality Treatment Design and Assessment Approaches

Finally, methods applied to the designs of any of these water quality controls should be matched to the process by which the device performs. As discussed above the most efficient water quality treatment design treats the large fraction of smaller flows and bypasses all less frequent but larger flows. However storm water runoff is stochastic. Accounting for these random runoff variations in water quality designs is difficult. To address these difficulties, this Project recommends water quality treatment designs be based probabilistically on the variation of the mean annual average rainfall of local storms for volume-based (quiescent treatment) design, and the median of the annualized 90th percentile rainfall intensity for flow-based (dynamic treatment) designs. These design strategies inform both the types of weather statistics best used for water quality design as well as the approaches best used in assessing water quality control performance. Similarly, given the stochastic nature of precipitation and the normal process strategy for water quality design and treatment, performance should be assessed at seasonal scales.

On these bases, the Project recommends significant changes in current application and design practices for OGS_h and sedimentation basins in Anchorage. These recommendations include changes to standard design storms, parameters, and elements. At the core, recommendations call for two basic design methodologies developed around the standard storm water quality treatment strategy of low-flow treatment and peak-flow bypass. These include application of design approaches to achieve: (a) a seasonal sum-of-loads removal over the full range of particle size fractions calculated over the variation of the mean annual rainfall rate for storage treatment devices (EPA probabilistic, or volume capture, method), or (b) a flow rate-based removal at a threshold particle size calculated at the median annual 90th percentile rainfall rate for dynamic treatment devices (90th percentile, or dynamic capture, method), each summarized as follows:

- Probabilistic (Volume Capture) Method (storage performance measure):
  - Apply adjusted mean and variation of mean rainfall rate over a seasonal period to design sum-of-loads removal for devices with significant storage.
  - Calculate for both dynamic and quiescent performance under Anchorage conditions.
  - Design device to achieve a total percent removal of spherical, non-charged mineral particles of: a) 90% of all particles 20 microns in diameter, and b) 75% of all particles 5 micron in diameter.
  - Provide waste storage in concert with a long-term maintenance schedule, adjusted for basin-specific headwater conditions.

- 90th Percentile (Dynamic Capture) Method (flow performance measure):
    - Use the adjusted median annualized 90th percentile rainfall rate to design dynamic particle removal for treatment devices having no significant storage.
    - Calculate for dynamic performance under Anchorage conditions.
    - Design device to achieve a total percent dynamic capture of spherical, non-charged mineral particles of: a) 90% of all particles 100 microns in diameter, and b) 40% for all particles 20 microns in diameter.
    - Provide waste storage in concert with an annual maintenance schedule, adjusted for basin-specific headwater conditions, but having a capacity not less than 1.2 times the estimated mean annual wash-off volume of treatable particles.

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TECHNICAL APPENDICES

A. PROJECT TECHNICAL SUMMARY

A.1 Project History (performance of project summary)

In 2011, Municipality of Anchorage (MOA) Watershed Management Services (WMS) contracted with HDR to assist in the performance evaluation of the treatment components in their MS4 system as required under Permit AK52558.

The study plan was designed in collaboration between WMS and HDR scientists and engineers during the summer of 2011. The study was intended to monitor the performance efficiency of components of the MS4 system including sedimentation basins and the more advanced hydrodynamic oil and grit separators (OGS). The history of these two component studies is separated below:

A.1.1 Sedimentation Basin Study

- During the summer of 2011, three sedimentation basins were chosen and instrumentation selected. Influent and effluent from the three basins was to be monitored making 6 total instrumentation sites.
- Flow measurement weirs were installed at those sites lacking an in place weir during the fall of 2011. Also at this time mounting brackets for instrumentation were installed at each site.
- A single telemetry site with a cellular linked Onset® data logger, solar cell, barometric pressure, rain gage and water depth pressure transducer was installed at the upstream C Street site by September 23.
- Four YSI® Sonde 600OMS (previous MOA inventory) recording temperature, conductivity, turbidity and water level were installed at C Street Up (CSTUP), C Street Down (CSTDOWN), Minnesota Up (MINNUP) and Meadow Up (MDWUP) at about the same time (late September). These installations were also fitted with mounts for Gore Sorbers® to test for organic contaminants and sorbers were fitted for the fall 2011 period. These installations were primarily to test the performance and viability of the instrumentation. For the most part the instrumentation performed well although one sonde was identified as needing a new pressure transducer and larger external batteries were purchased to extend removal and maintenance intervals.
- A single fall rainfall event was sampled for water quality on 10/25/2011 for total suspended solids (TSS).
- Two more sampling events occurred. The first on 11/01/2011 to test for total organic carbon (TOC) to determine if it would skew the turbidity readings taken by the YSI Sonde.
- With the exception of MDWUP, the instrumentation was removed during freeze up in November 2011. MDWUP was left in for the winter and continued to record data throughout the winter months of 2011-2012.
- A single midwinter rain on snow storm event was sampled for TSS and TOC at MDWUP on 12/04/2011.
- One sonde was rebuilt and an additional 3 YSI sondes and cables were ordered during winter 2011/2012

- Instrumentation at all Up sites was calibrated, installed, and running again by 3/20/2012.
- Instrumentation at all Down sites was calibrated, installed, and running by 4/4/ 2012
- The spring sampling Gore Sorbers were installed on 4/3/2012 and summer and fall sampling Sorbers were installed and removed as the seasons dictated with a total of 4 sorbers being used at each site location.
- Continuous data for turbidity, conductivity, depth of flow, and temperature were collected throughout the summer and fall of 2012 with water quality grab samples collected during storm events. Grab sampling rounds were triggered by watching weather forecasts as well as weather radar and satellite imagery for advanced warning and then deploying field crews when the rain gage and flow levels indicated a storm of sufficient intensity to sample. This method worked well, with crews on call 20 hours/day, 7 days a week. Grab samples were collected from 26 storms from fall 2011 to fall 2012.
- In support of the sedimentation basin performance analysis, field measurements were taken of basin geometry, control structures and pond depths in late September 2012. Depths were taken with a weighted measuring tape and elevations tied to known elevation benchmarks with a laser level. Field maps were produced from GIS mapping and as built drawings.
- Continuous rainfall and barometric pressure data was collected throughout the study for latter analysis and comparison to flow and turbidity rates.
- Long term weather records from NWS were collected and analysis with Synop analytical software to give an updated picture of Anchorage weather patterns and storm statistics.

A.1.2 OGS Study

The OGS study was planned in two components: 1) to look at the sediments captured in select existing hydrodynamic separators and 2) to quantify removal efficiencies of a typical hydrodynamic separator under laboratory conditions. There were original plans to instrument the inflow and outflow from an installed device but the difficulty of instrumentation and working confined space requirements made this option too expensive and it was ultimately discontinued.

- Four OGS were sampled during the fall of 2012. Two sites each from Alaska Dept of Transportation (ADOT) and MOA facilities. The ADOT sites were mostly arterial roadway basins, one MOA site was a residential basin and one was a school parking lot. Only hydrodynamic separators were targeted but one site was determined to be a conventional OGS after the fact. Sites were selected to represent a cross section of OGS basins with consideration of cleaning needs and logistical coordination with maintenance crews. Samples were collected from sediments discharged at treatment facilities from vacuum trucks, total volumes of sediments captured were estimated and samples sent to a local lab for particle size distribution analysis and organic content. Laboratory results were received in late fall 2012.
- Bench top testing of a Stormceptor STF 900 was contracted out to Good Harbour Labs in Ontario Canada using Anchorage street sediments collected during 2011. The sediments were combined and particle size distribution determined by a local laboratory DOWL-HKM. These sediments were shipped to Good Harbour for testing inputs.
- Actual testing took place during the summer of 2012 with a final report issued in late fall 2012.

A.1.3 Data Analysis

Data from these studies was analyzed during the fall and winter of 2012-2013. Extensive literature reviews were completed as part of this analysis and study data was leveraged with this research to give an overall view of the MS4 treatment train and the position and affect of the various components including street loading rates, street sweeping processes, Anchorage weather statistics, sediment washoff, catch basin sediment capture as well as OGS and sedimentation processes.

B. PERFORMANCE ANALYSES

B.1 Sediment Transport, Catch Basin and OGS Performance Analyses

B.1.1 Description of Technical Approach

During 2012, two studies were completed for regulatory compliance of the MOA MS4 permit requiring study of the efficiency of oil and grit separators (OGS). In addition to the two studies a review of historic street sediment loading records was performed and a modeling effort was undertaken of the sediment transport train up to and including the OGS.

The first study was to assess particle size distribution, organic content and quantity of accumulated sediments removed by OGS in various environments within the Municipality of Anchorage (MOA).

The second study was laboratory testing of pollutant removal efficiency on a commonly used (in the MOA) OGS using sediments removed from Anchorage streets. These sediments were collected manually for study purposes and also from street sweeping maintenance activities.

Review of historic street loading included previous studies and maintenance records from MOA street maintenance.

The modeling effort follows a sample of street sediment from initial road sanding to its deposition into a sedimentation basin or receiving water. This model looks at the changes in character and quantities of the sediments as it moves through each component of the MS4 treatment system.

B.1.2 OGS Sediment Sampling Study

The Anchorage MS4 contains a wide variety of Oil and Grit Separators (OGS) from simple vault structures to more technologically advanced Hydrodynamic units. The size of the drainage basins, size of the units, type of land use and responsible agencies also varies widely.

The OGS sediment sampling studying included four OGS that were tested using protocol outlined in Appendix D-2 QAPP (Municipality of Anchorage, 2012). Two of the OGS units testedwere on streets owned and maintained by the Alaska Department of Transportation (ADOT) and two were on MOA owned and maintained streets. Initially all four units were to be hydrodynamic OGS but one unit ultimately proved to be a standard vault separator with a skimming baffle.

This study looked at the total accumulated sediments in the separator vaults over a known time interval between cleanings. Both ADOT and MOA maintenance departments keep records of cleanings and made these available to the field samplers.

Sediments were extracted from each unit using standard vacuum trucks and maintenance crews employed by, or under contract to the respective owners. Sampling protocol was in place for these efforts but no effort was made to monitor the cleaning process. Removed sediments were delivered to normal treatment facilities and contained for measurement in ad hoc containment areas.

Because of the highly liquid nature of the sediments, the samples were allowed to settle and much of the excess water decanted from the sample. The resultant settled mass was measured for total volume and a representative sample collected and sent to DOWL-HKMfor particle size distribution (PSD) and organic content (TOC) analysis. Due to the ad hoc nature of the settlement containment and decanting process, the quantity of the accumulated sediments are a first order estimate at best.

Table B.1 lists characterization parameters of the sampled OGSs and selected results, the full lab results as well as the field notes may be found in Appendix D. A map of sampled sites is contained in Appendix E.1.

Table B.1 OGS Accumulated Sediment Sampling

OGS Basin	Basin Area (sq. ft)	Total Curb miles in basin	Basin Type	OGS Unit Model	Time since last cleaning (years)	Estimated Volume of accumulated sediments (cubic feet)	Estimated dry weight of sediments (lb)	Percent passing #200 sieve (75 micron)	Organic Content of sediments
Old Seward and 74th Ave	770,000	.82	Arterial	STC 3600	0.70	11	1656 lb	10%	3.9%
Juneau Street N. End	4,568,000	7.17	Residential	STC 13000	1	60	9034 lb	33%	20.7%
Tudor Rd West of Lake Otis	400,000	.57	6 lane arterial with divider	STC900	1.85	6.75	1016 lb	17.2	4.4%
Mears Middle School 100th Ave and Bayshore Dr.	447,600	1.07	School Parking area	UK. “T” baffled tank	0.85	4	602 lb	34.9	9%

Volume of sediments is a measured value from field notes. Basin area and curb length are from GIS analysis. Dry weight is estimated from approximately 150 lb/ cubic foot based on partially drained sediments. Organic content and fine fraction (<#200 sieve) are from laboratory results.

Captured loads showed a wide variability in part due to differences in the characteristics of the drainage basins and also because of variability between the sediment extraction and decanting methods used by different crews. This later factor was difficult to control in the production maintenance environment.

- Captured sediments do show some trends worth noting as follows:
    - Organic content of the samples is significantly higher in the residential and school parking basins where lawn clippings and leaf litter are a larger contributor to the total washoff load. This organic content may also skew the Particle Size Distribution (PSD) toward the finer fraction because as the organic particle are broken up by the testing apparatus and pass into the finer screens.
    - All PSDs are overall finer than the PSD for tested street sediments. This could be attributed to several factors, related to either selective mobility of the finer particles during smaller storm event or the capture of courser sediments in the catch basins and piping system upstream of the OGS.
    - PSDs of OGS on arterial streets (where traffic volumes and speeds are greater) are courser than from the less energetic environments of residential and parking areas.
    - OGS captured PSDs show an average fine fraction (<20 micron) of 16%. This is substantially above what would be expected given OGS performance curves developed from bench top testing. The reasons for this are likely related to the predominance of small flow events and the fining of the influent sediments by upstream processes. Reasons aside, the numbers support the use of bench top testing data as a conservative estimate for removal efficiencies by OGS devices.
    - Even with crude removal and measurement methods, there are some indications that the removed quantities may be larger than what can be accounted for by simple mobility calculations of street loading data. Two factors are implicated here. One is the unquantified contribution of the spring snow melt season. Total loads measured at the sedimentation basins indicate that a portion of the total loading could be attributable to the spring melt event. The other factor is the documented build up over the summer that keeps the street loading nearly constant in spite of multiple storm events. Successive small storms acting over the course of the summer on a regenerating mass of street sediment will mobilize a larger total mass than indicated by the analysis of mobility for a one time large storm event.

B.1.3 Bench Top OGS Testing

A second analytical effort was directed at laboratory testing of a commonly used hydrodynamic separator to quantify its performance based on the PSD of Anchorage streets sediments. These proprietary units are tested during development and certification. Testing commonly adheres to New Jersey Department of Environmental Protection (NJDEP) or other state testing protocols but the sediment pollutant loads and composition are different than those found in Anchorage streets environments. This study was an attempt to make a correlation between the NJDEP testing and the conditions found on Anchorages street. The study was contracted out to Good Harbour Labs in Toronto. The complete Good Harbour report can be found in Appendix F.

B.1.3.1 Péclet Number

As part of the study, a non dimensional Péclet number relating the flow rates, particle settling velocities, and the unit's dimensions is related to removal efficiency performance and compared at various pollutant and hydraulic loading rates. The resultant performance curves show a strong correlation between Péclet number and removal efficiencies within brand specific families of OGS designs and also across varying pollutant and hydraulic loading rates. A previous study (Wilson, Mohseni, Gulliver, Hozalski, & Stefan, 2009) showed a similar strong correlation. This correlation between removal efficiency and particle size settling velocity and units dimensions will make it possible for designers to calculate Anchorage based removal efficiencies from NJDEP testing results and will allow the MOA to use NJDEP testing of different OGS families as a performance specification for future installations without individually testing each device under Anchorage conditions.

B.1.3.2 Test Sediment Preparation

Street sweeping sediments collected from MOA streets during 2011 were shipped to Toronto for use in testing. These sediments were analyzed for PSD by Anchorage local lab DOWL-HKM and the coarser fraction of the samples was removed at the #4 sieve to represent the natural process of the system whereby the coarsest particles are left on the street or settled in the pipe system prior to reaching the OGS.

B.1.3.3 Study Fine Sediment Losses

The results of this study point to problems associated with handling of sediment samples or their metering into the influent stream. The finer fraction of this sample appears to have been lost during some phase of the PSD testing, transport, handling or during the actual testing process. Because of this loss the testing is not considered valid for the capture rates of particles finer than 25µm. As an alternative to this truncated removal efficiency performance curve, a study of a similar unit done by the University of Florida (Florida, 2008) for Rinker Materials was accessed. Armed with the scalable justification provided by the Péclet number analysis, Rinkers tests were used to predict removal efficiencies for the remaining fine particle fraction.

Results of the benchtop testing are contained in the Good Harbour report with the exception of the completed removal efficiency performance.

Table B.2 and Figure B.1 shows the complete removal efficiency for a Stormceptor® unit run at maximum designed flow capacity.

Table B.2 Stormceptor OGS Sediment Removal Efficiency

Particle Size		Removal Efficiency
Inch/Sieve size	Microns	
3"	75000	100.00%
2"	50000	100.00%
1 1/2"	37500	100.00%
1"	25400	100.00%
3/4"	19000	100.00%
1/2"	12500	100.00%
3/8"	9500	100.00%
#4	4750	100.00%
#10	2000	100.00%
#20	840	100.00%
#40	420	100.00%
#60	250	100.00%
#100	149	100.00%
#140	105	95.50%
#200	75	86.60%
	35.2	72.70%
	22.4	48.46%
	13.1	21.67%
	6.6	12.31%
	4.6	9.80%
	3.2	5.29%
	1.3	2.29%

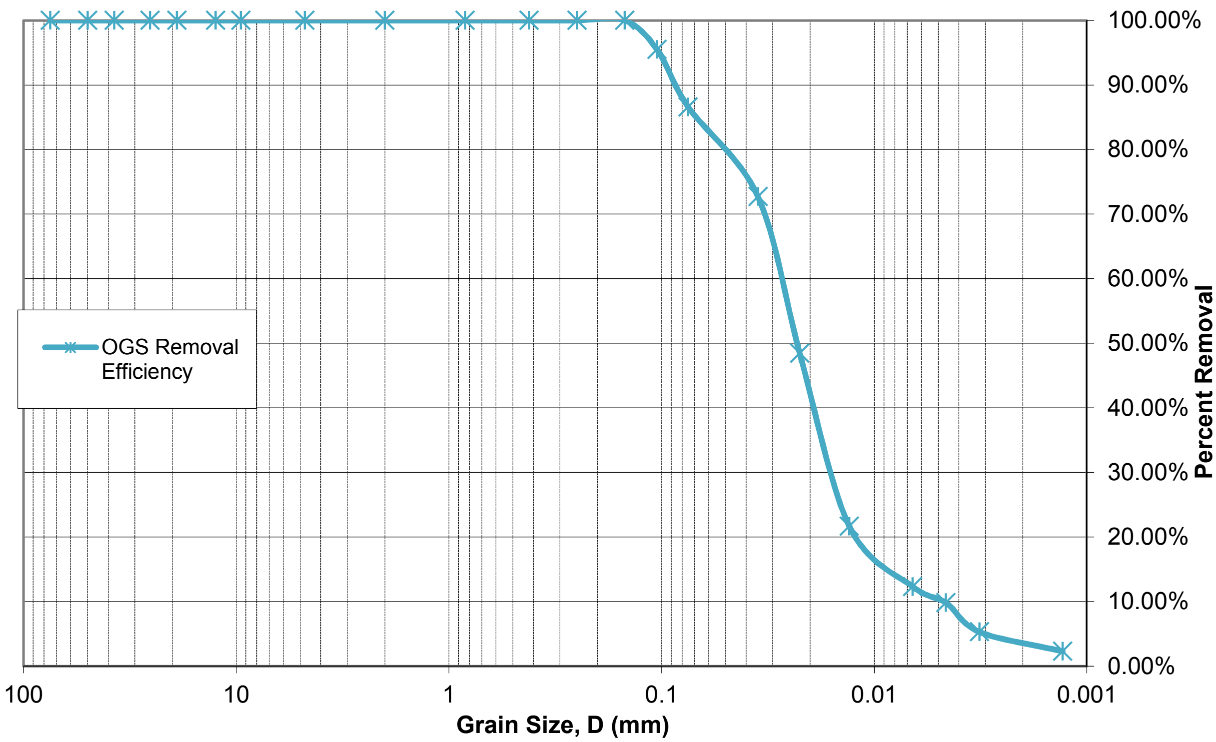


Figure B.1 Stormceptor OGS Sediment Removal Efficiency Graph

B.1.4 2)Catch Basin sediment capture

After the sediment load enters the curbside catch basins, it enters the MS4 and begins the treatment process. At this time no effort has been made to quantify the open ditches and swails that may connect parts of the piped system. The first link in the enclosed systems are the catchbasins, pipes and manholes that make up the transport system. These parts of the system can be effective at removing the coarser components of the pollutant load. The following is an analysis of this part of the system.

1. The standard catch basin was modeled as a basic settling structure and was chosen due to its position in the system and will see the lowest loading rates and therefore represent the best settling environment prior to the OGS.

A standard catch basin is 4 feet in diameter and contains an 18 inch catch area below the outflow pipe invert per standard MOA specifications (MASS). Many older catch basins do not meet these requirements but new and rebuilt parts of the system should meet these requirements.

Modeled using the standard method of equating overflow loading rate to particle settling velocity indicates that the standard catch basin is capable of capturing particles as fine as 35-40 micron. Because of the configuration of a catch basin this non-conservative analysis was further modified with a short circuiting/configuration/turbulence factor. The resultant capture rates for various standard particle sizes is given in the table below:

Table B.3: Catch Basin Capture Rates for Various Standard Particle Sizes

Particle Size			
Micron (µm)	**Sieve**	**Particle Settling Velocity, fps**	**% Settled**
9500	3/8"	177.45515	69.68%
4750	#4 Sieve	44.36379	65.17%
2000	#10 Sieve	7.86505	58.59%
850	#20 Sieve	1.42062	50.87%
425	#40 Sieve	0.35516	43.58%
250	#60 Sieve	0.12289	37.30%
150	#100 Sieve	0.04424	30.67%
106	#140 Sieve	0.02209	25.87%
75	#200 Sieve	0.01106	20.95%
34.5		0.00234	10.43%
22		0.00095	5.80%
20		0.00079	5.03%
12.9		0.00033	2.44%
9.2		0.00017	1.33%
6.5		0.00008	0.69%
4.6		0.00004	0.35%
3.2		0.00002	0.17%

Catch basin design studies (Pitt & Clark, 2002) support these numbers and also indicate that settling rates decline sharply when the accumulated sediments have reduced the water depth below 8 inches.

B.1.5 Sediment Transport Modeling

The Sediment Transport Model as described below is a combination of basic mass transport equations, small catch basin settling models, and the Good Harbour OGS efficiency results, bracketed with previous data from street sanding specifications, street loading and washoff rates studies, and 2012 OGS sediment capture sampling. Figure B.2 shows the change in character of the initial street traction sand as it moves through the system. A discussion of recommended loads and abstraction rates for design and planning purposes follows.

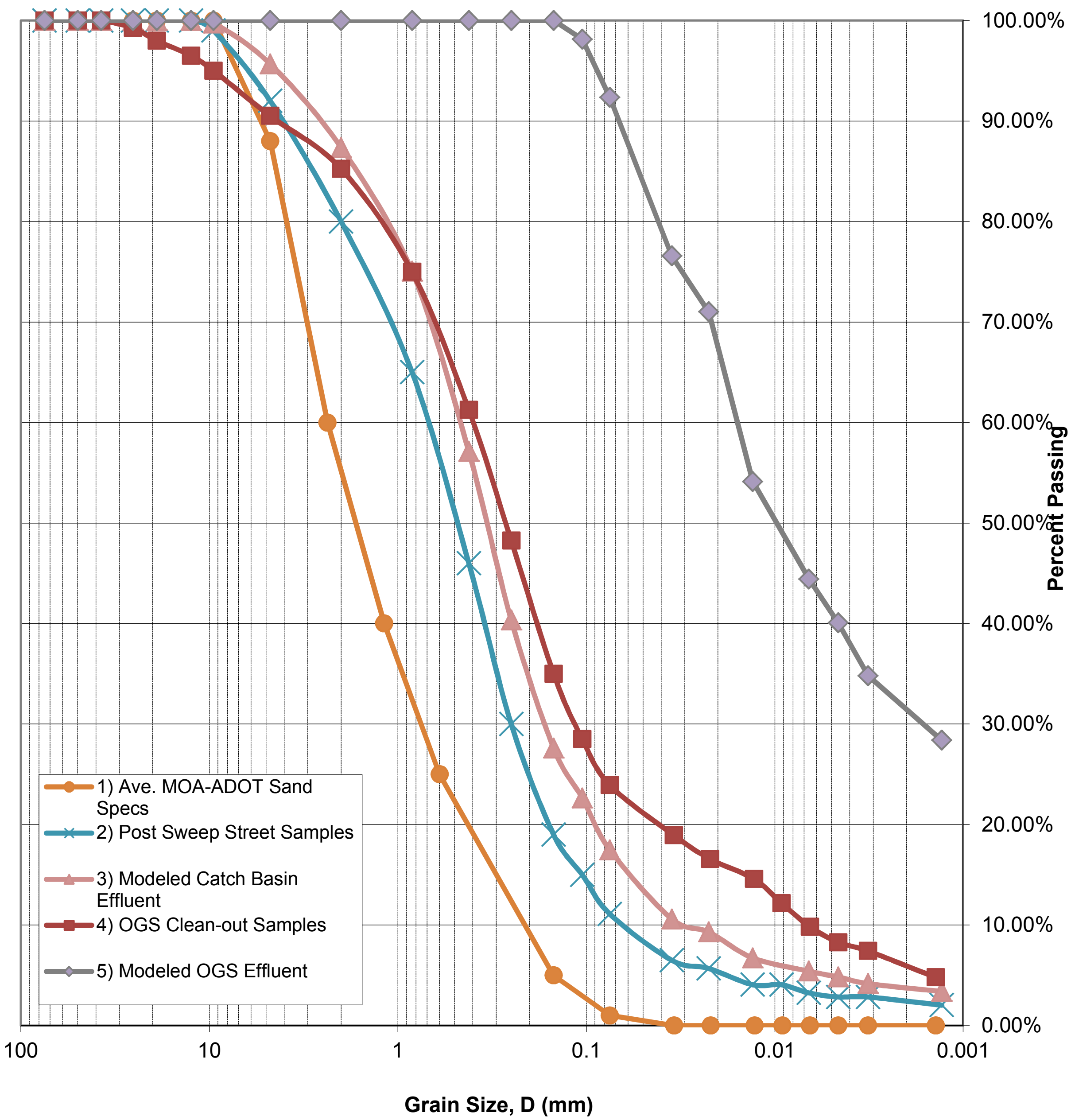


Figure B.2 MS4 Sediment PSD Characteristics

The lines on Figure B.2 represent various stages along the treatment train as follows:

1. Line 1is an average gradation of the street traction sand taken for MOA and ADOT street sand specifications. This is a representation of the sand that is spread on the streets for traction. By necessity more of the loading is associated with intersections where traction is at a premium and less is spread on straight non intersection stretches of roadway. This sand as it is laid down is relatively clean, may contain salt to make it workable and contains little or no organic content.

After the sand is spread on the roadways, some is bound in street and gutter ice, some is side cast off the roadways onto the shoulder and some is scrapped up and hauled to snow disposal sites. The fraction remaining at the start of the spring melt cycle is concentrated along the gutters and near the intersections. These deposits have been milled by traffic to some extent and may pick up other fine fraction components from organics, trackout from side alleys, and/or wind blown sediments.

There are presently no studies that quantify how much street loading is available for wash off before spring snow melt flows but data from post melt sampling indicate that the loads are in excess of 10 times the measured summer street loading (Report, 2012). The 2012 monitoring of sedimentation basin influent indicate that between 5-10 percent of the total pollutant load is attributable to the spring wash off event. These numbers appear low compared to the available street loading but snow melt runoffs are less energetic for mobilizing sediments (with the possible exception of ponded areas), sediment loads are more evenly distributed across the entire surface and much of the sediments are locked up in the ice and snow layers.

After snow melt is substantially complete one of the major components of the treatment train begins in the form of spring street sweeping. This process may be one of the most cost effective ways to remove pollutants from the system while they are on street level and not yet mixed with washoff water. Spring sweeping removes 9/10th of the total load from the street and parking lots, mills the accumulated organic matter, and leave much of the finer fraction behind. The action of both the street sweeping and continued traffic tends to concentrate the remaining sediment load along the street gutters where it is readily transported by subsequent rainfall/runoff events. Parking lots loads remain more evenly distributed.

2. Curve 2 is the PSD of numerous combined street sediments sampled taken throughout the summer after spring sweeping has been completed. These samples were collected in a previous study using vacuum and/or broom sweeping methods on a variety of roadway types throughout the summer period. This PSD shows the effects of milling and abstraction of the courser particles by sweeping and may also show an increase in organic content. Organic particles from tree, shrub and lawn litter is thought to make up a signification portion of the total pollutant load. Even though this element of the washoff is difficult to settle using conventional methods due to its low specific density, it shows up as a large component of the trapped loads in residential OGS samples. The literature indicates that organic loading may be as high as 30-40% by weight and the organic content of our 2012 OGS sampling was as high as 20%.

Using the post sweep street PSD results as a starting point, continued processing by the MS4 system was modeled. The first two components of the system are a basic washoff model and settlement in the initial catch basins.

The wash off model (Appendix C.1) compares the tractive forces generated by washoff flows from a typical one year recurrent storm event with the critical Shields number of the curb concentrated sediments. It projects that resultant washoff force is sufficient to mobilize all but the coarsest fraction of the accumulated sediments. Smaller storms will preferentially mobilize the finer fractions and also the low density organic material. This simplified model looks only at a single large storm event to see what fraction of the accumulated dirt will enters the storm drain system during this yearly event. It concludes that most of what is available on the street will be mobilized by the end of the fall storm season. What is not modeled is the continued build up of new sediments throughout the summer months, sweeping effects and their effects on mobilization into the system. The total load mobilized over the course of the summer may be several times larger than the initial street loading after the spring sweep. The 2011 MS4 Street Sweeping Report shows that total street loading is a relatively uniform 1000 lb/ curb mile in the spring for all road types, decreases on collector and residential streets but increases on arterial streets during the summer (Report, 2012).

The catch basin abstraction model looks at a standard catch basin with a top inlet, 4 foot diameter manhole section and 18 inch storage capacity catch below the outflow pipe invert. This is the standard MASS design for new catch basins, it assumes that these catch basins are not inline with the main storm conveyance piping but are constructed off line and are cleaned/maintained so that the sumps are less than 1/3rd full during a washoff event. Even when modeled as a simple inefficient settling basin using standard hydraulic loading rate analysis these basins are shown to be relatively effective at removing a large portion of the coarser fraction from the washed off sediments. Our modeling shows 20% to 60% removal of the + #200 (75µ) fraction. Because of the low density of organic particles, catch basins may have little effect on their removal.

Based on this representation and the PSD of sediment entering the system it can be hypothesized that a properly functioning catch basin is capable of removing 40% of the initial sediment load. This removal is concentrated in the coarser particle sizes. Literature (Pitt & Clark, 2002) suggests that 30-40% capture rates are possible for these devices. It is recommended that 30% capture be used for design and planning purposes.

3. Line 3 in Figure B.2 shows the hypothetical PSD from the sediment stream as it exits the catch basins. At this point from an initial load of 100 grams on the street surface, 99 grams has entered the drop inlet, 30 grams may be retained in the catch basins and 70 grams continue to the next control structure in this case a hydrodynamic OGS.
4. Line 4 represents the average PSD of the 4 OGS samples collected in 2012. This line deviates from the hypothetical influent PSD (line 3) in 2 areas but given the small data set and sampling methods it appears to lend credence to the assumptions that properly sized hydrodynamic OGS devices can capture a large percentage of the courser fraction particles and can also be effective in capturing significant fine fraction particles. Applying the capture efficiencies of the OGS bench top study to the PSD exiting the catch basins results in curve 5 on Figure B.2.
5. Line 5 shows the hypothetical PSD of sediments passing through a properly sized OGS. At this point most of the particles coarser than 100 µm have been removed and the initial load of 100 grams has been reduced to less than 10 grams. This appears to be supported by empirical evidence from the 2012 sedimentation basin study. Even though status of upstream OGSs in these systems was unknown or nonexistent, very

little build up of coarse settled particles was noticed in the quiescent areas behind the gauging weirs.

Conclusions

Looking at the initial load on the streets and using known data points, modeling, empirical evidence and best engineering judgment the initial MS4 treatment train can be seen in Table B.4.

Table B.4: MS4 Treatment Train Sediment Characterization

Point in system	Load (unitless)	D50 (microns)
Applied to Streets	Unknown	1750
On the Streets Before Spring Melt	Unknown	Unknown
On Streets After Spring Melt	9,000	Unknown
On the Street After Spring Sweep	1000	480
Washed off Streets	995	470
Catch basin Effluent	700	350
OGS Effluent	100	10

These numbers indicate that street sweeping removes the largest portion of the initial load at approximately 8000 units, followed by the OGS at 550 units, the catch basins at 350 units and finally the sedimentation basins that remove a portion of the final 100 units.

Recommended sediment loading rates to be used in design calculations

Based on this preliminary analysis the following loading rates are recommended for design purposes. When more information becomes available about street loading and sweeping effectiveness these recommendations will be updated.

These rates pertain to the areas of basins that are the main contributing areas of sediments mobilized into the MS4. These include curb and gutter streets and parking areas that drain directly to the MS4. Streets and parking areas that drain to properly designed and vegetated swale systems are not included. These vegetated ditch and swale systems are effective in removing a large percentage of the coarse sediment fraction and as such contribute little to the overall treatable load entering the catch basin and OGS devices.

Annual street wash off loading rates:

Curb and gutter streets and streets not bordered by connected parking lots:

Arterial streets	4000 lb/ curb mile
Residential streets	2500 lb/curb mile

Curb and gutter streets bordered by connected parking lots. Adjacent parking lots will be defined by minimum threshold depth, and must be directly connected to the street.

Arterial Streets	8,000 lb/curb mile
Residential Streets	5,000 lb/curb mile

Parking Lots with separate drainage systems

Parking Lots	1500 lb/ acre

B.2 Sed. B. Performance Analyses and Results

B.2.1 Description of Technical Approach

This section outlines the stormwater treatment performance analyses of three select sedimentation basins within MOA. The three basins are: Minnesota Drive, C Street, and Meadows Street. Three methodologies were utilized to determine stormwater treatment performance: the current MOA Design criteria manual, a probabilistic method outlined by the EPA, and a data based approach based on field measurements gathered during the summer of 2012. These three analysis methodologies determine the overall performance efficiency of the sedimentation basins and provide guidance for basin modifications to improve water treatment processes.

B.2.1.1 Calculate DCM Performance

Introduction and Procedure

The following procedure outlines the stormwater treatment analyses methodology outlined in the Municipality of Anchorage Design Criteria Manual 2007 (MOA Project Management and Engineering Department 2007).

1. The 2007 MOA Design Criteria Manual (DCM) – Revision 2 manual methodology outlines how to design a sedimentation basin in section 2.11. These design recommendations were modified for the purposes of this project to determine the water quality treatment performance of the three sedimentation basins by performing the following procedures:
    a. Obtain the 2-year contributing basin runoff hydrographs as discussed in MOA DCM Section 2.5.
    b. Determine the approximate storm sedimentation basin surface area during the 2 year 24 hour event utilizing chapter 6 of the TR-55 method regarding detention basin hydraulics (United States Department of Agriculture 1986)
    c. Determine the settling velocities (fps) for each particle size (micron sizes 1.3 to 9500) as see in Table B.5.

Table B.5: Sedimentation Basin Influent PSD

Particle Size, mm	Percent Passing	Settling Velocities, fps
9.5	100.00%	177.455
4.75	100.00%	44.364
2	100.00%	7.865
0.84	100.00%	1.421
0.42	100.00%	0.355
0.25	100.00%	0.123
0.15	100.00%	0.044
0.11	98.15%	0.022
0.08	92.41%	0.011
0.04	76.80%	0.002
0.02	71.27%	0.001
0.01	54.08%	0.0003
0.01	44.29%	0.00008
0.01	39.30%	0.00004
0.003	39.03%	0.00002
0.001	28.18%	0.000003
pan	0.00%	0.0000005

d. Take the inverse of the settling velocity for each particle size to determine the required unit hydraulic loading rate (surface area for every 1 cfs, sec/ ft or ft^2/cfs).

e. Multiply the inverse settling velocities by the peak flow of the 2-year hydrograph. This will provide the required settling area to remove 100% of each specific particle size.

f. Divide the sedimentation basin area during the 2 year 24 hour event (step b) by the required settling area for each particle size (step e). This calculation provides the approximate dynamic performance percent removal for particle sizes (micron sizes 1.3 to 9500).

g. Multiply the average particle size removal rate by the mass (in grams) of each particle size (determined by separating 100 grams of sediment based on the PSD) as seen in Table B.5. This will give you the grams removed through settling.

h. Find the total mass captured by the basin to determine the overall percent removal performance. This is the primary removal performance metric for the basin.

i. Additional basin characteristic is the hydraulic loading rate of the basin. This is determined by dividing the peak flow by the treatment surface area. This is a commonly used metric that is easily comparable to other methods.

j. Average particle detention time is another basin characteristic. This is a commonly used metric that is easily comparable to other methods. Calculate this by dividing the total basin volume by the average 2 year 24 hour inflow hydrograph.

$$t_n = \frac{V_B}{Q_{ME}}$$
$$t_n = nominal\ detention\ (residence)\ time, seconds$$
$$V_B = Total\ Basin\ volume, cf$$
$$Q_{ME} = mean\ effluent\ discharge\ rate, cfs$$

k. This DCM estimated basin performance and hydraulic characteristics will allow a direct comparison with the two other performance analysis methods (Probabilistic and data based), and allows for sedimentation basin criteria recommendations in Appendix B.2

Results

Table B.6 below summarizes the 2 year 24 hour event statistics, contributing area inflow and sedimentation basin removal efficiencies.

Table B.6: DCM Removal Performance Summary

	Units	C street	Minnesota	Meadows
2 year 24 hour storm Average Storm Intensity	in/hr	0.053	0.053	0.053
2 year 24 hour Storm depth	in	1.26	1.26	1.26
Peak Runoff Inflow	ft^3/s	21.39	18.01	10.84
Sedimentation Basin Treatment Area	ft^2	158,078	49,658	26,452
Hydraulic Loading Rate, HLR	ft/s	1.32E-04	3.63E-04	3.54E-04
Average Removal Efficiency	%	88.78%	81.13%	81.19%
Average Detention Time	sec	25,142	6,481	5,879

The average storm intensity for 2 year 24 hour event of the basins was determined using the Intensity-Duration-Frequency (IDF) curves presented in the 2007 MOA Drainage Design Guidelines, as seen in Figure B.3 (MOA Project Management and Engineering Department 2007). The same intensity was utilized for each of these subbasins because orographic affects are minimal.

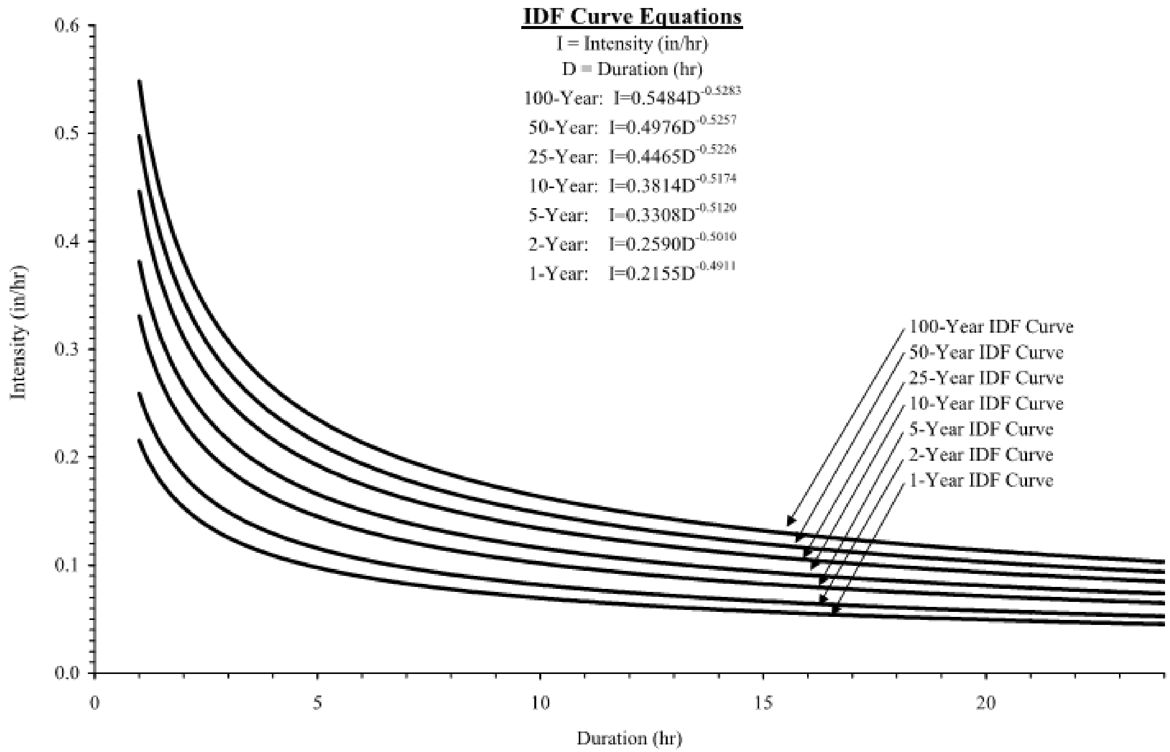


Figure B.3: IDF Relationships for Anchorage Alaska TSAIA

The storm depth was determined by multiplying the average intensity by 24 hours, or referencing table 6-2: MOA Base Storm Volumes within the Drainage Guidelines (MOA Project Management and Engineering Department 2007).

The basin treatment area was determined using the methodology in Chapter 6 of the TR-55 manual, Storage Volume for Detention Basins, which balances the inflow discharge and runoff volume with the available storage and outflow discharge (United States Department of Agriculture 1986). This provides an approximate storm treatment water surface elevation and surface area based on the treatment basin hydraulics.

The average removal rate percentage estimates the amount of sediment retained by the sedimentation basin from particle settling. This particular percentage represents a weighted average based on the incoming particle size distribution.

The hydraulic loading rate is basin inflow divided by basin water surface area. This parameter is commonly used in water treatment equations, and is an easily comparable parameter when evaluating different methods. There is an inverse relationship between hydraulic loading rate and particle settlement: as hydraulic loading rate decreases, particle settlement increases.

Conclusion

Based on the DCM method the removal efficiency of C Street (approximately 89%) which is marginally greater than both Minnesota and Meadows (approximately 81%), which performed similarly. The detention time and hydraulic loading rate follow similar patterns to the removal efficiency. These values are comparable to the data based analysis (Appendix B.2.3) and the project method analysis (Appendix B.2.3) which will be discussed in Appendix B.2.4 Performance Summary.

B.2.2 Calculate Data-Based Analyses Performance for Sedimentation Basins

Introduction and Procedure

The following procedure outlines the stormwater treatment analyses for the continuous data gathered during the 2012 summer for the project basins (C Street, Minnesota, and Meadows). The data analysis for the collected data includes a storm by storm analysis estimating the hydrodynamic parameters associated with basin efficiency (steps 1 through 4), and a sum-of-loads approach that outlines basin performance over the summer (steps 5).

1. Isolate the hydrographs for separate storms by defining the beginning and end of the hydrograph. Define the beginning by identifying the date and time of the initial precipitation measurement for each storm event as defined by the 2012 SYNOP analysis in Appendix C.3. Define the end time and date of the hydrograph when the only influent flow is attributed to base flow.
2. Calculate the nominal detention time, t_n, by dividing the total basin volume by the average storm discharge.

$$t_{ni} = \frac{V_B}{Q_{ME}}$$

$$t_{nb} = \frac{\sum_{i=1}^{n} t_{ni}}{n}$$

$$V_B = Total\ basin\ volume, cf$$
$$Q_{ME} = mean\ basin\ discharge\ rate\ for\ storm\ event, cfs$$
$$i = discrete\ storm\ counter$$
$$n = total\ number\ of\ storms\ during\ the\ summer\ of\ 2012$$
$$t_{ni} = nominal\ detention\ (residence)\ time\ for\ a\ storm$$
$$t_{bi} = average\ nominal\ detention\ (residence)\ time\ for\ a\ basin$$

3. Calculate the number of Continuously Stirred Tank Reactors (CSTR) by averaging the number of CSTRs from all storms. This parameter is equal to the turbulence factor defined in the probabilistic performance methodology (Appendix B.2.3)

$$N_i = \frac{t_n}{(t_n - t_p)}$$
$$N_b = \frac{\sum_{i=1}^{n} N_i}{n}$$
$$N_i = number\ of\ CSTRs\ for\ a\ discrete\ storm$$
$$N_b = Average\ number\ of\ CSTRs\ for\ the\ basin$$
$$t_p = time\ of\ the\ peak\ effluent\ pollutant\ load, secs$$
$$t_m = mean\ time\ within\ the\ sedimentation\ basin$$

4. Calculate the hydraulic efficiency of the sedimentation basin by averaging the efficiency from all the storms.

$$\lambda_i = \frac{t_p}{t_n}$$
$$\lambda_b = \frac{\sum_{i=1}^{n} \lambda_i}{n}$$
$$\lambda_i = hydraulic\ efficiency\ for\ a\ discrete\ storm$$
$$\lambda_b = average\ hydraulic\ efficiency\ for\ a\ basin$$

5. Determine the effective volume ratio by utilizing the number of CSTRs and the basin hydraulic efficiency. The equation to determine effective volume is below.

$$e_i = \frac{\lambda_i}{\left(1 - \frac{1}{N_i}\right)}$$
$$e_b = \frac{\sum_{i=1}^{n} e_i}{n}$$
$$e_i = effective\ volume\ ratio\ for\ a\ \ discrete\ storm$$
$$e_b = average\ effective\ volume\ ratio\ for\ a\ \ basin$$

6. Finally determine the total summer mass influent, mass effluent for each sedimentation basin. This allows you to determine the percent of sediment captured over the summer. The procedure for this process is detailed in Appendix C.4.1 and Appendix C.4.2.

The performance estimated sediment removal from the data will allow direct comparison with the two other performance analysis methods (Probabilistic and DCM). This analysis also helps define the sedimentation basin criteria recommendations in AppendixB.3.

Results

Table B.7 through Table B.10 outlines the basin hydraulic characteristics and performance for the three project sites and summarizes the averages for easy comparison.

Table B.7: C Street Sedimentation Basin Hydraulic Characteristics

Storm Event	t_{peak}	$t_{nominal}$	t_{mean}	λ	N	e
-	secs	secs	secs	-	-	-
1	91,808	267,473	138,607	0.34	2.96	0.52
3	-	-	-	-	-	-
5	99,009	156,573	115,202	0.63	7.11	0.74
7	-	-	-	-	-	-
8	90,900	310,374	99,000	0.29	12.22	0.32
27	78,300	299,928	107,100	0.26	3.72	0.36
28	18,900	89,642	30,600	0.21	2.62	0.34

Table B.8: Minnesota Sedimentation Basin Hydraulic Characteristics

Storm Event	t_{peak}	$t_{nominal}$	t_{mean}	λ	N	e
-	secs	secs	secs	-	-	-
1	36,899	69,204	58,506	0.53	2.71	0.85
3	24,301	92,564	50,405	0.26	1.93	0.54
5	29,696	42,988	56,691	0.69	2.10	1.00
7	50,398	107,067	79,201	0.47	2.75	0.74
8	14,400	98,150	46,800	0.15	1.44	0.48
27	41,400	65,420	78,300	0.63	2.12	1.20
28	50,400	79,977	71,100	0.63	3.43	0.89

Table B.9: Meadows Sedimentation Basin Hydraulic Characteristics

Storm Event	t_{peak}	$t_{nominal}$	t_{mean}	λ	N	e
-	secs	secs	secs	-	-	-
1	4,496	75,719	27,896	0.06	1.19	0.37
3	26,104	105,281	35,103	0.25	3.90	0.33
5	-	-	-	-	-	-
7	10,799	94,768	31,499	0.11	1.52	0.33
8	900	85,882	28,802	0.01	1.03	0.34
27	-	-	-	-	-	-
28	-	-	-	-	-	-

Table B.10: 2012 Hydraulic Efficiencies Summary

	Units	C street	Minnesota	Meadows
Average Number of CSTRs for a Basin, N_b	count	5.73	2.36	1.91
Average Hydraulic Efficiency ratio, λ_b	-	0.35	0.48	0.11
Average Effective Volume ratio, e	-	0.45	0.81	0.34
Average Nominal Detention Time, t_n	-	224,797	79,338	90,412
Average Mean Detention Time, t_m		98,101	63,000	30,825

Storms events are identified in Appendix C.3.1. The storms identified above we selected to account for a range of storm volumes, intensities and modes. The averages from these storms coarsely represent the hydraulics of each sedimentation basin. Nominal detention time determines the average time for a runoff event to pass through the basin in ideal conditions with complete basin volume utilization. The mean detention time indicates the approximate average time for a runoff event based on the hydrograph and effluent pollutant concentration data. A longer detention time indicates decreased velocities and increased chance for sediment settlement. The detention time for C Street is the largest, respectively, for both these categories.

The other three hydraulic characteristics quantify various methods for analyzing basin performance. A study by Persson, completed in 1999, indicates that a λ>0.75 indicates good hydraulic efficiency; 0.5<λ≤0.75 indicates satisfactory hydraulic efficiency; and λ≤0.5 indicates poor hydraulic efficiency (N. S. J. Persson 1999). In that study, the other hydraulic characteristic was measured as well and this project correlated the hydraulic efficiency performance with the other two parameters to come up with the following ranges of good, satisfactory, and poor performances.

$$For\ good\ hydrualic\ efficiency:$$
$$\lambda > 0.75$$
$$N > 5.91$$
$$e > 0.90$$
$$For\ satisfactory\ hydrualic\ efficiency:$$
$$0.5 < \lambda \leq 0.75$$
$$2.0 < N \leq 5.0$$
$$0.60 < e \leq 0.90$$

$$For\ poor\ hydrualic\ efficiency:$$
$$\lambda \leq 0.5$$
$$N \leq 2.0$$
$$e \leq 0.60$$

The hydraulic characteristics equations are founded experimentation that utilizes a tracer as a point (slug) contamination, and assumes the almost all the pollutant slug leaves the treatment basin. Since no tracers tests were performed on the sedimentation basins the project identified total suspended solids (TSS) as a surrogate. In this scenario, the pollutant influent is time distributed and only part of the pollutant load leaves the basin, which is the intent of the sedimentation basin. The hydraulic characteristics of the project sedimentation basin should be relatively well represented even though TSS effluent data is utilized as a surrogate to determine peak effluent pollutant time (t_p).

The overall removal efficiency of the basins is detailed in Appendix C.4. A summary of these results are seen below.

Table B.11 Sum of Loads Removal Efficiencies

	C Street	Minnesota	Meadows
Total Mass into Basin, cubic yards	15.78	18.30	2.87
Total Mass out of Basin, cubic yards	3.78	11.24	2.30
Performance Removal Efficiency, %	69.71%	38.58%	19.86%

The total mass in and out of the treatment basin was calculated using a multiple linear regression (MLR) equation that correlates flow and turbidity with TSS (Appendix **C.4**). This allowed the calculation of the total suspended mass of sediment entering the sedimentation basin and the total amount of sediment that was retained within each sedimentation basin. The retained amount was then divided by the total influent mass, which provided the removal efficiency. These values are comparable to the removal efficiencies calculated for the DCM and probabilistic methods (Appendix B.2.2 and Appendix B.2.4 respectively).

Conclusion

Based on the data based method, the removal efficiency is 69.71% for C Street, 38.58% for Minnesota and 19.86% for Meadows. The CSTR count, hydraulic efficiency and effective volume ratio approximate this removal performance as well as indicate short circuiting occurring. These values are comparable to the DCM analysis (Appendix B.2.1.1) and the project method analysis (Appendix B.2.3) which will be discussed in Appendix B.2.4 Performance Summary.

B.2.3 Calculate Project Method Performance for Sedimentation Basin

Introduction and Procedure

The following procedure outlines the stormwater treatment analyses using a probabilistic method defined by the EPA (EPA 1986). This methodology is an evaluation tool that accounts for both dynamic settling and quiescent settling in a sedimentation basin.

1. Set the adjustment factor, N, for the sedimentation basin equal to value calculated in the data based analysis Appendix B.2.2. This factor accounts for turbulence factor of the basin.

2. Apply the mean storm rainfall volume (determined in Appendix C.3: 2012 Project Climate and WQ hydrology) and mean storm intensity to the drainage basin area to determine runoff flow (cfs) and volume (cf).
3. Determine the mean storm sedimentation basin stage based on the storm runoff flow and volume using the TR-55 methodology outlined in Chapter 6: Storage Volume for Detention Basins (United States Department of Agriculture 1986).
4. Use the area determined in the previous step to define the hydraulic loading rate (HLR). HLR is equal to the runoff flow divided by the sedimentation basin surface area. The surface area is determined using known contour data and the sedimentation basin stage determined in step 3.
5. Define the particle size classes which represent a range of particles diameters. Choose the settling velocities for each particle size class using the smallest diameter for each class (micron sizes 1.3 to 9500). Table B.5 below articulates the particle size settling velocities for each class.

6. Input the values outlined above and calculate the mean removal efficiencies for each particle size based on the dynamic removal equation outlined below:

$$R_{md} = 1 - \left[1 + \frac{v_s}{n(HLR)}\right]^{-n}$$
$$v_s = settling\ velocity\ for\ a\ specific\ particle\ diameter, fps$$
$$n = turbulence\ factor$$
$$HLR = hydrualic\ loading\ rate, fps$$
$$R_{md} = mean\ dynamic\ removal\ efficiency, \%$$

7. Calculate the maximum removal rate for each particle size using the following equation and inputs

$$Z = 1 - e^{-\left(\frac{v_s}{HLR}\right)}$$
$$v_s = settling\ velocity\ for\ a\ specific\ particle\ diameter, fps$$
$$HLR = hydrualic\ loading\ rate, fps$$
$$Z = maximum\ removal\ efficiency, \%$$

8. Calculate the long term dynamic removal percentage for each particle size using the following equations and input.

$$R_{ld} = Z\left[\frac{r}{r - \ln\left(\frac{R_{md}}{Z}\right)}\right]^{(r+1)}$$
$$Z = maximum\ removal\ efficiency, \%$$
$$COV_{QR} = Coefficient\ of\ Variation\ of\ Runoff\ flow\ discharge$$
$$r = recipricoal\ square\ of\ COV_{QR}, \qquad r = \left(\frac{1}{COV_{QR}{}^2}\right)$$
$$R_{md} = mean\ dynamic\ removal\ efficiency, \%$$

9. The long term dynamic removal for each particle size denotes the percent of particles that settle through dynamic conditions over the life of the structure if maintained properly. Determine the mass removed based on the particle size distribution and a 100 gram proxy

sediment load located in Table B.5(include sediment ≥0.0013 mm). This total mass removal provides a weighted average long term dynamic removal percent for the 100g sediment load. Determine the fraction of particles not removed by dynamic settling using that weighted average.

$$f_D = 1 - \frac{R\_(ld\ average)}{100}$$
$$f_D = fraction\ of\ particles\ \ not\ removed\ through\ dynamic\ settling$$

10. Another large treatment component for a basin is the quiescent settling and treatment performance. To begin, determine the volume of the sedimentation basin during base flow conditions.
11. Determine the ratio of the basin volume divided by the mean storm runoff volume. This ratio indicates if the sedimentation storage is larger, equal to, or smaller than the mean storm runoff volume.

$$\frac{V_B}{V_R} = \frac{Sedimentation\ basin\ storage\ volume}{Mean\ storm\ runoff\ volume}$$

12. Determine the mean interval between storm mid points and the coefficient of variation of runoff volume as discussed in Appendix C.3.1 2012 SYNOP Storm Event Analysis and Identification.

$$\Delta_m = mean\ interval\ between\ 2012\ storm\ mid\ points$$
$$COV_{VR} = coefficient\ of\ variation\ of\ 2012\ runoff\ volume$$

13. Calculate the emptying rate for each particle size for the sedimentation basin during inter-event periods.

$$\Omega = v_s A_{QR}$$
$$\Omega = emptying\ rate$$

14. Calculate the emptying ratio for each particle size in the sedimentation basin during inter-event periods.

$$E = \frac{\Delta_m \Omega}{V_R}$$
$$E = emptying\ rate\ ratio$$

15. Use the nomograph in Figure B.4 to determine the effective storage volume ratio for each particle size based on the inputs calculated in step 11 and step 14.

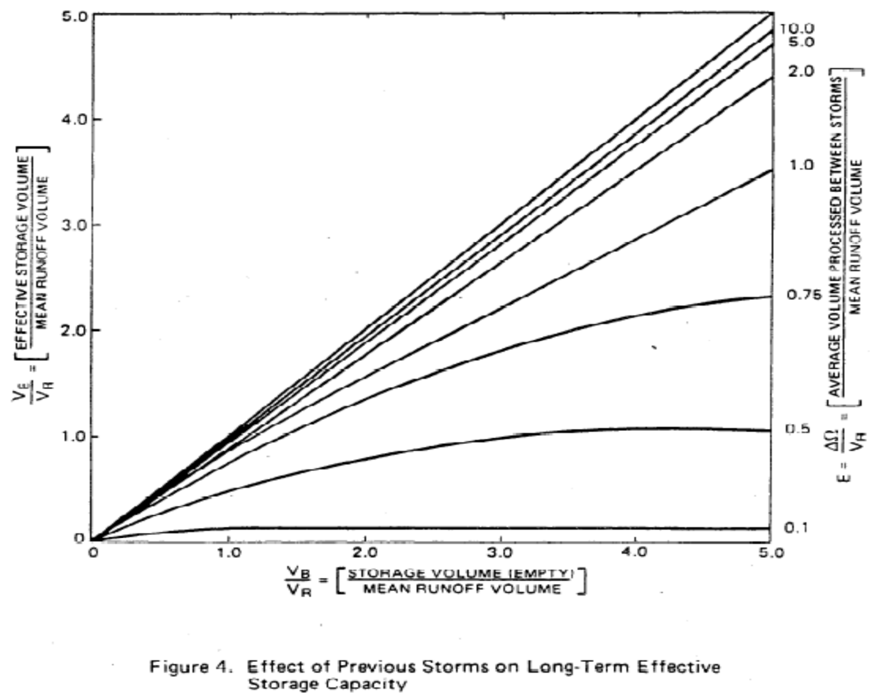


Figure B.4: Effect of Previous Storms on Long-Term Effective Storage Capacity

16. Use the nomograph in Figure B.5, the effective storage volume ratio and coefficient of variance for runoff volume to determine the percent removal for each particle size.

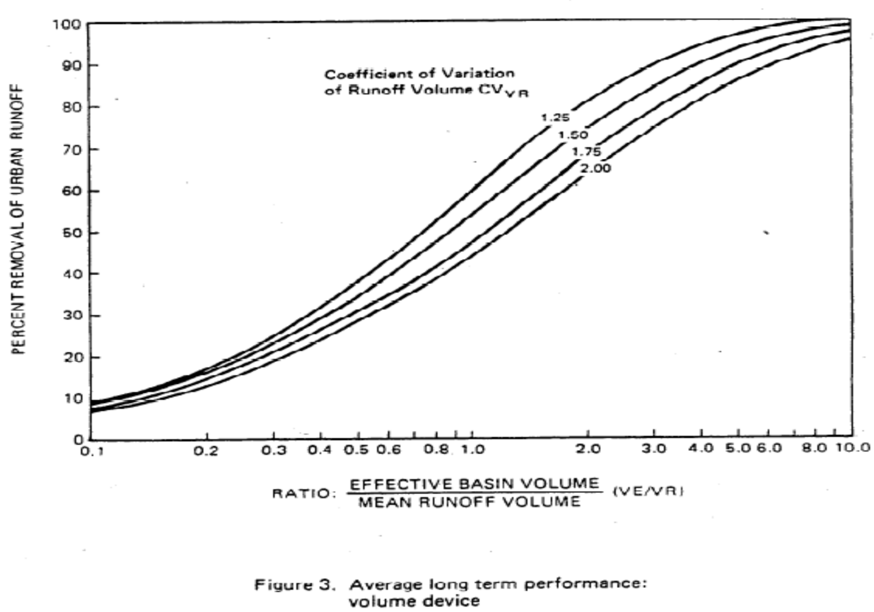


Figure B.5: Average long term performance volume device

17. The quiescent removal for each particle size denotes the percent of that particle size that settles between storm events. Calculate the mass removed through quiescent process by using the sediment mass not treated through dynamic processes. This provides weighted average quiescent removal efficiency. Determine the fraction of particles not removed by quiescent settling.

$$f_Q = 1 - \frac{R\_(Q\ average)}{100}$$
$$f_Q = fraction\ of\ particles\ \ not\ removed\ through\ quiescent\ settling$$

18. Finally the overall sedimentation basin performance can be determined with the equation below.

$$Overall\ \%\ Removal = \left[1 - \left(f_Q \times f_D\right)\right] \times 100\%$$
$$f_Q = fraction\ of\ particles\ \ not\ removed\ through\ quiescent\ settling$$
$$f_D = fraction\ of\ particles\ \ not\ removed\ through\ dynamic\ settling$$

Results

The rainfall statistics developed in Appendix C.3.1 *2012 SYNOP Storm Event Analysis and Identification* were applied to the drainage basin characteristics developed in Appendix C.2 to determine the average runoff flow and volume. The Table B.12 below summarizes the event precipitation and runoff values.

Table B.12: Input Calculated Values for Removal Efficiency Calculations

	Units	C Street	Minnesota	Meadows
Precipitation Volume, V_m	in	0.34	0.34	0.34
Precipitation Avg. Intensity, I_m	in/hr	0.03	0.03	0.03
Coefficient of Variation of Runoff flow rate, COV_{QR}	-	0.659	0.659	0.659
Reciprocal Square of COV_{QR}, r	-	2.304	2.304	2.304
Coefficient of Variation of Runoff Volume, COV_{VR}	-	1.422	1.422	1.422
Mean Interval between storm mid points, Δ_m	hrs	110.21	110.21	110.21
Runoff Volume, V_R	ft^3	296,082	271,174	151,298
Runoff Discharge, Q_R	ft^3/s	11.36	9.57	5.76

The precipitation values above represent the mean rainfall volume (inches of precipitation) and the mean rainfall intensity for the summer of 2012. The remaining rainfall statistics (coefficient of variation and the reciprocal square of COV_{QR}) were calculated using the 2012 rainfall data set.

To convert the rainfall intensity to discharge, the rational equation provided the average discharge. To convert the average storm volume (inches) to the runoff volume (cubic feet) the rational equation was used after depression storage was accounted for.

The values above, along with the sedimentation basin geometries (Appendix C.2.2) were combined to develop the mean dynamic and quiescent hydraulic characteristics. A summary of the calculated sedimentation basin values for the dynamic removal efficiencies are located in Table B.13.

Table B.13: Dynamic Removal Efficiencies

Dynamic Removal Calculation	Units	C street	Minnesota	Meadows
Adjustment Factor, N	-	5.73	2.36	1.91
Mean Runoff Event Stage, E_{QR}	ft	82.178	36.346	112.650
Mean Storm Runoff Volume, V_R	ft^3	296,082	271,174	151,298
Dynamic Sed Basin Storage, V_S	ft^3	28,183	17,968	9,877
Dynamic Volume Ratio, V_S/V_R	-	0.095	0.066	0.065
Runoff Flow In, Q_R	ft^3/s	11.36	9.57	5.76
Peak Flow Out, Q_o	ft^3/s	8.85	8.61	5.22
Sed Basin Mean Treatment Area, A_{QR}	ft^2	157,726	49,059	25,959
Hydraulic Loading Rate, HLR	ft/s	7.204E-05	1.950E-04	2.219E-04
Quiescent Volume Ratio, VB/VR	-	1.816	0.430	0.421

The mean runoff event stage is the mean water surface elevation during the mean runoff event. This value determines the dynamic sed. basin storage, the basin discharge, and ultimately the hydraulic loading rate. The dynamic basin storage is the volume of water above the weir invert. The hydraulic loading rate is the primary variable used in determining dynamic removal rate using this probabilistic method.

Finally the quiescent volume ratio characterizes the volume from the bottom of the pond to the water surface elevation with respect to the mean runoff event. By having a large quiescent volume ratio, the quiescent percent removal increases significantly.

The mean hydraulic characteristics above reflect the removal efficiencies of each treatment basin. The table below summarizes the predicted lifetime performance of each sedimentation basin.

Table B.14: Removal Efficiency Summary

	C street	Minnesota	Meadows
Sed Basin Long term Dynamic removed fractions, R_{Ld}	75.18%	61.03%	58.27%
Avg. Percent Quiescent Removal, R_{Lq}	45.50%	20.00%	17.29%
Overall Avg. Removal (Dynamic and Quiescent)	86.47%	68.83%	65.8%

The calculated overall percent removal indicates the percentage of sediment each treatment basin removes during the designed structure life.

When compared to the data based removal rates seen in Table B.11 it is evident that this methodology overestimates the percent removal similar, but not as drastic, to the current DCM methodology. One reason for this over estimation is that the calculations assume that all the basin surface area and the entire quiescent volume are utilized for treatment, which is an ideal conditions assumption. To account for short circuiting and effective basin volumes, the data based hydraulic parameters, λ (hydraulic efficiency) and e (effective volume), were used, respectively, to modify the surface area and VB/VR ratio to more accurately reflect the actual hydraulic characteristics of each basin. The surface area was by multiplied by λ (a range from 0 to 1) and the VB/VR ratio was multiplied by e (a range from 0 to 1). The new modified surface area and VB/VR ratios reflect the surface area and basin volume most likely used to treat the influent pollutants.

By modifying the probabilistic method outlined by the EPA, the removal efficiency was still slightly larger than the data based removal rates. To bring the calculated removal rates closer to the data based removal rates, the N, λ and e variables were reduced by 25%. Table B.15compares the original hydraulic characteristics, basin geometries and removal rates to the modified values to illustrate how this method paired with hydrodynamic calculations can represent basin removal performance.

Table B.15: Sedimentation Basin Performance Parameters

Original Hydraulic, Geometries and Performance Values				
	Units	**C street**	**Minnesota**	**Meadows**
Sed Basin Mean Treatment Area, A_{QR}	ft^2	157,726	49,059	25,959
Quiescent Volume Ratio, VB/VR	-	1.816	0.43	0.421
Average Number of CSTRs for a Basin, N_b	count	5.73	2.36	1.91
Average Hydraulic Efficiency ratio, λ_b *	-	-	-	-
Average Effective Volume ratio, e_b *	-	-	-	-
Overall Avg. Removal (Dynamic and Quiescent)	%	86.47%	68.83%	65.80%
Modified Hydraulic, Geometries and Performance Values				
	Units	**C street**	**Minnesota**	**Meadows**
Sed Basin Mean Treatment Area, A_{QR}	ft^2	41,403	17,661	2,141
Quiescent Volume Ratio, VB/VR	-	0.613	0.261	0.107
Average Number of CSTRs for a Basin, N_b	count	4.29	1.77	1.4325
Average Hydraulic Efficiency ratio, λ_b	-	0.2625	0.36	0.0825
Average Effective Volume ratio, e_b	-	0.3375	0.6075	0.255
Overall Avg. Removal (Dynamic and Quiescent)	%	71.29%	53.48%	28.92%
Data based Average Removal Performance				
	Units	**C street**	**Minnesota**	**Meadows**
Overall Avg. Removal (Dynamic and Quiescent)	%	69.71%	38.58%	19.86%

*λ and e were not used in the original removal performances and are left blank in the table

By applying the hydrodynamic characteristics to the probabilistic methodology, the estimated overall removal percentage of each basin begins to represent the data based analysis more accurately. The modified performance values stated in the table above represent a modified probabilistic performance methodology for detention basins as outlined by the EPA (Enironmental Protection Agency 1986).

Conclusion

The data and results indicate that the performance from highest sediment removal rate to lowest is: C Street, Minnesota, and Meadows. The C street treatment basin has much larger geometries, surface area and volume ratio, in comparison to the runoff metrics. The lower HLR for C Street indicates that the incoming flow is distributed over a much larger area, respectively, than the other two basins. This lowers water velocities within the treatment basin and settles out more sediment.

The quiescent treatment for each basin accounts for how much of the total storm volume is retained within the basin, and also how many average storms have to occur before that particular volume discharges from the basin. C Street’s quiescent volume is larger than the Minnesota and Meadows basins. C Street’s basin volume allows for a mean runoff volume to be treated for a longer period of time and directly increases the amount of sediment removed.

B.2.4 Performance Summary for Sedimentation Basins

Introduction

The two of the three performance methodologies outlined in previous sections (B.2.2 and B.2.4) need to be evaluated against the data based analysis to determine which method predicts removal efficiencies more accurately.

Results

The tables below summarize some overarching performance metrics that were discussed in the relative sections (B.2.2, B.2.3, and B.2.4).

Table B.16: Summary of Method Performance Efficiencies

	DCM Performance	Prob. Performance	Data Performance
C Street	88.78%	71.29%	69.71%
Minnesota	81.13%	53.48%	38.58%
Meadows	81.19%	28.92%	19.86%

The MOA DCM Performance method only accounts for dynamic treatment and does not account for hydraulic performance or quiescent treatment. As calculated, the DCM methodology over estimates overall removal efficiencies. The lack of a hydraulic efficiency, a turbulence factor and an effective volume factor ignores hydraulic characteristics which affect the removal performance for sedimentation basins.

A slightly modified probabilistic performance methodology allows for performance estimates similar to the data based measurements. The removal performance estimate accuracy is increased because the methodology uses hydraulic performance adjustments and quiescent removal performance. Accounting for these additional factors allows for more complete analysis, unlike the current DCM performance method.

The data based analysis is considered the control performance analysis, and, like the other methods, have pros and cons. Instrumentation maintenance, scouring, and hydraulic characterization include the areas where data precision was reduced.

Data variation was an issue for the data based performance method due to possible maintenance errors and non-functional measurement instrumentation. The instrumentation cannot account for basin scour and any occurrences of scour would greatly reduce sediment removal performance. Finally, hydraulic characteristic equations are based on non-ideal empirically derived conditions (as previously discussed). In reality, our pollutant load is time distributed and only part of the sediment exits the basin. Conceptually the calculated hydraulic efficiencies reflect the empirically derived conditions, but because of the variation will not match up precisely.

Conclusions

The three performance analyses results provide evidence for the site modification of Minnesota and Meadows. No modifications are recommended for C Street because removal performances and hydraulic characteristics were satisfactory.

A fore bay pond with a distributive weir is recommended for the Meadows basin because of hypothesized short circuiting. The fore bay pond should attenuate the incoming runoff and distribute flow evenly throughout the existing pond. This would potentially increase hydraulic efficiencies and lower the hydraulic loading rate by effectively utilizing more surface area.

Performance efficiencies can be increased at Minnesota by regarding and reinforcing the constructed wetlands to eliminate predicted short circuiting and installing a distributive weir at the outlet of the current fore bay pool. The distributive weir at the fore bay outlet will allow for additional runoff attenuation as well as provide even flow distribution into the constructed wetlands. This will reduce the likelihood of a low flow channel developing in the regarded and reinforced wetlands. The modifications will potentially reduce the hydraulic loading rate by effectively utilizing more surface area and lowering discharge rates.

Between the two theoretical performance methods, the probabilistic method characterizes more basin geometry features and accounts for hydraulic characterization by utilizing an adjustment factor N. This geometric and hydraulic accountability allows for more appropriate MOA design guidance recommendation with respect to sedimentation/treatment basins. The design element summary and performance criteria recommendations for sedimentation basin are discussed in Appendix B.3.1 Sedimentation Basin Design Elements Summary and Criteria Recommendations.

B.3 Design Methods Recommendations

B.3.1 Sedimentation Basin Design Criteria Recommendations

B.3.1.1 Introduction

This section outlines the required information and design recommendations for a sedimentation basin. The required inputs include: historic precipitation data, contributing basin area, land cover and drainage conveyance, and system sediment loading. The proposed design methodology is based on the probabilistic method accounting for both dynamic and quiescent settling and is the same method utilized in Appendix B.2.3. Combing the design inputs with the design methodology provides appropriate regulatory guidance for sedimentation basin design.

B.3.1.2 Sedimentation Basin Design Input Summary

The first design element for a sedimentation basin is the precipitation patterns within the Anchorage Bowl. A sedimentation basin is a volume-storage treatment device and the design is centered on a probabilistic method (EPA 1986). The inputs required for the method are located in Table B.17 below.

Table B.17: Sedimentation Basin Precipitation Statistics

Statistic	Variable	Unit	Value
Historical Mean Storm Intensity	i_m	in/hr	0.03
Historical Mean Storm Volume	v_m	in	0.24
Coefficient of Variation for Runoff flow rate	COV_{QR}	-	0.676
Coefficient of Variation for Runoff Volume	COV_{VR}	-	1.317

The values from this table are derived from a SYNOP analysis of 34 years of complete summer data for the Anchorage Bowl. A detailed explanation of this analysis can be found in Appendix C.1.1. Applying the storm intensity and volumes identified in Table B.17 with an appropriate contributing drainage area runoff model derives the next two design inputs: peak runoff discharge and total runoff volume. An outline discussing one method of hydrologic calculations which provides the peak runoff flow and total runoff volume is outlined in Appendix C.2.1 but other MOA approved hydrologic methods are available to determine peak discharge and total volume. These inputs are the next variables required for the sedimentation basin design methodology and are in Table B.18 located below.

Table B.18: Contributing Drainage Basin Characteristics

Peak Flow	Q_R	ft^3/s	
Runoff Volume	V_R	ft^3	

Note: This is an example table that would be populated for project specific design.

The final design input is sediment loading characterization. Based on the sediment transport analysis in Appendix C.1.1 and understanding the sediment removal within the drainage network, outlined in Appendix B.1.1, the appropriate sediment particle size distribution entering a sedimentation basin is located in Table B.19 below.

Table B.19: Recommended Sedimentation Basin influent PSD

Particle Size, mm	Percent Passing	Settling Velocities, fps
9.5	100.00%	177.455
4.75	100.00%	44.364
2	100.00%	7.865
0.84	100.00%	1.421
0.42	100.00%	0.355
0.25	100.00%	0.123
0.149	100.00%	0.044
0.105	98.15%	0.022
0.075	92.41%	0.011
0.0352	76.80%	0.002
0.0224	71.27%	0.001
0.0131	54.08%	0.0003
0.0066	44.29%	0.00008
0.0046	39.30%	0.00004
0.0032	39.03%	0.00002
0.0013	28.18%	0.000003
pan	0.00%	0.0000005

The influent PSD was also represented as 100 gram pollutant slug to determine the proposed sedimentation basin removal efficiency.

B.3.1.3 Sedimentation Basin Design Recommendations

A sedimentation basin treats water through dynamic (storm events) and quiescent (between storm events) means. The current DCM methodology only addresses the dynamic treatment process. The probabilistic method proposed accounts for both dynamic and quiescent settling.

Based on the most efficient project basin analyzed in 2012, certain geometric and hydraulic parameters are recommended to provide similar removal efficiencies. This section provides a sedimentation basin design procedure with recommendations regarding water quality performance and sediment removal.

The target removal rate for a sedimentation basin is 90% of the sediment load specified in the design inputs section above. The dynamic performance efficiency is primarily affected by the hydraulic loading rate (HLR = Q_R/A_{QR}) and the quiescent performance efficiency is primarily influenced by the total sedimentation basin volume to runoff volume ratio (V_B/V_R). A designer can manipulate both the sedimentation basin surface area (A_{QR}) and volume (V_B) to achieve the 90% removal efficiency. A minimum of 4:1 basin length to basin width ratio is recommended to provide minimal short circuiting and subsequently adequate sediment removal. Table B.20 below identifies discrete HLR and V_B/V_R combinations that provide a 90% removal efficiency using the 4:1 length to width ratio. This table assumes a N value of 5, and effective volume of 1, and a hydraulic efficiency of 1. Use Table B.20 with the input peak runoff discharge (Q_R) and input total runoff volume (V_R) to determine an appropriate surface area (A_{QR}) and basin volume (V_B) given the project site constraints.

Table B.20: Estimated Hydraulic Scenarios for an Overall Sediment Removal Rate of 90%

Hydraulic Scenario	HLR	VB/VR
1	6.87E-06	0
2	9.81E-06	0.362
3	1.23E-05	0.591
4	1.84E-05	1.190
5	2.45E-05	1.744
6	4.91E-05	3.351
7	7.36E-05	4.298

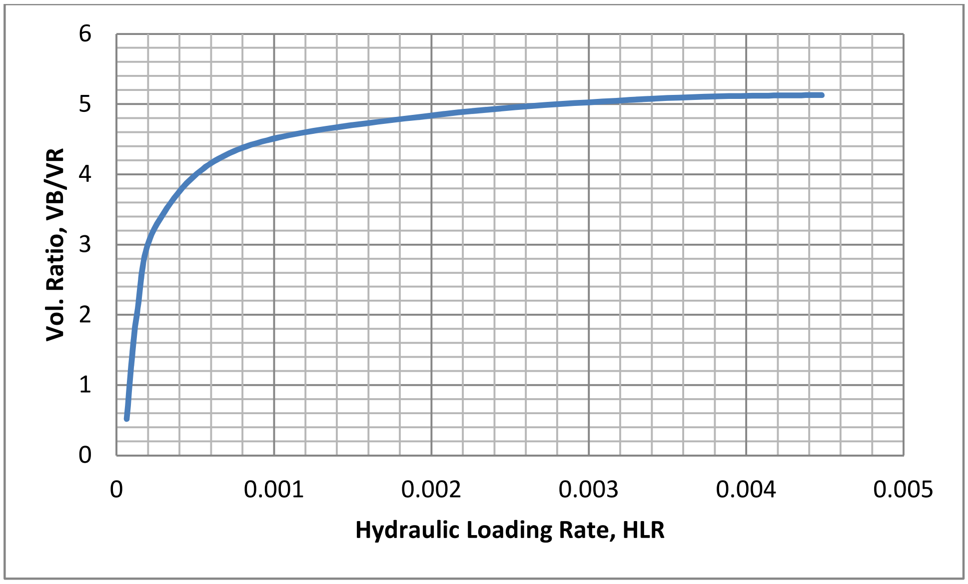


Figure B.6: Graphic Representation of Hydraulic Scenarios for 90% Removal Efficiency

Design the site specific basin geometry and process using the derived surface area and volume. Figure B.7 displays sedimentation basin configurations with a range of hydraulic efficiencies based on hydrodynamic studies and the C street basin configuration (Persson 2000, N. S. J. Persson 1999). These schematics are simplified to represent basic geometries and hydraulics.

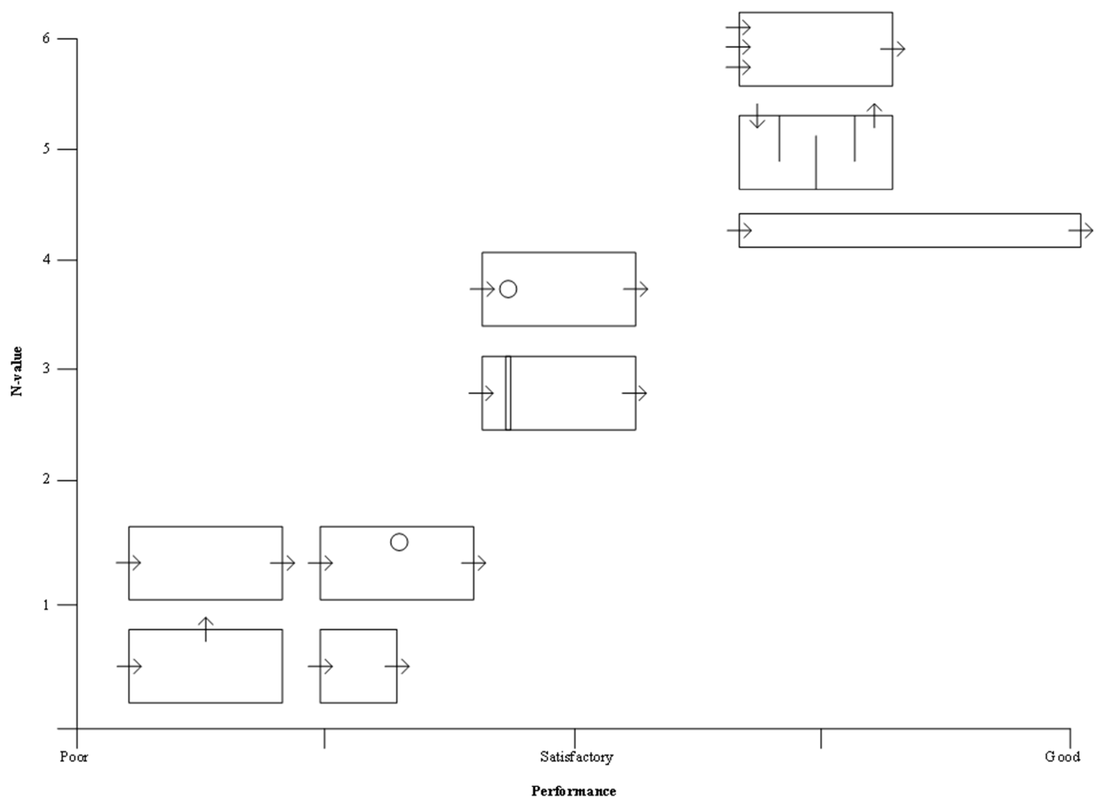


Figure B.7: Basin Configurations and Performance

The above schematics large N values (greater than 2) are intended as recommendations and variation from these sedimentation basin configurations is appropriate as long as data is provided to support good hydraulic efficiencies (Persson 2000) and minimal short-circuiting (Persson 2000).

During the final design process verify the sedimentation removal efficiency of the sedimentation basin with a hydraulic analysis using the recommended probabilistic analysis, as seen in steps 1-11 or Appendix B.2.4, to verify the sediment removal performance of the final basin design. Sediment diameter's smaller than 0.0013 mm should not be included in the removal efficiency analysis.

1. Input the values determined through the design process and calculate the mean removal efficiencies for each particle size based on the dynamic removal equation outlined below:

$$R_{md} = 1 - \left[1 + \frac{v_s}{n(HLR)}\right]^{-n}$$

$v_s = settling\ velocity\ for\ a\ specific\ particle\ diameter, fps$

$N = short\ circuiting\ adjustment\ factor. Assumed\ to\ be\ 3\ for\ design\ purposes$

$HLR = hydrualic\ loading\ rate, fps$

$R_{md} = mean\ dynamic\ removal\ efficiency, \%$

2. Calculate the maximum removal rate for each particle size using the following equation and inputs

$$Z = 1 - e^{-(\frac{v_s}{HLR})}$$

$v_s = settling\ velocity\ for\ a\ specific\ particle\ diameter, fps$

$HLR = hydrualic\ loading\ rate, fps$

$Z = maximum\ removal\ efficiency, \%$

3. Calculate the long term dynamic removal percentage for each particle size using the following equations and input.

$$R_{ld} = Z\left[\frac{r}{r - \ln\left(\frac{R_{md}}{Z}\right)}\right]^{(r+1)}$$

$Z = maximum\ removal\ efficiency, \%$

$COV_{QR} = 0.676;\ Coefficient\ of\ Variation\ for\ Runoff\ flow\ discharge$

$r = recipricoal\ square\ of\ COV_{QR}, r = \left(\frac{1}{COV_{QR}{}^2}\right)$

$R_{md} = mean\ dynamic\ removal\ efficiency, \%$

4. The long term dynamic removal for each particle size denotes the percent of particles that settle through dynamic conditions over the life of the structure if maintained properly. Average the long term dynamic sediment removal efficiencies and determine the fraction of particles not removed by dynamic settling.

$f_D = 1 - R_{ld\ average}$

$f_D = fraction\ of\ particles\ \ not\ removed\ through\ dynamic\ settling$

5. Determine the ratio of the basin volume divided by the mean storm runoff volume.

$$\frac{V_B}{V_R} = \frac{Sedimentation\ basin\ storage\ volume}{Mean\ storm\ runoff\ volume}$$

6. Calculate the emptying rate for each particle size for the sedimentation basin during inter-event periods.

$\Omega = v_s A_{QR}$

$\Omega = emptying\ rate$

7. Calculate the emptying ratio for each particle size sedimentation basin during inter-event periods.

$$E = \frac{\Delta_m \Omega}{V_R}$$

$$E = emptying\ rate\ ratio$$

$$\Delta_m = mean\ interval\ between\ 2012\ storm\ mid\ points$$

8. Use nomograph in Figure B.8 to determine the effective storage volume ratio for each particle size.

$$\frac{V_E}{V_R} = Effective\ Storage\ Volume\ Ratio$$

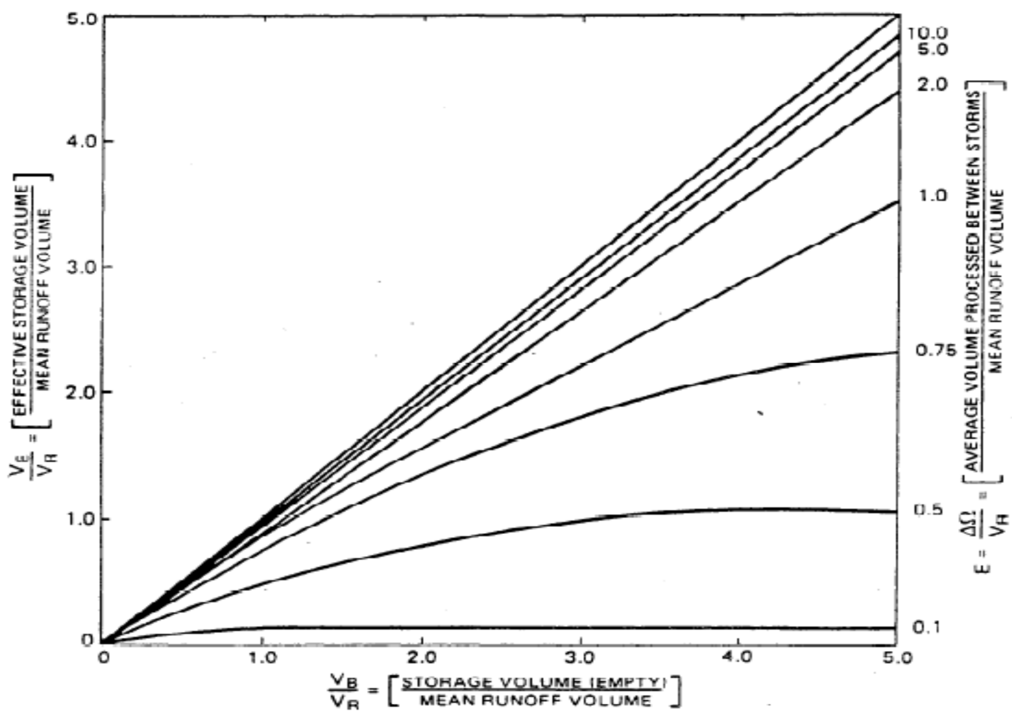


Figure 4. Effect of Previous Storms on Long-Term Effective Storage Capacity

Figure B.8: Effect of Previous Storms on Long-Term effective Storage Capacity

9. Use the nomograph in Figure B.9, the effective storage volume ratio and Coefficient of variance of runoff volume to determine the percent removal for each particle size.

$$COV_{VR} = 1.317, Coefficient\ of\ Variance\ for\ Runoff\ Volume$$

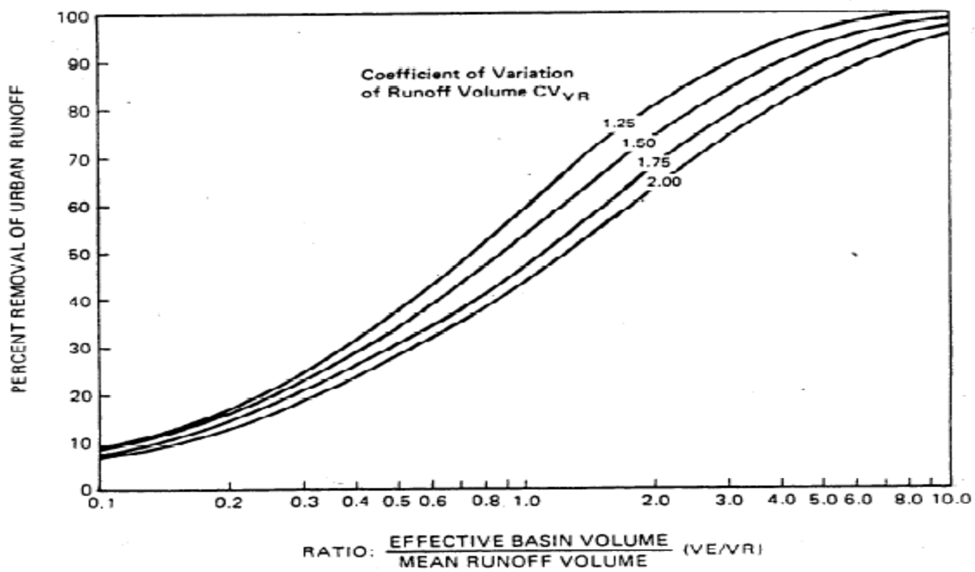


Figure 3. Average long term performance: volume device

Figure B.9: Quiescent Removal Nomograph

10. The quiescent removal for each particle size denotes the percent of that particle size that settles between storm events. Average the quiescent removal efficiencies and then determine the fraction of particles not removed by quiescent settling.

$$f_Q = 1 - R_{Q\ average}$$
$$f_Q = fraction\ of\ particles\ \ not\ removed\ through\ quiescent\ settling$$

11. Finally the overall sedimentation basin performance can be determined with the equation below.

$$Overall\ \%\ Removal = \left[1 - \left(f_Q \times f_D\right)\right] \times 100\%$$
$$f_Q = fraction\ of\ particles\ \ not\ removed\ through\ quiescent\ settling$$
$$f_D = fraction\ of\ particles\ \ not\ removed\ through\ dynamic\ settling$$

If the overall percent removal efficiency is not equal to 90% then modifications to the proposed design should developed and steps 1 through 11 should be completed until 90% removal efficiency is obtained.

Conclusion

Sedimentation basins are the final structure in regards to treatment of stormwater within MOA. These structures are intended to settle out spherical non charged particles that were not captured through hydrodynamic separators or other processes in the storm drain system. Sedimentation basins should have a larger removal rate and smaller target particle diameter when compared to

OGS. Based on the performance analysis is recommended that a properly designed sedimentation basin should remove 90% or more of the basin influent TSS and 75% or more of the 5 micron and greater particle diameters. The current design criteria methodology might need to be modified to account for hydraulic characteristics and quiescent settling to more accurately predict the performance removal of sedimentation basins.

B.3.2 OGS and Catch Basin Design Criteria Recommendations

As currently written, the design criteria for OGS in the Design Criteria Manual (DCM) Section 2.13 contains several components as follows:

Flow capacity is based on water quality protection parameters as laid out in Table 2-1 of the DCM which states:

Treat the initial 0.5 inches of post-development runoff from each storm

Provide water quality treatment rate for post development runoff at a minimum of 0.005 inches per minute.

Sediment storage capacity is addressed in DCM section 2.13B as twice the one year accumulation specified in Table 2-4.

Lastly, Water Quality Parameter are specified as 80% reduction of the inorganic sediment particles equal to or greater than 100 micron and 25% reduction of inorganic particles less than 100 micron.

Additionally the Alaska Depart of Environmental Conservation (ADEC) regulates oil and grit separators with the following directives:

"One of the design criteria for projects using oil and grit separators, is that to obtain an ADEC letter of non-objection for discharge to storm sewers, an applicant must demonstrate that the proposed oil and grit separator(s) has (have) the ability to remove at least 50 percent of particles 20 microns in size from storm water runoff during the 2–year, 6-hour rain event." (ADEC, 2009)

These requirements can be difficult to reconcile with one another and with the parameters and studies that are industry standards for testing OGS. The resultant ambiguity in design criteria could be removed and replaced with specific criteria based on industry testing standards, and rainfall runoff from a specific intensity rainfall event.

The following are recommended changes:

Most manufactures publish a flow rate capacity for their different units as each unit is generally a member of similarly designed but differently scaled devices. This published flow rate is the maximum discharge (Q) at which the device can effectively remove sediment from the flow without scouring previously trapped sediments. Above the maximum rate, the device or installation must have a bypass structure to route larger flow around the device and avoid rentrainment of sediments..

Designers use various methods for determining design flow, but all start with the input of a sustained rainfall intensity or storm event. The ADEC and DCM currently specify a Soil Conversation Service (SCS) defined 2 yr 6 hr storm with a specified hydrograph. This storm is an artificially generated event with very high peak rainfall intensity based on conditions that may or may not be applicable to Anchorage weather patterns. There is little guidance how to apply this storm event to runoff flows. The DCM and ADEC guidance shows a wide range in rainfall intensity parameters. In order of intensity:

Table B.21: Regulatory Rainfall Intensities

Regulatory Parameter	Rainfall Intensity (in/hr)
Peak 5 minute intensity of 2 year 6 hour storm event , 0.062 in/ 5 min	0.744
Minimum treatment rate from DCM 0.005 in/min	0.30
Peak hour intensity from 2 yr 6 hr storm	0.195
Mean intensity of 2 year 6 hour storm	0.088
Treat the first .5 inch runoff from each storm	Subject to interpretation?

For comparison, the NWS Anchorage hourly rainfall intensity data records dating back 45 years (394,200 records) contain less than 30 records of recorded rainfall greater than 0.2 in/hr. From this data, it is recommended that the capacity of OGS installations be based on a annual sum of loads approach, one that would treat the nationally accepted 90% of all storm flows from a given system. This would reduce interpretive ambiguity and give designers a specific rainfall intensity on which to base runoff analysis.

Based on a median annualized ranking of all hourly rainfall records, in Anchorage the intensity below which 90% of cumulative rainfall depth occurs is 0.12 in/hr. This data was truncated at a rainfall of 0.02 in/hr below which no recordable runoff occurred on the 2012 study basins. The following table shows the distribution of these 90% rainfall Median intensives.

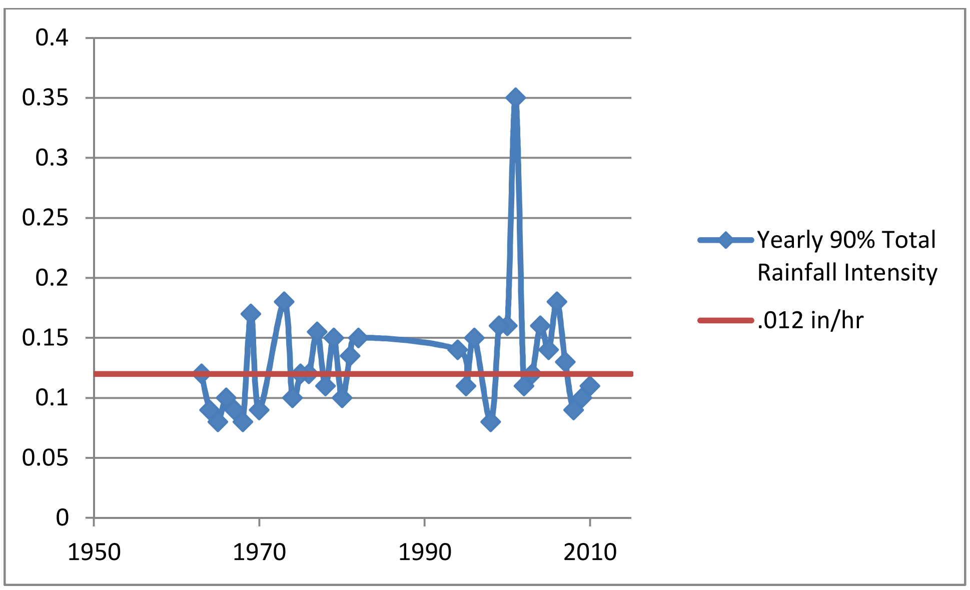


Figure B.10: Yearly 90 Percent Rainfall Intensities

For water quality we propose that the performance of OGS units be tied to an industry standard testing method. We propose that NJDEP certified/ NJCAT protocol (or similar) be used. These protocols govern testing with predetermined pollutant loading at specified PSDs

OGS design as laid out in the MOA DCM should be simplified and rewritten to take advantage of the NJDEP performance testing already being undertaken by most major manufactures. We recommend that OGS designs be based on the following parameters.

1. Documented NJDEP removal efficiency based on new testing protocol (due out in 2013). Allow scaling within brand specific families of devices as justified by Péclet number analysis. Require that the specified OGS meet specific removal efficiencies based on standardized testing using sediment with specified particle size distribution. 90% removal of NJDEP calculated yearly loading is the recommended target. Using a certified unit from within the brand family scale other devices within that same family using unit dimensions, particle settling and flow rates (the Péclet number). With this method it will be possible to size units for basin of varying sizes and characteristics using certified testing results.
2. Design treatment capacity based on modeled runoff flows at full build out and a specific rainfall intensity and orographic multiplier. Base this intensity on annualized weather records analysis utilizing the 90th percentile total depth. These numbers are significantly lower than the current peak intensities specified for the 2 yr 6 hr storm in the DCM. This lowering of design flows will lead to the installation of smaller cheaper units and should over time result in more OGS installations and better overall performance of the MS4. The larger sized units currently required by DCM storm intensities are not cost effective as the removal efficiency of most units plateaus above a Péclet number of 3 and/or 90% removal of bench tested sediment loads. (Wilson, et al. 2009)
3. Require momentum type high flow bypass structures (to prevent scour during large storm events) and 100% isolation valving with bypass piping to allow for maintenance in structures with base flow.
4. Adjust sediment storage capacity requirements. Base this capacity on a one year accumulation assuming 60%wash off of 4000 lb per curb mile, 8000 lb per curb mile w/connected parking and 1500lb/acre disconnected parking. Discount these accumulation rates for the 40% abstraction by properly designed and maintained catch basins.
5. Require direct access, sloped floors, low friction surfaces or other features to ease maintenance.
6. Require yearly inspection and cleaning when accumulated sediments are above 20% of the manufactures specified sediment storage capacity.

Adjust catch basin design as follows:

1. Increase sump depths to accommodate 40% capture of the 4000lb/8000lb/1500lb accumulation rates as listed in the OGS requirements. Yearly accumulation will be based on 900 feet of curb and/ or drained parking lot surface. Density of sediments 110 lb/ft$^{3.}$ Sump capacity will be measured to a point 10 inches below the outflow pipe invert. Maintain the current 18" minimum catch depth. Example: 900 feet of curb at 8000 lb/curb mile would produce 5 ft^3 of yearly sediment. A 4 ft diameter catch basin will hold 12.57 ft^3 per foot of depth. For this application the minimum 18 inch catch below the inverts would be required to meet yearly storage requirements.
2. Require all catch basins to be installed off line to prevent scour of accumulated sediments.
3. Require yearly inspection, clean when sediments depth is greater than 3 inches.

C. PROJECT DATA ANALYSES

C.1 MOA-Wide Mean Climate and WQ Hydrology

C.1.1 SYNOP Analysis of AIA historic Precipitation

The Synoptic Rainfall Analysis was conducted using National Weather Service (NWS) historic precipitation data for the summer months from the Anchorage International Airport rain gauge site. The data set used began 05/01/1963 and ended 10/31/2010 and only used uninterrupted data collection for analysis during the summer months (continuous measurements from May 1st to October 31st). The data was analyzed using the statistics module of Storm Water Management Model (SWMM) program v5.0.022 developed by the EPA (Environmental Protection Agency 2012). The final results describe Anchorage bowl precipitation patterns through rainfall event statistics that include variables: storm inter-event time, storm volume, storm duration and storm mean intensity.

C.1.1.1 Procedure

The following procedure outlines how the original data was modified and analyzed.

1. The original NWS data from rain gauge 500280 was formatted into a user input file (UIF) as required by the SWMM Statistics module.

The user input file contains only summer rainfall data for each calendar year that contained uninterrupted data from May 1st to October 31st.

The dataset was analyzed to produce unique events by choosing the inter-event time (delta, Δ) so that the coefficient of variance for the storm inter-event time is equal to 1 ($COV_{\delta} = 1$) (Driscoll, et al. 1989).

Statistics for the following variables were provided with the chosen inter-event time: delta (Δ, δ), volume (V, v), duration (D, d), and mean intensity (I, i).

C.1.1.2 Results

On an average summer (May 1st to October 31st) in the Anchorage Bowl, approximately 40 storms occur. The average storm lasts approximately 13 hours has a precipitation volume of approximately 0.24 inches at a mean intensity of 0.026 in/hr. Storms occur more frequently in August and September but on average are separated by approximately 91 hours (centroid of storm to centroid of storm) this equates to a 78 hour dry period(no measurable rainfall) between storms. The statistics shown in Table C.1 summarize the summer rainfall statistics within the Anchorage Bowl.

Table C.1: Historical Summer Rainfall Event Statistics 1963 to 2010

	Delta, hrs	Volume, in	Duration, hr	Mean Intensity, in/hr	Peak Intensity, in/hr
Minimum Value	13	0.02	1	0.01	0.01
Maximum Value	795	3.46	92	0.155	0.38
Standard Deviation	91.83	0.317	12.92	0.0178	0.048
Mean Value	91.05	0.241	13.170	0.026	0.0548
Coefficient of Variance	1.01	1.317	0.981	0.676	0.875
90th Percentile	201.5	0.60	30.00	0.048	0.120
75th Percentile	110.5	0.33	19.00	0.033	0.075
50th Percentile	58.0	0.12	9.00	0.022	0.045
25th Percentile	34.5	0.05	4.50	0.015	0.025
5th Percentile	20.0	0.03	2.00	0.010	0.014

C.1.1.3 Conclusion

Statistically defining the historical data for all summer rainfall events provides a benchmark for the 2012 precipitation year. In Appendix C.3 the 2012 precipitation year is analyzed using the same methodology and is compared to the statistics developed in the results section of this section.

C.1.2 Street Sediment Loading and Wash off Modeling

Objective

To examine and model the movement of sediment pollutant load from initial deposition on streets during winter to a sedimentation basin or receiving water. This examination is mostly based on established transport and settling equations with confirmation from field observations and previous studies.

C.1.2.1 Street Sediment Loading

Anchorage sediment loads on the stormwater system are found on streets and are initially deposited during the winter months as traction enhancing sands and fine gravel. These gravels vary in coarseness depending on the source and are applied at variable rates dependent on weather and type of surface. The Anchorage MS4 Street Sweeping Report for 2011 (Sediments 2012) analyzed and summarized the deposited street sediment loads after snowmelt. A similar study from 2001 looks at the loading of sediments in large commercial parking lots (P. L. Sediments 2001). The sediment loads after snowmelt runoffs are summarized in these reports and characterize the particle size distributions (PSD) and locations of a majority of particulates entering the system for the summer precipitation runoff events.

Sediment loads prior to spring sweeping are concentrated at deposition locations, near intersections and in main traffic areas, and may reach concentrations of 52,000 lb per curb mile. Average concentrations on larger streets are approximately 10,000 lb per curb mile and on residential streets approach 6000 lb/curb mile. Commercial parking lots average 14,400 lb/ acre.

Mobilization of the sediment loads primarily come from street sweeping, storm wash off and traffic mobilization, but are augmented by buildup parameters.

Post spring sweeping loads are relatively constant for all streets at 1000 lb/curb mile and 1440 lb/acre for commercial parking lots. Sediment loads decrease on all roadways as fall approaches, with the exception of arterial streets. Arterial streets maintain a relatively constant 1000lb/curb mile through the summer and fall sweeping events, while residential and collector streets decrease by 1/3rd and 2/3rd respectively. Commercial parking lots show the greatest decline, entering the fall with less than 1% of the initial spring loading. This may be due to more rigorous sweeping by of commercial establishments seeking to put forward a neat and clean appearance for consumers.

C.1.2.2 Washoff Modeling

Washoff modeling was completed for a hypothetical 300 foot section of street with an estimated contributing drainage area. Sediment of a known particle size distribution was distributed evenly along the gutter. The peak precipitation intensity from a one year recurrent storm (as defined by the historical SYNOP analysis, Appendix C.1) was applied to the contributing drainage area. The precipitation intensity and contributing drainage area produced a variable runoff and channel flow along the gutter which subsequently mobilizes the sediment in the gutter. The approximate percentage of sediment mobilized during these flows was determined using open channel hydraulics and shear stress equations.

<u>Procedure</u>

The 300 foot section of road was divided into 10 segments of 30 feet to determine the variable channel flow. The procedure outlines the calculations for one segment and then discusses how the segments interact within the 300' street section. The rational method was used to determine the hydrologic runoff for a 30 foot segment of roadway and other connected impervious areas:

$$Q = CIA$$

Q = Discharge Runoff (cfs)
C = 0.9, Runoff Coefficient
I = 0.225, Intensity of Rain (in./hr.)
A = 0.024 (for one segment), Area (acres)

The runoff coefficient was set to 0.9 which assumed the contributing drainage area was all impervious. The contributing drainage area for the segment was 20 feet of street, 8 feet of sidewalk, and direct connected impervious (DCI) area (roof and driveway areas) which was assumed to be 25% of the street and sidewalk area. The total contributing drainage area for the segment is 1,050 square feet or 0.024 acres. The one year hourly peak intensity was of 0.225 in/hr was developed from the historical rainfall analysis in Appendix C.1.1. Figure C.1 depicts the runoff process. The input and output flow variables outlined in the figure will be addressed at the end of this report section.

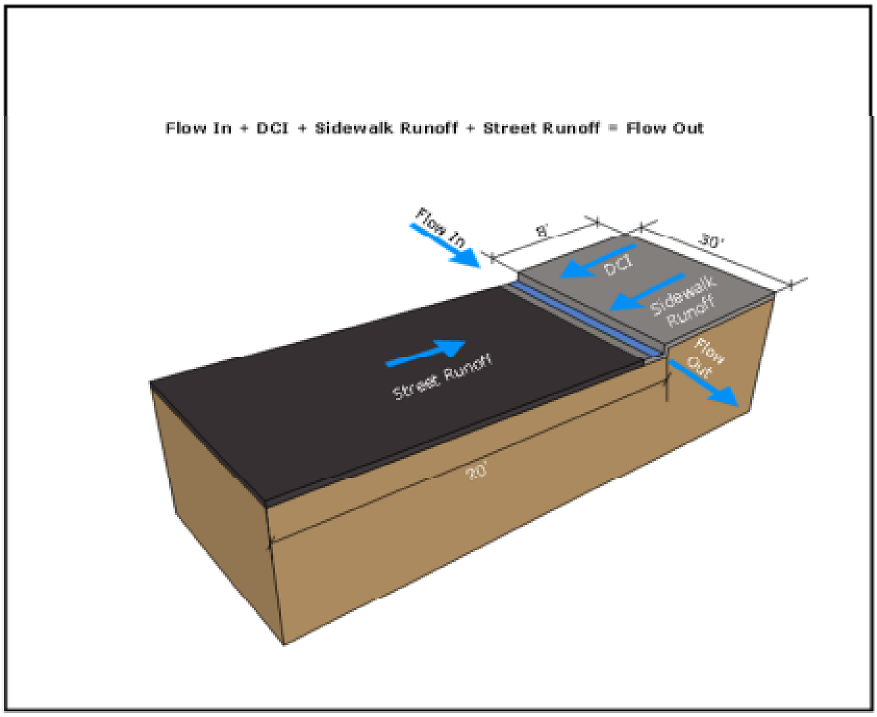


Figure C.1: Street Segment Runoff

These values produce a runoff discharge for the segment. This discharge is transformed into channel flow where manning's equation was utilized to determined the open channel hydraulic geometries.

$$Q = \frac{1.4875}{n} A R^{2/3} S^{1/2}$$

Q = channel flow calculated from rational equation (cfs)
n = 0.02 manning's coefficient
A = cross section area (ft^{2})
P_w = wetted perimeter (ft)
R = hydraulic radius, $\frac{A}{Pw}$
S = 0.01, Slope (ft/ft along the curb)

A manning's value of 0.02 was used for a concrete surface. The assumed average slope for the street was 1%. The cross sectional area and wetted perimeter are solved for by assuming a triangular channel with a 5% side slope (based on typical MOA curb dimensions) and assuming the channel flow is equal to runoff flow. The flow depth can be extracted from manning's equation and applied to the depth-slope equation to determine boundary shear stress along the channel bottom.

$\tau_w = \lambda_w SD$

τ_w = boundary shear stress (lb/ft^2)
λ_w = 62.4, unit weight of water (lb/ft^3)

S = 0.01, Slope (ft/ft)
D = Open channel flow depth (ft)

Solve for the boundary shear stress by using a street slope of 1%, a unit weight of water of 62.4 lb/ft^3 and a flow depth from manning's equation. The boundary shear stress is then compared to the critical shear stress for different particle sizes. The critical shear stress of each particle can be determined using multiple references including critical shear measurement the Corps of Engineers (Fischenich 2001) and/or utilizing shields parameter (Clark 2002). For coarser particles above #10 sieve the shields parameter is used. It is estimated that once the finer sediments are washed off, the particles that remain will be similar to a well sorted stream bed for which shields parameter is commonly used. Figure C.2 illustrates the open channel hydraulics, shear stress, and mobilization of particles in the gutter.

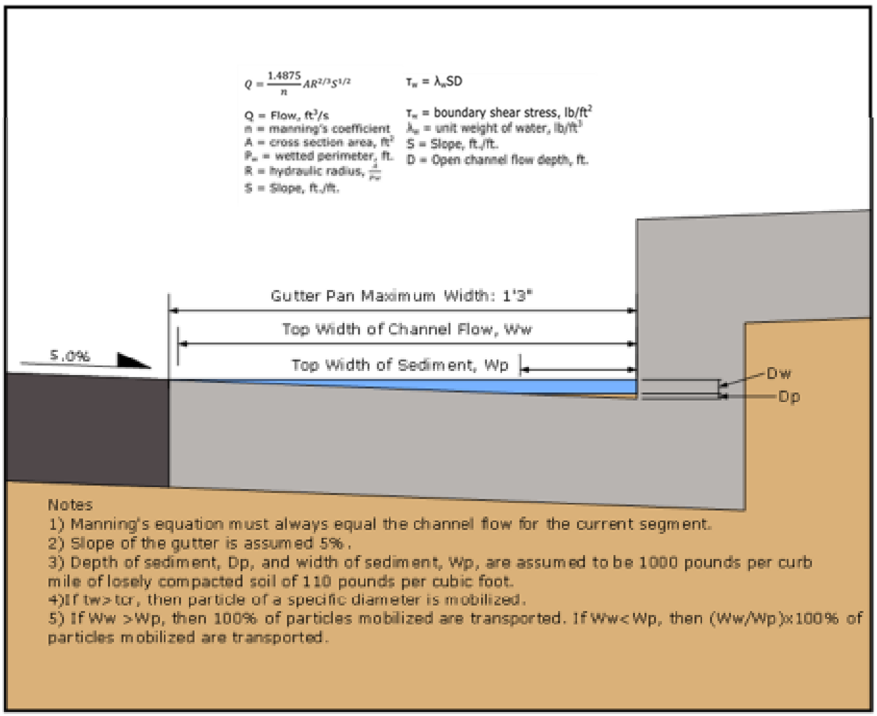


Figure C.2: Channel Flow Dimensions and Variables

If the boundary shear stress is greater than the critical shear stress then the particle is considered to be mobilized. When mobilized, the particle assumes the velocity of the flow and travels a distance equal to the velocity times the duration of the rainfall event (1 hour). This distance is then compared to the distance of the segment (30') and if it is greater than the segment distance the particle is transported to the downstream segment.

The flow of the downstream segment equals the runoff generated for that segment plus the additional flow from the upstream segment. This changes the hydraulic geometries, shear stress and particles mobilized. This process is continued in each of the connected segments (transporting flow from one segment to the next) until 300' is reached, which is the typical distance between catch basins.

Results and Conclusions

This model is not intended to give a precise and/or accurate estimate of the total sediment load entering the municipal separate storm sewer systems (MS4). It is believed that the total load entering the system over the course of a year may be several times greater than the 1000 lb/curb mile shown here because of the varying contributions from parking lots, street sweeping scheduling, sediment buildup rates, storm timing and intensity and other factors. These sediment build up and transport assumptions are to be verified by street sediment studies in 2013.

However, the washoff analysis indicated that the yearly peak storm intensity produces a concentrated curb flow with sufficient forces to mobilize approximately 97% of particles below the 3/8in particle diameter. It assumes that sediments are concentrated in the gutter pan through street sweeping, traffic and sheet flow forces. This modeling effort indicates that whatever street sediment is concentrated along the gutters after the spring melt and spring street sweeping can be moved into the storm drain system during the summer/fall rainfall period.

C.2 2012 Project Basins Analysis

C.2.1 2012 Project Contributing Basin Characterization Report

Characterizing the contributing drainage areas for the three project sedimentation basins is essential for a thorough hydrologic analysis of the basins. The characterization defines the drainage area based on extents of the conveyance system (storm drain system), appropriate surface runoff boundaries, land cover, depression storage, and the rational equation. The quantitative definition of the drainage area allows for hypothetical design storms simulation.

C.2.1.1 Procedure

To determine drainage basin characterization the following steps were followed.

1. Define the subbasin boundaries for the three project sedimentation basins (C Street, Minnesota Drive and Meadows Street). Each subbasin is located in the MOA Hydography Geo Database (HGDB). The subbasin boundary defines both the conveyance network and surface runoff boundaries associated with each project sedimentation basin.
2. Define land cover area in each subbasin using 2009 Municipality of Anchorage orthoimagery (Municipality of Anchorage Watershed Management 2009). The land cover values are divided into 8 categories: barren; indirectly connected impervious land surfaces; directly connected impervious land surfaces; street; wetland; lake; landscaped; forest.
3. Define the runoff coefficient for each land cover type identified in each subbasin using the MOA Rational Runoff Coefficients Identified in section 7.1.3 of the 2007 Drainage Design Guidelines (Municipality of Anchorage 2007).
4. Define depression storage for each land cover type using the MOA depression storage parameters as defined in section 7.1.1 of the MOA 2007 Drainage Design Guidelines
5. Determine peak runoff discharge, for the average storm event, for each drainage area by using the rational equation and mean storm intensity defined by the Appendix C.1: SYNOP Analysis of AIA Historic Precipitation.

$$Q_{Ri} = C_i I_m A_i$$

$$Q_{MR} = \sum_{i=1}^{n} Q_{Ri}$$
$$Q_{MR} = sum\ of\ all\ land\ cover\ flows\ for\ a\ subbasin, cfs$$
$$n = total\ number\ of\ land\ cover\ types\ within\ a\ subbasin$$
$$Q_{Ri} = flow\ for\ a\ particular\ land\ cover\ within\ a\ subbasin, cfs$$
$$C_i = runoff\ coefficient\ for\ a\ particular\ land\ cover$$
$$I_m = historical\ mean\ storm\ intensity, in/hr$$
$$A_i = area\ for\ a\ particular\ land\ cover\ with\ a\ subbasin, acres$$

6. Determine mean storm runoff volume for each drainage area by applying the mean storm volume and accounting for runoff volume loss from the depression storage.

$$V_{Ri} = C_i(V_m - D_{si})A_i$$
$$V_{MR} = \sum_{i=1}^{n} V_{Ri}$$
$$V_m = mean\ historical\ storm\ volume, inches$$
$$D_{si} = depression\ storage\ of\ a\ particular\ land\ cover\ within\ a\ subbasin, inches$$
$$V_{Ri} = Runoff\ volume\ for\ discrete\ land\ cover\ type, cubic\ feet$$
$$V_{MR} = Mean\ runoff\ volume\ for\ contributing\ drainage\ basin, cubic\ feet$$

7. Calibrate the depression storage and runoff coefficient variables for the 2012 mean precipitation event using time series rain gauge and time series discharge data gathered for the project sedimentation basins in the summer of 2012.

C.2.1.2 Results

The mean rainfall volume based on the 2012 summer precipitation records was 0.34 inches. This is 0.10 inches greater than the mean rainfall volume determined from the historical precipitation records. This storm volume was applied to the drainage basin characterization which provided for the total calculated runoff volume.

The mean rainfall intensity for the 2012 summer precipitation record was 0.028 in/hr and is approximately the same as the historical mean intensity. The intensity was used to provide the average storm peak discharge experienced by the project basins. Subbasin characteristics for the three project sedimentation basins are shown in Table C.1, Table C.2, and Table C.3. Appendix E.2 displays a map of the contributing basins.

Table C.2 C street Drainage Basin Characteristics

Land Cover Type	Area, A_i	Runoff Coefficient, C_i	Depression Storage, D_{si}	Land Cover Area Percent of Total
01STREET	68.560	0.820	0.100	8.13%
02DCI	279.386	0.820	0.100	33.13%
03ICI	69.579	0.750	0.200	8.25%
05FOREST	178.317	0.120	0.830	21.14%
06WETLAND	63.949	0.160	1.333	7.58%
07LANDSCAPED	183.526	0.200	0.200	21.76%

Table C.3 Minnesota Drainage Basin Characteristics

Land Cover Type	Area, A_i	Runoff Coefficient, C_i	Depression Storage, D_{si}	Land Cover Area Percent of Total
01STREET	77.833	0.820	0.100	13.42%
02DCI	265.630	0.820	0.100	45.81%
03ICI	37.309	0.750	0.200	6.43%
05FOREST	81.510	0.120	0.830	14.06%
06WETLAND	22.319	0.160	1.333	3.85%
07LANDSCAPED	93.808	0.200	0.200	16.18%
08LAKES	1.497	0.000	1.333	0.26%

Table C.4 Meadows Drainage Basin Characteristics

Land Cover Type	Area, A_i	Runoff Coefficient, C_i	Depression Storage, D_{si}	Land Cover Area Percent of Total
01STREET	49.436	0.820	0.100	13.11%
02DCI	125.012	0.820	0.100	33.14%
03ICI	49.311	0.750	0.200	13.07%
05FOREST	37.582	0.120	0.830	9.96%
06WETLAND	50.233	0.160	1.333	13.32%
07LANDSCAPED	65.637	0.200	0.200	17.40%

The runoff volume and discharge from a storm event is directly attenuated by the amount of impervious surface in a contributing drainage basin. Impervious surface in a drainage basin converts more rainfall into stormwater due to land cover characteristics such as low infiltration and depression storage and ultimately increase total volume and peak discharge into sedimentation basins. Impervious surface is also contributes a majority of the sediment as discussed in Appendix C.1.2.2.

C.2.1.3 Conclusion

The runoff volume and discharge from a storm event is directly attenuated by the amount of impervious surface in a contributing drainage basin. Impervious surface in a drainage basin converts more rainfall into stormwater due to land cover characteristics such as low infiltration and depression storage and ultimately increase total volume and peak discharge into sedimentation basins. Impervious surface also contributes a majority of the sediment because of the gravel and sand applied during winter for vehicle traction as discussed in Appendix C.1.2.2. The sediment from impervious surfaces is verified because sample PSD from road networks match closely to sediment extracted from OGS. During rainfall events these particulates become mobilized and enter the drainage network (Appendix C.1.2.2 Washoff Analysis).

C.2.2 2012 Project Treatment Basin Characterizations Report

The project sedimentation basins vary in performance due to contributing drainage areas and design features within each basin. This appendix describes the differences between each project basin site and the resultant hydraulic differences.

C.2.2.1 Procedure

Sedimentation basin characterization was conducted in the following steps.

1. Obtain as-built documents for each basin. Identify all inlets, outlets, treatment features, number of treatment cells and type of treatment cell.
2. Field inspect each sedimentation basin to determine existing condition status of all inlets, outlets, treatment structures, treatment cell connections and treatment cells.
3. Survey elevations using a local benchmark to determine the following features:
    a. inlet pipe elevation
    b. outlet weir and or pipe elevation
    c. elevation of water for each treatment cell
    d. depth of sedimentation basin along flow path
    e. Cross section geometries at critical hydraulic structures
4. Using as-built and field verified data calculations were made of the following sedimentation pond geometries
    a. Bottom of Pond Area (ft^2)
    b. Base flow operating Pond Area (ft^2)
    c. Effective Operating Pond Area (ft^2)
    d. Effective total operating volume (bottom of pond to water elevation) (ft^2)
    e. Stage-Discharge relationship for the outlet using the appropriate weir equation
    f. Stage-Surface Area relationship using aerial imagery, contours and as built drawing data.
    g. Stage-Storm Storage relationship (Storage based on volume above pond base flow elevation i.e. 0.0 storage at base flow elevation) using the stage-surface area relationship and average treatment cell depth.

C.2.2.2 Results

Appendix E.2 illustrates the three project basins and denotes the inlet and outlet stations.

C Street Sedimentation Basin

The treatment basin located at C Street contains three treatment cells with a sheet pile influent distribution structure and 40’ broad crested outlet weir. The treatment cells (in treatment order) consist of an open water upper pond, a wetland, and an open water lower pond where the broad crested weir is located. Of the three project basins, C Street is the largest in surface area and volume. Figure C.3 illustrates the project basin.

Table C.5 and Table C.6 illustrate the general basin characteristics along with the stage-performance summary.

Table C.5: C Street Basin Characteristics

	Upper Pond	Wetlands	Lower Pond
Bottom of Pond Area A_{bottom} (ft^2)	29,845	38,198	3,620
Base Level Area, A_B (ft^2)	93,410	131,101	24,712
Depth, (ft)	7.00	1.00	7.00
Total Volume, V_B (ft^3)	410,795	38,763	88,177

Table C.6: C Street Stage Performance Summary

Stage, ft	Sedimentation Basin Discharge, ft^3/s	Surface Area, ft^2	Storm Storage, ft^3
82.000	0.00000	156448.2	0.0
82.500	43.51920	160038.5	78726.2
83.000	131.32430	163628.8	157452.3
83.500	256.38400	167219.1	236178.5
83.999	417.65690	170802.2	314747.1

Minnesota Sedimentation Basin

The treatment basin located north of the Dimond Boulevard and Minnesota Drive intersection contains three treatment cells with a submerged multiple v-notch weir that outflows to a control culvert into Campbell Creek. The treatment cells (in treatment order) consist of an open water upper pond, a wetland, and an open water lower pond with the submerged v-notch weir. Additionally the wetland cell is bounded on the upstream and downstream end by a shallow rock weir extending the whole width of the pond. Minnesota is the second largest in surface area and volume of the study basins. Figure C.3 illustrates the project basin. Table C.7 illustrate the general basin characteristics along with the stage-performance summary in

Table C.8.

Table C.7: Minnesota Sedimentation Basin Characteristics

	Upper Pond	Wetlands	Lower Pond
Bottom of Pond Area A_{bottom} (ft^2)	5,697	23,755	2,299
Base Level Area, A_B (ft^2)	14963	24591	8109
Depth, (ft)	4.75	1.50	6.75
Total Volume, V_B (ft^3)	71,074.25	36,886.50	54,735.75

Table C.8: Minnesota Stage-Performance Summary

Stage, ft	Sedimentation Basin Discharge, CFS	Surface Area, SF	Storm Storage, CF
35	0.00000	43702.5	0.0
35.5	0.06993	45647.8	23831.5
36	0.39013	47593.0	47663.0
36.5	12.06777	49538.3	71494.5
37	39.65098	51483.5	95326.0

Meadow Sedimentation Basin

The treatment basin located north of the East 68th Avenue and Meadows Street intersection contains two treatment cells but only the upper cell was studied in an effort to quantify the performance of its simple pond structure. The studied cell contained one treatment cells with a partially submerged broad crested outlet weir. The treatment basin consists of a single open water treatment cell with a concrete broad crested weir discharging to twin corrugated metal arch pipe culverts. Meadow is the smallest in surface area and volume of the study basins. Figure C.3 illustrates the project basin. Table C.9 and Table C.10 illustrate the general basin characteristics along with the stage-performance summary.

Table C.9: Meadows Sedimentation Basin Characteristics

	Pond
Bottom of Pond Area A_{bottom} (ft^2)	3876
Base Level Area, A_B (ft^2)	24600
Depth, (ft)	5.00
Total Volume, V_B (ft^3)	63,735

Table C.10: Meadows Stage-Performance Summary

Stage, ft	Sedimentation Basin Discharge, CFS	Surface Area, SF	Storm Storage, CF
112.300	0.0000000000	24573.34	0
112.500	2.1099000000	25282.50	4985.415871
113.000	11.4487000000	27055.40	18063.09432
113.500	23.7012000000	28828.30	32007.02716

114.000	37.9299000000	30601.20	46804.78461

C.2.2.3 Conclusions

Sedimentation basin characteristics and performance curves define three conceptually different basins. Along with these variations, the contributing drainage basin runoff will provide unique hydrologic conditions for each site that contribute additional insight of the performance response of each sedimentation basin. Both the hydraulic and hydrologic variation supplies adequate insight to sedimentation basin performance and consequently provides support for project basin modification recommendations to improve sediment treatment (Appendix B.2.4) and design criteria recommendations for future sedimentation basin design and construction (Appendix B.3.1).

C.3 2012 Project Climate and WQ Hydrography

C.3.1 SYNOP Storm Event Analysis and Identification

In 2012 a rain gauge was placed by the influent station for the C street sedimentation basin. The gathered precipitation data was analyzed to determine independent rainfall events. The analyzed summer data begins 04/30/2012 and ends 10/05/2012. The dataset was analyzed using the SYNOP statistics module in the Storm Water Management Model (SWMM) program v5.0.022 developed by the EPA (Environmental Protection Agency 2011). The final results describe the 2012 Anchorage bowl precipitation patterns through rainfall event statistics that include variables: storm inter-event time, storm volume, storm duration and storm mean intensity.

C.3.1.1 Procedure

Precipitation data was gathered and analyzed using the following procedures:.

1. The Hobolink tipping bucket rain gauge was installed on 04/30/2012 at the influent station at C Street sedimentation basin. Data was downloaded for the season from the Hobolink website in excel format.
2. A SWMM formatted user input file was created from the excel spreadsheet and covered the time period of May 1st to September 30th. Rainfall measurements in the user input file reflect inches of rain accumulated in one hour.
3. The dataset was analyzed to produce unique events by identifying an inter-event time (delta, Δ) so that the coefficient of variance for the storm inter-event time is equal to 1 ($COV_{\delta} = 1$).
4. Modify the data set by removing all storms with a total storm volume of 0.01 inches. Storm events with this minimum volume do not produce numerically significant runoff and do not impact the storm runoff performance of sedimentation basins.
5. Provide statistics for the following variables with the chosen inter-event time: delta (Δ, δ), volume (V, v), duration (D, d), and mean intensity (I, i).

C.3.1.2 Results

A list of the 29 storms developed with a SYNOP analysis for the 2012 summer season is outlined in Table C.11 below.

Table C.11: List of 2012 Storm events and associated Storm Values

Event Number	Start Date	Total Volume, in.	Peak Intensity, in/hr	Mean Intensity, in/hr	Event Duration, hrs
1	5/24/2012	0.18	0.04	0.016	29
2	5/27/2012	0.1	0.03	0.017	6
3	6/3/2012	0.11	0.03	0.018	10
4	6/6/2012	0.05	0.02	0.017	3
5	6/12/2012	0.66	0.07	0.029	28
6	6/14/2012	0.02	0.01	0.01	10
7	6/25/2012	0.13	0.08	0.022	35
8	6/28/2012	0.18	0.06	0.036	6
9	7/3/2012	0.08	0.03	0.013	12
10	7/10/2012	0.02	0.02	0.02	1
11	7/12/2012	0.63	0.1	0.024	87
12	7/21/2012	0.74	0.11	0.031	36
13	7/30/2012	0.34	0.09	0.031	18
14	8/1/2012	0.04	0.02	0.013	4
15	8/2/2012	0.22	0.07	0.037	8
16	8/4/2012	0.02	0.01	0.01	12
17	8/6/2012	0.02	0.01	0.01	2
18	8/16/2012	0.09	0.04	0.022	5
19	8/18/2012	0.05	0.02	0.012	7
20	8/20/2012	0.3	0.06	0.025	30
21	8/23/2012	0.41	0.16	0.082	5
22	8/24/2012	0.02	0.01	0.01	2
23	8/26/2012	0.27	0.05	0.025	24
24	8/30/2012	0.47	0.08	0.025	55
25	9/3/2012	0.74	0.14	0.057	47
26	9/7/2012	0.15	0.14	0.075	2
27	9/12/2012	0.38	0.06	0.038	10
28	9/14/2012	1.05	0.18	0.03	77
29	9/18/2012	2.44	0.31	0.049	139

General statistical results from the 2012 summer rainfall events within the Anchorage Bowl are shown in Table C.12 and Table C.13.

Table C.12 2012 Summer Precipitation Statistics

	Delta, hrs	Volume, in	Duration, hr	Mean Intensity, in/hr	Peak Intensity, in/hr
Minimum Value	24.5	0.02	1	0.010	0.01
Maximum Value	623.5	2.44	139	0.082	0.31
Standard Deviation	113.01	0.486	31.33	0.0183	0.066
Mean Value	110.21	0.342	24.48	0.028	0.071
Coefficient of Variance	1.025	1.422	1.280	0.659	0.935

Table C.13: 2012 Precipitation Statistics Comparison

	Historical (1963-2010) Precipitation Statistics	2012 Precipitation Statistics
Avg. Storm Vol., inches	0.24	0.342
Avg. Storm Intensity, in/hr	0.026	0.028
Avg. Storm Duration, hr	13.17	24.48

C.3.1.3 Conclusion

The 2012 precipitation statistics can be directly compared to the historical summer precipitation statistics. The statistics summary in Table C.12 indicates storms during the 2012 summer had approximately the same average storm intensity, but because of the longer duration, had much greater average storm volumes (approximately 42% larger with respect to historical storm volumes). The rainfall events during September were specifically large and were exemplified with local flooding that occurred throughout Anchorage Bowl streams. When looking from a statistical stand point, the largest storm volume for 2012 was the 5th largest historical storm volume with respect to the historical precipitation analysis completed in Appendix C.1.1.

The basic 2012 precipitation analysis indicates the sedimentation basins experienced a larger volume of storm water than an average Anchorage summer. Understanding the measured performance of the sedimentation basin for 2012 in historical context supports hypothetical performance over the lifetime of these structures.

C.3.2 Storm Runoff Analysis and Identification

Storm runoff identification and analysis requires storms to be defined through statistical analysis (Appendix C.3.1) and field data gathered at project sedimentation basin inlets and outlets. The major variables used to indicate runoff events are storm precipitation, base flow, and TSS concentrations (if applicable to the particular storm). This appendix provides a complete data set of runoff events for each project sedimentation basin.

C.3.2.1 Procedure

1. Convert time series data gathered at each sedimentation basin inlet and outlets from depth of water to discharge (cfs) based on the weir and pipe dimensions at each site. Graph the inlet and outlet discharge-time series data for a sedimentation basin on the same chart.

2. Utilize one of the methods below to determine the beginning and end of a runoff event for the inlet and outlet stations: Straight-Line Method, Fixed-Base Method or Variable-Slope Method (Lindeburg 2001).
3. If none of these methods can be utilized, apply the following steps to determine the runoff beginning and endpoints.
4. Mark the beginning of each rainfall event (T_{01}+ T_{02}+ T_{03}+… T_{0n}) on the discharge time graph for the inlet station as defined by the analysis completed in Appendix C.3.1 (see Table C.14 in Appendix C.3.1). This time demarcates the beginning of the runoff event.
5. Identify the base flow for the inlet station for each sedimentation basin (Q_{bf1}+ Q_{bf2}+ Q_{bf3}+… Q_{bfn}).This occurs at the first precipitation of a storm.
6. Identify the final rainfall time (t_{r1}+ t_{r2}+ t_{r3}+… t_{rn}) for each precipitation event for the inlet station.
7. Identify the final peak TSS concentration time (t_{p1}+ t_{p2}+ t_{p3}+… t_{pn}) at the inlet station for a storm event (i.e. T_{0n}) which occurs before the next consecutive event (i.e. T_{0n+1}).
8. Identify the end time of a runoff event (T_{f1}+ T_{f2}+ T_{f3}+… T_{fn}) when the inlet station base flow has returned (Q_{bf1b}+ Q_{bf2b}+ Q_{bf3b}+… Q_{bfnb}) by adhering to the following conditions:

$$\begin{aligned}&If\ Q_{bfnb} = Q_{bfn}\ before\ the\ next\ storm\\ &and\ T_{fn} > t_{rn}\ and\ T_{fn} > t_{pn}\\ &then\\ &T_{fn}\ occurs\ at\ the\ time\ Q_{bfnb} = Q_{bfn}\end{aligned}$$

$$\begin{aligned}&If\ Q_{bfnb} \neq Q_{bfn}\ before\ the\ next\ storm\\ &then\\ &T_{fn}\ occurs\ at\ the\ time\ T_{fn} > t_{rn}\ and\ T_{fn} > t_{pn}\ and\ Q_{bfnb} \approx Q_{bfn}\end{aligned}$$

9. Determining T_{fn} for the second scenario may not be possible if Q_{bfnb} is significantly larger than Q_{bfn}. Apply engineering and scientific judgment to determine if the receding limb of a storm is complete before the beginning of the next runoff event. Apply steps 4 through 9 to outlet stations as well.
10. After identifying the beginning and end times for all runoff events and stations, analyze paired inlet and outlet stations and discard runoff events that contain any of the following inconsistencies.
    a. Missing data (discharge, TSS, or precipitation) for either the inlet or outlet stations.
    b. Corrupted data due to equipment malfunction
    c. Runoff events that produce numerically insignificant peak flows when compared to base flow
11. Provide a final list of accepted storms for each of the project sedimentation basins.

Results

Table C.14 indicates which storm during the summer of 2012 had complete data with minimal errors.

Table C.14: Verified 2012 Runoff Events

Event Number	Complete Storm Data C Street	Complete Storm Data Minnesota	Complete Storm Data Meadows
1	Yes	Yes	Yes
3		Yes	Yes
5	Yes	Yes	
7		Yes	Yes
8	Yes	Yes	Yes
11	Yes	Yes	
12	Yes	Yes	
13	Yes	Yes	
15	Yes	Yes	
18	Yes		
19	Yes		
20	Yes		
21	Yes		Yes
23	Yes		
24	Yes		
27	Yes	Yes	
28	Yes	Yes	

The most common cause for incomplete data at the sties was instrumentation malfunction, maintenance error, or insignificant data measurements. Examples of these errors include erratic measurements of turbidity or depth; incomplete installation after maintenance procedures; and no significant runoff discharge after low volume storm events (usually ≤0.02 inches). Since all parts of the continuous measured data were used in the storm by storm analysis, incomplete data sets within storm events would make the storm unusable.

Meadows had the fewest storms due to worse than normal malfunctions of the outlet instrumentation.

Table C.15, Table C.16 and Table C.17 further detail the runoff events for each basin by identifying the storm peak mode (number of peak flows within a single runoff event), the peak flow and the peak TSS concentration.

Table C.15: C Street 2012 Runoff Event Characteristics

Runoff Event	Peak Mode	Peak Flow, cfs	Peak TSS Concentration, mg/L
1	Unimodal	4.29	392.30
3	Unimodal	3.08	411.27
5	Multimodal	9.85	543.72
9	Unimodal	10.47	515.07
10	Unimodal	8.43	516.04
14	Multimodal	9.28	340.00
15	Multimodal	15.50	319.00
17	Bimodal	6.29	201.69
19	Unimodal	12.00	266.38
22	Unimodal	4.37	534.23
23	Unimodal	1.04	38.90
24	Multimodal	7.95	288.81
25	Unimodal	18.75	358.58
27	Multimodal	12.03	298.93
28	Multimodal	9.47	608.57
31	Unimodal	32.74	271.14
32	Multimodal	48.47	289.96

Table C.16: Minnesota 2012 Runoff Event Characteristics

Runoff Event	Peak Mode	Peak Flow, cfs	Peak TSS Concentration, mg/L
1	Unimodal	3.98	899.09
3	Bimodal	3.80	279.77
5	Multimodal	9.89	424.21
9	Multimodal	3.03	452.36
10	Unimodal	8.11	448.51
14	Multimodal	13.87	855.32
15	Multimodal	14.36	605.04
17	Unimodal	9.31	399.16
19	Bimodal	12.70	467.29
31	Unimodal	13.95	682.22
32	Multimodal	19.79	556.57

Table C.17: Meadows 2012 Runoff Event Characteristics

Runoff Event	Peak Mode	Peak Flow, cfs	Peak TSS Concentration, mg/L
1	Unimodal	2.26	377.67
3	Unimodal	2.07	85.33
5	Multimodal	6.50	188.19
9	Multimodal	3.10	277.20
10	Unimodal	5.30	166.00
25	Unimodal	12.96	398.18
31	Unimodal	10.52	279.26
32	Multimodal	17.27	159.17

Of these three basins C Street had the largest runoff flows with moderate TSS concentrations. Minnesota had moderate runoff flows with the highest TSS concentrations. Finally meadows had the lowest flows and TSS concentrations.

Conclusions

The runoff analysis and identification is the preliminary steps required for the hydraulic characterization and the relationship between precipitation storms and runoff events. Hydraulic characterization utilizes runoff identification because unique runoff events need to be identified before multiple hydraulic characteristics can be calculated. Runoff identification also provides a feedback loop to the 2012 precipitation analysis (Appendix C.3.1) and helps identify which storm event volumes and intensities should be classified as actual storm events with respect to stormwater.

C.4 2012 Project Treatment Device Performance

The parameters of temperature, conductivity, water depth, and turbidity were collected using a YSI Sonde 600 OMS V2 set to collect at 15 minute intervals at each station. Total suspended solids (TSS) was tested by collecting grab samples and then analyzed by SGS Laboratories in Anchorage, Alaska. All sample collection was done according to the MOA QAPP and standard sampling methods. The MOA QAPP can be found in Appendix D and equipment maintenance practices can be found in Appendix E.

Data Quality Validation was completed on all water quality and hydrology data collected for the project. Continuous data collected from YSI sondes were compared to calibration data recorded during each calibration event. Data was then adjusted with the three point calibration data for turbidity and calibration data for electrical conductivity. During calibration in the field, water depth was set to the staff gage on site.

In the office, once data had been downloaded from the YSI sondes, data points were noted in the database. Due to sensor malfunctions large portions of data were unusable for analysis at Meadow Street Down. Other site sensors would randomly miss readings. The largest problem was with turbidity data. There were several times when the probe wiper was over the sensor while the sensor was actively taking a reading. These points were removed from the data set before analysis occurred.

Laboratory analyzed data was validated using relative percent difference (RPD) between primary and duplicate samples collected in the field for precision as defined in the QAPP and are found in Table C.18 (Municipality of Anchorage 2012). Data was also validated based on laboratory matrix spikes (MS) and matrix spike duplicates (MSD) as well as laboratory control spikes (LCS) and laboratory control spike duplicate (LCSD).

Table C.18 RPD for Field Samples

PARAMETER	QAPP Precision	4/5	4/9	5/5	5/24	6/6	6/12	6/13	7/21	7/22	7/30	8/2	8/2 (b)	8/16	8/20	9/5	9/12
TSS	25	**48**	3	0	5	1	6	1	4	**26**	3	5	4	1	1	2	1
BOD	NA	14					54										3
FC	60	6					19										2

Although there were two samples that were higher than the QAPP RPD for TSS, they were not rejected. Stormwater quality can be highly variable from one point to the next due to the system design, run off rates, and intensity of storms. Therefore the elevated RPDS are believed to reflect the heterogeneity of stormwater quality rather than the precision of the sampling.

Once data was validated, continuous data was uploaded to the MS4 monitoring database and each station data sets were compiled into single files to be exported for data analysis.

C.4.1 NTU/TSS/Flow Quantitative Correlation Analysis

Turbidity and discharge data collected from the YSI Sonde was paired with TSS results from grab samples analyzed in the laboratory. If sample times did not match directly, turbidity and discharge readings that were closest to the sample collection time were used. Once data points were compiled for each station, a multiple linear regression was used to determine the relationship between TSS, turbidity, and discharge (Wagner, et al. 2006). Each station was analyzed separately as each station has unique characteristics for determining the relationship between TSS and turbidity (Thackston and Palermo 2000).

Multiple Linear Regression (MLR) was performed in the statistical package SPSS (IBM Corp. Released 2010). Residuals were tested for normality. If normality was not obtained, variables were log transformed and retested. Once normality was achieved, the MLR was run to determine a trend line that was significant at 0.05. Based on the R^2 value, strength of the relationship between variables was determined (higher the value stronger the relationship). The results are as follows:

Table C.19 TSS formula based on MLR

Station	Formula	R^2	p value
C STREET UP (CSTUP)	TSS=3.995+(.638*Turbidity)+(.386*Discharge)	0.760	.000
C STREET DOWN (CSTDOWN)	TSS=4.105+(.667*Turbidity)	0.445	.001
MINNESOTA UP (MINNUP)	TSS=-.008+(.622*Turbidity)+(.357*Discharge)	0.655	.000
MINNESOTA DOWN (MINNDOWN)	TSS=-11.821+(.792*Turbidity)	0.620	.000
MEADOW STREET UP (MDWUP)	TSS=-.033+(.649*Turbidity)+(.311*Discharge)	0.784	.000
MEADOW STREET DOWN (MDWDOWN)	TSS=1.01+(.453*Turbidity)+(.642*Discharge)	0.630	.031

The relationship between the variables for each station was found to be strong enough to decently predict TSS based on turbidity and discharge with C St Down having the lowest capability. Twenty-four samples were used to create the relationship for each station except for Meadow Down with only ten samples. A potential increase in the R^2 value could be obtained by collecting more data over a longer period of time. The Meadow Down equation shows decent capability to predict TSS, however, with a lack of samples for analysis the formula could drastically change with the addition of samples. It is important to note that these formulas are site specific and are not to be used at other sedimentation basins. In order to create turbidity and discharge conversion to TSS, data points would need to be taken at the specific sedimentation basin in question and a regression created (Thackston and Palermo 2000).

Once MLR was completed for each station and determined to have a relatively strong relationship and be significant ($p<0.05$), the trend line formula from the analysis was used to determine TSS values from collected turbidity and discharge points without an associated TSS laboratory sample at each station.

C.4.2 Mass Transport Analyses, Including Stratified Event Plots

C.4.2.1 Sum of Loads Mass Transport Approach

Once TSS was determined using the regression equation, mass transport analysis was completed. First the amount of TSS transported throughout the entire study was calculated including TSS found at baseflow. The amount of TSS entering the sedimentation basin, the amount leaving the basin, and the amount retained during both summer and spring can be found in the following tables.

Table C.20 Mass TSS transport through Sedimentation Basins

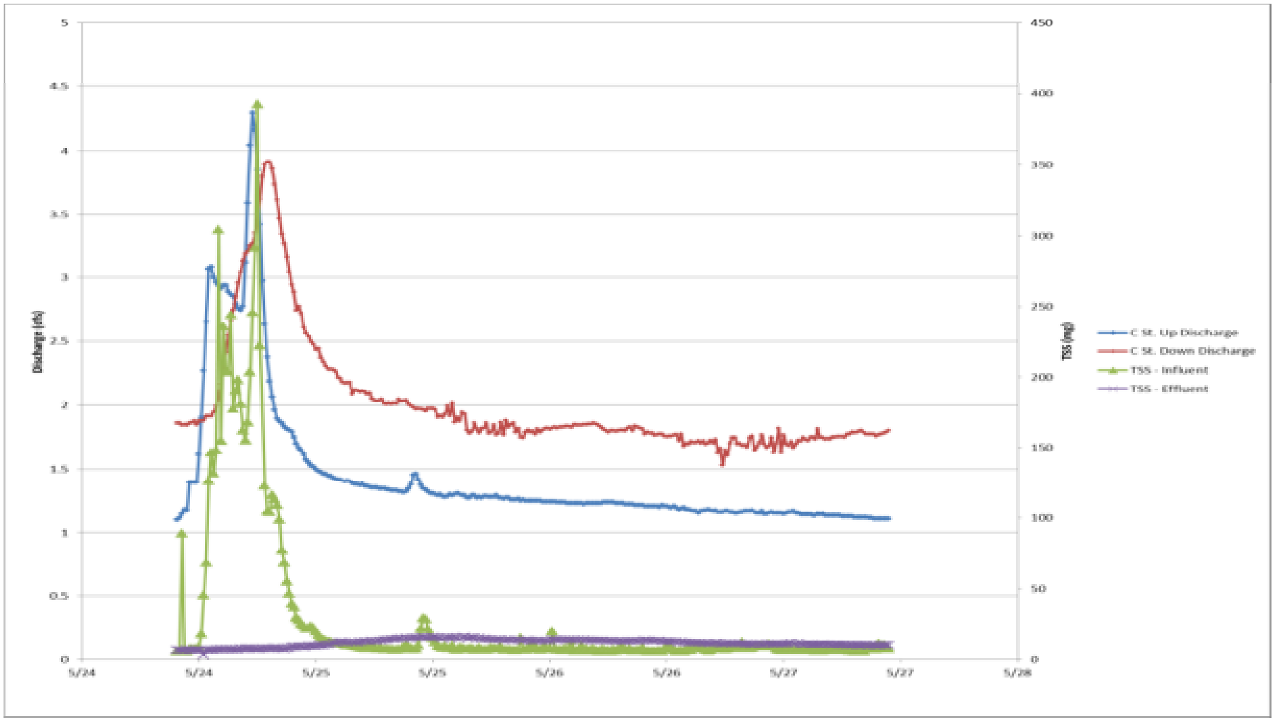
	TSS Influent (cy)	TSS effluent (cy)	TSS retained (cy)(%)
Spring			
C Street	5.34	2.93	2.41 (45.14)
Meadows	1.39	1.16	.23 (16.26)
Minnesota	8.57	5.68	2.89 (33.76)
Summer			
C Street	13.54	4.64	8.9 (65.85)
Meadows	2.87	2.30	0.57 (19.86)
Minnesota	6.78	3.77	3.01 (44.4)

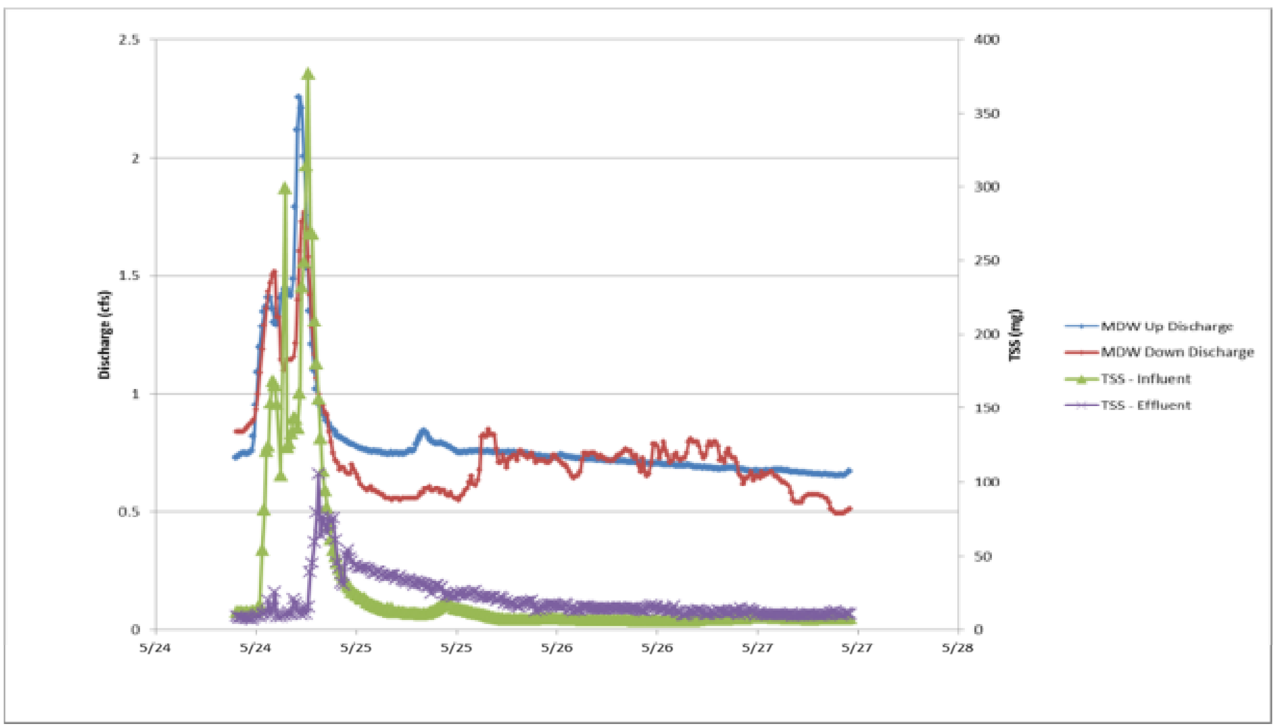
The TSS values shown in Table C.20 were calculated using continuous data that had a value for both the influent and effluent stations. There were gaps in the data when a probe was not properly functioning. Therefore, when data was not present for a particular time at a station, data from the associated station was not used. The removal rates for sedimentation basins are realistically represented by only using validated influent- effluent paired data. It is possible that more sediment load was retained during the summer months based on in field observations at both Meadow and Minnesota Street basins, but the percent retained is potentially unaffected.

C.4.2.2 Individual Storm- based Mass Transport Approach

The variety of summer storms that occur in the Anchorage Bowl was considered during this study. The specific storm size and intensity could potentially effect how sedimentation basins collect, retain, and scour TSS. Specific storms from the approximate 29 that occurred were chosen for analysis based on completeness of data and differences in storm mechanics. For each storm analyzed, the event mean concentration of TSS was calculated along with peak concentration.

To determine sedimentation basin response to specific storms, visual queues are used to characterize each site. Figure C.3 shows the first storm of the year (May 24) for each sedimentation basin. This storm had one major peak and returned to base flow and each sedimentation basin TSS removal rate is shown to be different. C Street and Minnesota Drive sedimentation basins TSS effluent shows a minimal amount of TSS moving out of the sedimentation basin after the peak. Whereas, Meadow Street sedimentation basin had a peak in the TSS effluent towards the end of the peak in the TSS influent.





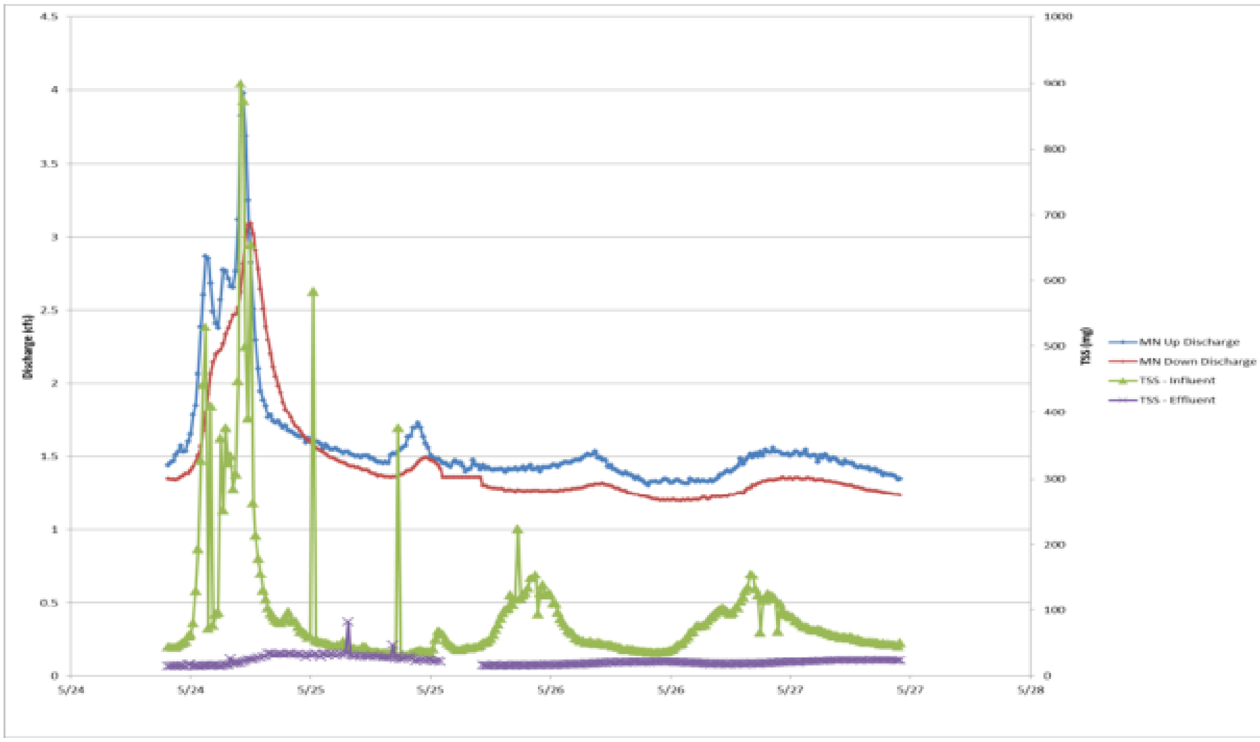
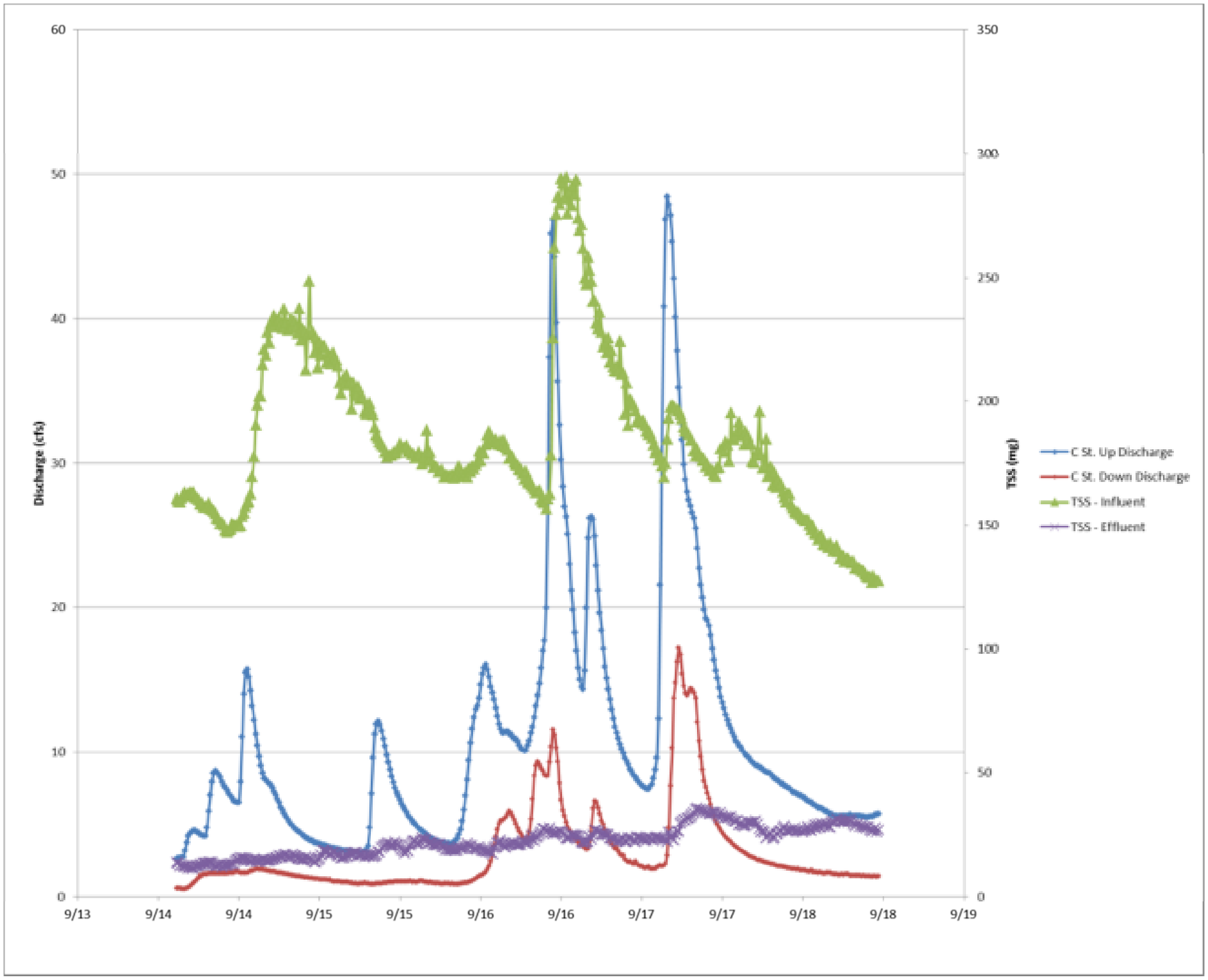
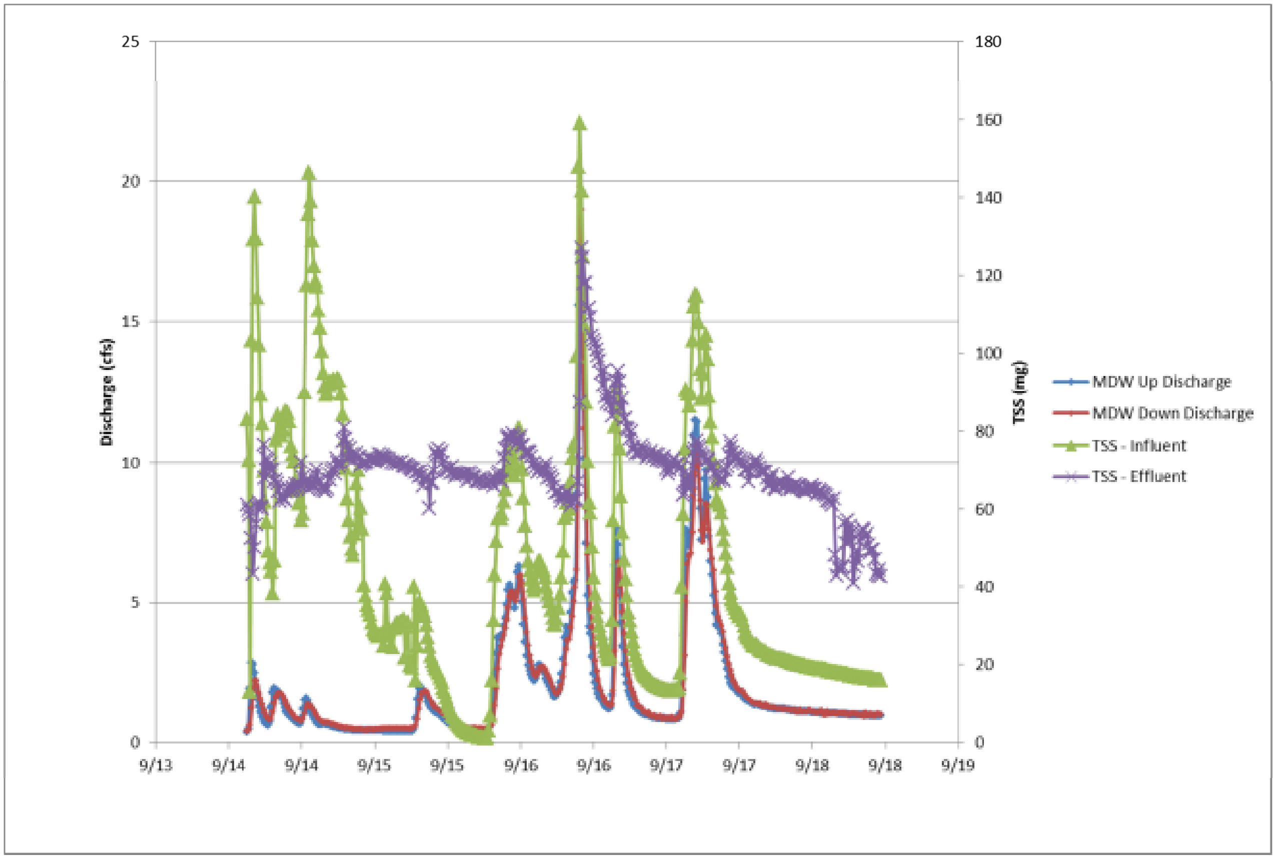


Figure C.3 Storm 1 Hydro and Pollutographs

Another type of storm to occur in the Anchorage area is one that has many peaks or multi-modal. Due to the multiple peaks, TSS influent also showed multiple peaks at all three sedimentation basins as seen in Figure C.4. However, the responses vary. The C Street sedimentation basin TSS effluent only started to increase after the two largest discharge peaks. TSS effluent at Meadow Street, although lower than the influent, matched the peaks of the influent. In many instances the effluent TSS is larger than the TSS influent. The TSS at both sites for Minnesota Drive sedimentation basin followed the hydrograph trends, but TSS effluent was greatly reduced when compared to Meadow Street.


60
50
40
30
20
10
0
Discharge (cfs)
350
300
250
200
150
100
50
0
TSS (mg)
9/13
9/14
9/14
9/15
9/15
9/16
9/16
9/17
9/17
9/18
9/18
9/19
C St. Up Discharge
C St. Down Discharge
TSS - Influent
TSS - Effluent


Discharge (cfs)
TSS (mg)
25
20
15
10
5
0
180
160
140
120
100
80
60
40
20
0
9/13
9/14
9/14
9/15
9/15
9/16
9/16
9/17
9/17
9/18
9/18
9/19
MDW Up Discharge
MDW Down Discharge
TSS - Influent
TSS - Effluent

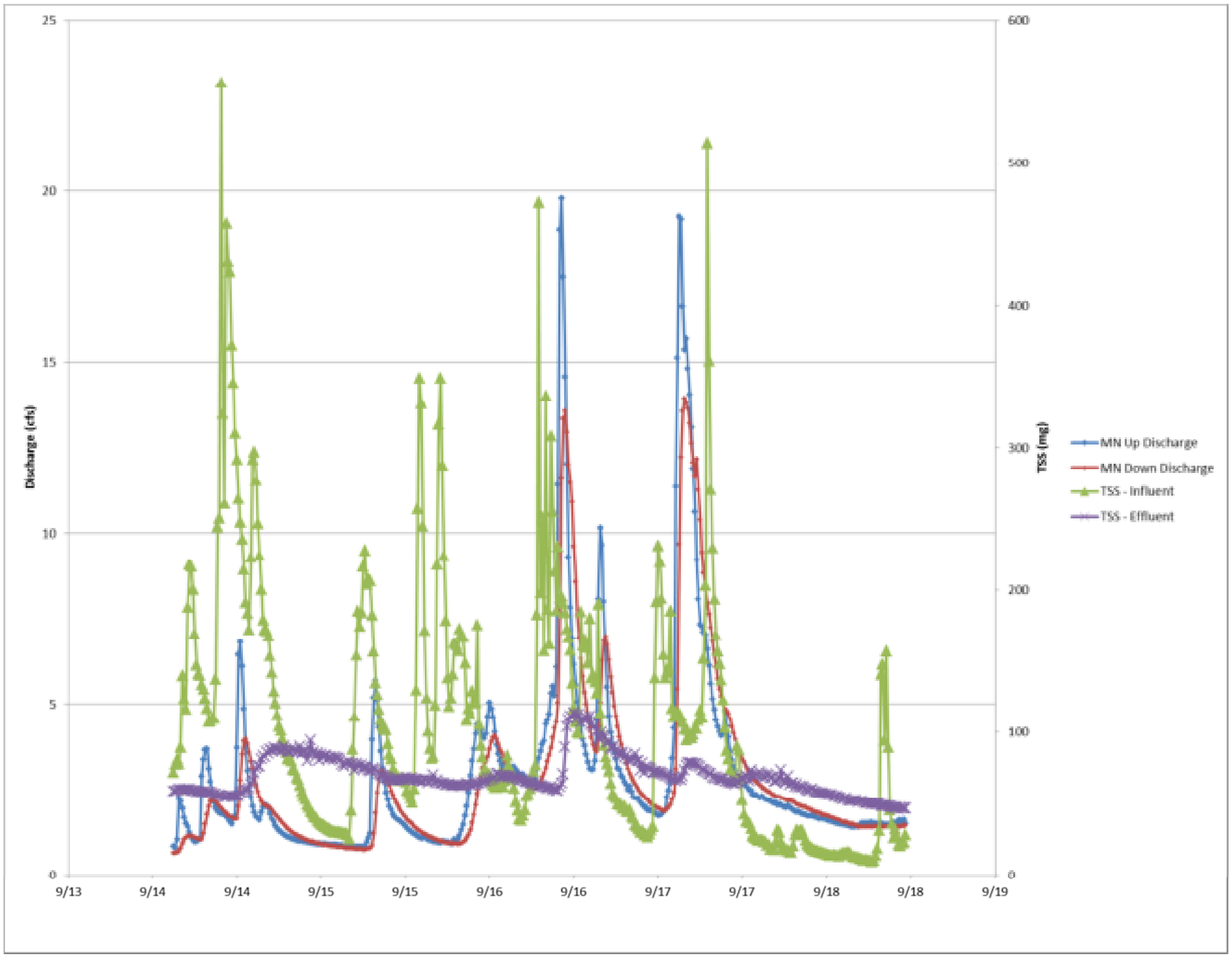


Figure C.4 Storm 32 Hydro and Pollutograph

C.5 PH, DO, BOD, Fecal Coliform, Organics, and Temperature

The water quality parameters of temperature, pH, and dissolved oxygen (DO) were tested in the field and results are discussed in this section. Biological oxygen demand (BOD), fecal coliform, and organics were tested by collecting grab samples and then analyzed by SGS Laboratories in Anchorage, Alaska. All sample collection was done according to the MOA QAPP and standard sampling methods. The MOA QAPP can be found in Appendix D and equipment maintenance practices can be found in Appendix E.

C.5.1 pH

During storm sampling in which samples were collected for laboratory analysis, pH and dissolved oxygen (DO) were also collected with a YSI 556 multiprobe. Probes were calibrated according to manufacturer's instructions before entering the field. When obtaining readings the probes were placed in the stormwater outfall behind the weir where the probe remained until readings were steady or five minutes, whichever came first. The readings were then recorded on the field form.

Table C.21 shows pH results during spring breakup sampling. All stations pH remained in the neutral range for a majority of spring break up. However, on the April 9 event, there were four stations that had an acidic pH. The values are not alarming as snowmelt and rainwater are naturally acidic (< 5.0 for snow and 5.6-5.7 for rainwater) due to reactions with normal levels of carbon dioxide in the atmosphere (U.S. Environmental Protection Agency 1980).

The more interesting results are how the sedimentation basin treats pH. The C Street basin pH decreases from influent to effluent during all three sampling events. Meadow Street basin pH decreases during the first event from influent to effluent, but increases during the next two events. Finally Minnesota Street basin pH remains the same for the first event and increases during the next two events. The April 9 event pH values are likely due to higher runoff from snowmelt and April 5 event occurring at the very beginning of breakup with less snowmelt runoff influence. The May 4 event was on the tail end of the breakup event and less snowmelt runoff once again and pH values for flow returning to neutral.

Table C.21 pH at Stations during Spring Break up

Station Name	4/5/2012	4/9/2012	5/4/2012
CSTDOWN	7.14	5.35	7.20
CSTUP	7.23	6.57	7.30
MDWDOWN	6.38	5.01	6.37
MDWUP	6.41	4.15	6.14
MINNDOWN	6.85	6.46	6.90
MINNUP	6.85	4.78	6.58

Summer stormwater pH values can be found in Table C.22. Most of the pH values are in the neutral range. However there are occasional values that are in the acidic range and are mostly seen in the influent of the sedimentation basins. The effluent for a majority of the sampling events was in the neutral range.

Table C.22 pH at Stations during Summer Storm Events

Station	5/24	6/6	6/12	6/13	6/25	6/28	7/12	7/15	7/16	7/21	7/22	7/30	8/2a	8/2b	8/16	8/20	8/23	8/30	9/5	9/12
CSTDOWN	7.31	7.74	7.4	7.69	7.77	7.84	7.89	7.83	7.8	6.9	7.56	7.17	7.50	7.27	7.48	7.39	7.58	7.75	7.54	7.84
CSTUP	7.3	6.81	3.8	7.7	7.79	7.8	7.85	7.78	7.85	7.13	7.66	*	7.9	6.99	5.83	6.86	7.56	7.56	7.66	7.72
MDWDOWN	6.79	6.22	5.9	7.29	7.68	7.37	7.9	7.45	7.4	7.13	7.26	*	7.44	6.37	7.3	6.58	7.42	7.32	7.34	7.5
MDWUP	6.68	5.31	6.35	7.28	7.62	7.39	7.41	7.25	7.35	6.73	7.26	*	7.40	6.1	5.73	6.4	7.33	7.33	7.15	7.39
MINNDOWN	7.08	7.15	6.07	7.48	7.77	7.67	7.67	7.73	7.56	5.68	7.36	*	7.57	7.55	6.43	6.99	7.55	7.48	7.45	7.72
MINNUP	6.78	5.37	5.4	7.48	7.4	7.52	7.49	7.57	7.63	5.64	7.51	3.75	7.58	2.32[a]	5.06	6.03	7.49	7.03	7.75	7.67

[a] denotes value is suspect

C.5.2 DO

Dissolved oxygen (DO) concentrations and saturation taken during storm sampling can be an indicator of a pollutant load be present. The lower the DO concentration, the higher the probability that there is either a chemical or biological component to the stormwater that is consuming oxygen either for a chemical reaction or for respiration. DO concentrations and saturations during spring breakup are shown in Table C.23

Table C.23 Spring Breakup DO concentrations and saturations by Station

	4/5		4/9		5/4	
Station	DO (mg/L)	DO (%)	DO (mg/L)	DO (%)	DO (mg/L)	DO (%)
CSTDOWN	10.88	75.4	9.4	65.7	13.79	111.7
CSTUP	11.39	81.3	9.5	72.5	15.65	118.3
MDWDOWN	9.81	71.4	8.21	60.6	13.31	103.4
MDWUP	10.43	74.1	9.06	69.9	14.07	106
MINNDOWN	9.86	70	8.79	65.2	13.97	113
MINNUP	11.95	85.5	9.52	70.2	15.2	117.6

The DO concentration and saturation for each sampling event are typically seen during spring break up. The first two sampling events show DO is about the same concentration, when water temperatures are still cold and most water is still in the form of ice. The saturations are low because the colder the temperature of the water the higher the amount of dissolved oxygen it can hold. As the temperatures of the water start to rise the lower the holding capacity of the water as seen in the May 4 event. The DO concentrations have increased, likely due to ice free conditions, and super-saturation is occurring due to warmer water temperatures (Dodds 2002).

During rain events in summer, sampling occurred around the clock depending on when the storm began and when the peak flow was taking place. Teams sampling in late evening and early morning used a different sampling probe that only collected pH measurements. This was due to the need for a simplier device during low light conditions. Therefore, there are many sampling events when DO measurements were not taken and are not shown in Table C.24.

All sedimentation basins showed a general trend of higher DO concentrations in influent than effluent and is typical and expected. The influent water is entering the sedimentation basin from a pipe that is sloped and is being exposed to oxygen. The water entering the sedimentation basin will likely increase in DO concentrations when entering the sedimentation basin since most of the outfalls are designed to allow the water to fall into the basin from a few feet above the water surface of the basin creating turbulence that allows DO to enter the water. The effluent is likely lower in DO concentration due to the sedimentation basin causing water detention and as water stands DO concentrations decrease over time.

Table C.24 Summer Storm Events DO Concentration and Saturation by Station

	5/24		6/12		6/6		7/21		7/30		8/2		8/20	
Station	**DO (mg/L)**	**DO (%)**	**DO (mg/L)**	**DO (%)**	**DO (mg/L)**	**DO (%)**	**DO (mg/L)**	**DO (%)**	**DO (mg/L)**	**DO (%)**	**DO (mg/L)**	**DO (%)**	**DO (mg/L)**	**DO (%)**
CSTDOWN	8.7	80.4	9.2	83	10.07	93.1	7.2	70.4	7.78	70	7.69	72.6	6.63	63.7
CSTUP	9.48	79.7	8.75	78	10.67	90.9	8.06	81.6	7.61	71.8	8.12	75.2	7.73	73
MDWDOWN	8.32	69.6	9.64	85	10.52	86	7.05	68.2	8.14	81.2	7.28	67	7.35	68.7
MDWUP	9.28	75.2	8.9	80.5	10.3	87.3	7.47	75.5	7.74	72	8.64	80.5	7.41	69.7
MINNDOWN	9.16	80.6	9.19	80	10.42	91.5	78.6	72.1	9.34	84.2	7.55	68.5	7.2	66.5
MINNUP	9.31	80.7	5.4	87	10.38	89.2	7.28	72.5	9.32	82	8.96	82.2	9.04	83

C.5.3 BOD

Biochemical Oxygen Demand (BOD) was sampled during three events throughout the study and is an index of how much oxygen is being demanded by materials that are biodegrading in the waterbody (Brooks, et al. 2003). The BOD tested during this study is known as BOD_5 which relates to the amount of carbonaceous demand. The BOD can also be used in assessing stream pollution and for comparison purposes.

Table C.25 Biological Oxygen Demand Concentrations by Station

	4/5	6/12	9/12
Station	**BOD (mg/L)**	**BOD (mg/L)**	**BOD (mg/L)**
CSTDOWN	6.02	U	U
CSTUP	6.16	7.99	4.76
MDWDOWN	6.40	5.43	5.35
MDWUP	4.75	8.37	5.26
MINNDOWN	6.21	2.95	4.08
MINNUP	7.47	10.70	3.58

U denotes Non-detect.

The results in Table C.25 show C Street sedimentation basin BOD decreased from influent to effluent. Meadow Street sedimentation basin had no clear pattern with increasing concentrations at the beginning of spring breakup to decreasing in June and slightly increasing in September. Minnesota Street sedimentation basin BOD concentration decreased from influent to effluent during the first two events, but then showed a slight increase in September. The highest values seen were during the June event.

C.5.4 Fecal Coliform

Fecal coliform was sampled during the same three events as BOD. Many streams in Anchorage, AK are listed as impaired due to fecal coliform. Therefore, determining if the sedimentation basin could potentially remove fecal coliform from storm water before it enters the stream would be useful. The three sampling event results are shown in Table C.26.

Table C.26 Fecal coliform results by Station

	4/5	6/12	9/12
Station	**FC col/100 mL**	**FC col/100 mL**	**FC col/100 mL**
CSTDOWN	42	440	56
CSTUP	240	636	488
MDWDOWN	96	4800	294
MDWUP	18	33800	378
MINNDOWN	246	2800	417
MINNUP	310	5700	292

Fecal coliform counts are the highest during the June sampling event. This is likely due to spring breakup releasing frozen material in snow and ice packs as well as the introduction of more wildlife (waterfowl) being in the area and making use of the sedimentation basins. In September, the fecal coliform counts return to lower counts as seen in early April.

C.5.5 Organics

C.5.5.1 DRO

Diesel Range Organics (DRO) a pollutant that is monitored in stormwater programs was monitored at both the influent and effluent of all three sedimentation basins. Instead of using grab samples taken during storm events, Gore sorber technology was used. Gore sorbers are passive collection devices that are placed in the water column being tested. The sorbers collect the analytes present in the water over a period of time. Analytes, when coming in contact with the sampler membrane, are partitioned out of solution, and diffused through the sampler membrane for sorption by the engineered adsorbents. Once sorbers are returned to the Gore Laboratory they are analyzed using thermal desorption-gas chromatography/mass spectrometry instrumentation following EPA method 8260. In this case, each sorber was attached near the YSI Sonde probes behind the weir in each outfall. The mass of DRO found in the sorber are shown in Table C.27.

Table C.27 Diesel Range Organic Mass Quantities by Station

	4/3		4/26		6/22		9/24	
Station	**DRO (ug)**	**Days in Water**	**DRO (ug)**	**Days in Water**	**DRO (ug)**	**Days in Water**	**DRO (ug)**	**Days in Water**
CSTDOWN	9.03	57	4.24*	23	3.09*	94	3.06	15
CSTUP	12.15	57	7.59*	23	4.53*	94	5.85	15
MDWDOWN	16.74	57	13.34*	23	6.07*	94	2.07	15
MDWUP	15.6	57	9.07*	23	8.76*	94	9.03	15
MINNDOWN	12.64	57	6.21*	23	3.12*	94	1.04	15
MINNUP	20.32	57	5.04*	23	6.68*	94	0.79	15

*Denotes trip blank was found to have detectable concentrations of DRO therefore values are high due to atmospheric contamination.

The trip blanks for DRO during the 4/26 and 6/22 sets of sorbers was found to be contaminated. The 4/26 trip blank had a result of 2.58 ug and the 6/22 trip blank result was 0.65 ug. Contamination could have occurred during shipment of the samples, during setting or pulling of the sorbers, even though measures were taken to not contaminate the sorbers, or for unknown reasons. Therefore, the mass values found in sorbers during these sampling periods are biased high due to contamination. The data is therefore rejected and should not be used in the analysis of DRO concentrations in the sedimentation influent or effluent.

The Gore laboratories are able to determine concentration in the water column by taking the mass of analyte found in the sorber, exposure time, and sampling rate for the analyte of interest. The sampling rate is obtained from controlled chamber experiments that take into consideration temperature, relative humidity, flow rate and vapor concentrations. This information is plugged into a formula to get the specific concentrations in the water column (See Appendix C.5 for Concentration Method Calculation Summary for Gore Module). The concentration results are shown in Table C.28 for each sample period.

Table C.28 DRO concentrations by Station

	4/3		4/26		6/22		9/24	
Station	**DRO (ug/L)**	**Days in Water**	**DRO (ug/L)**	**Days in Water**	**DRO (ug/L)**	**Days in Water**	**DRO (ug/L)**	**Days in Water**
CSTDOWN	1.44	57	1.05*	23	0.30*	94	1.72	15
CSTUP	1.60	57	0.72*	23	0.37*	94	2.28	15
MDWDOWN	1.73	57	0.93*	23	0.41*	94	1.42	15
MDWUP	1.79	57	0.78*	23	0.51*	94	2.70	15
MINNDOWN	1.55	57	0.66*	23	0.31*	94	1.07	15
MINNUP	1.87	57	0.60*	23	0.45*	94	0.95	15

C.5.5.2 GRO

Gasoline range organics (GRO) was also monitored by the use of the Gore sorber method. Trip blanks did not show any contamination for this analyte and therefore all the results are valid for use in analysis. Table C.29 includes GRO results for mass quantities in each sorber. Near the end of the rain season GRO was below detection limits (bdl) for a majority of the stations and the highest quantities were found during breakup season.

Table C.29 Gasoline Range Organic Mass Quantities by Station

	4/3		4/26		6/22		9/24	
Station	**GRO (ug)**	**Days in Water**	**GRO (ug)**	**Days in Water**	**GRO (ug)**	**Days in Water**	**GRO (ug)**	**Days in Water**
CSTDOWN	7.17	57	bdl	23	0.57	94	0.53	15
CSTUP	6.75	57	1.31	23	0.9	94	bdl	15
MDWDOWN	1.76	57	2.75	23	bdl	94	bdl	15
MDWUP	1.71	57	4.23	23	bdl	94	bdl	15
MINNDOWN	1.34	57	0.78	23	0.5	94	bdl	15
MINNUP	1.07	57	0.6	23	2.11	94	bdl	15

Table C.30 shows GRO concentration calculation results from the formula above. Concentrations were low and below detection limits towards the end of the season.

Table C.30 GRO concentrations by Station

	4/3		4/26		6/22		9/24	
Station	**GRO (ug/L)**	**Days in Water**	**GRO (ug/L)**	**Days in Water**	**GRO (ug/L)**	**Days in Water**	**GRO (ug/L)**	**Days in Water**
CSTDOWN	1.30	57	0.42	23	0.14	94	0.81	15
CSTUP	1.25	57	0.34	23	0.19	94	bdl	15
MDWDOWN	0.66	57	0.47	23	0.14	94	bdl	15
MDWUP	0.70	57	0.57	23	0.15	94	bdl	15
MINNDOWN	0.60	57	0.27	23	0.14	94	bdl	15
MINNUP	bdl	57	bdl	23	0.28	94	bdl	15

C.5.5.3 TPH

Total petroleum hydrocarbons (TPH) is the summation of all petroleum hydrocarbons tested and mass quantities in sorbers are found in Table C.31. These quantities are over the period of days shown in the table. Due to DRO concentrations found in the trip blanks, TPH concentrations for the events 4/26 and 6/22 also have high bias and should not be used for analysis. Therefore, to compare to AWQS concentrations must determined and are shown in

Table C.32. The AWQS for Total Aqueous Hydrocarbons (TAqH) is 15 µg/L. The results from the Gore sorber show that the values found in the influent as well as effluent are below the standard.

Table C.31 Total Petroleum Hydrocarbon Mass Quantities by Station

	4/3		4/26		6/22		9/24	
Station	**TPH (ug)**	**Days in Water**	**TPH (ug)**	**Days in Water**	**TPH (ug)**	**Days in Water**	**TPH (ug)**	**Days in Water**
CSTDOWN	14.6	57	4.62*	23	3.54*	94	3.46	15
CSTUP	17.39	57	8.63*	23	5.25*	94	6.06	15
MDWDOWN	18.1	57	15.52*	23	6.46*	94	2.1	15
MDWUP	16.92	57	12.43*	23	9.13*	94	9.27	15
MINNDOWN	13.69	57	6.83*	23	3.52*	94	1.22	15
MINNUP	21.15	57	5.52*	23	8.36*	94	0.94	15

Table C.32 TPH concentrations by Station

	4/3		4/26		6/22		9/24	
Station	**TPH (ug/L)**	**Days in Water**	**TPH (ug/L)**	**Days in Water**	**TPH (ug/L)**	**Days in Water**	**TPH (ug/L)**	**Days in Water**
CSTDOWN	1.76	57	1.09*	23	0.32*	94	1.81	15
CSTUP	1.87	57	0.76*	23	0.40*	94	0.92	15
MDWDOWN	1.79	57	0.99*	23	0.42*	94	1.43	15

MDWUP	1.85	57	0.9*	23	0.52*	94	2.73	15
MINNDOWN	1.61	57	0.69*	23	0.33*	94	1.14	15
MINNUP	1.90	57	0.63*	23	0.50*	94	1.03	15

C.5.5.4 BTEX

Benzene, toluene, ethylbenzene, and xylenes make up the BTEX group that was also tested using the Gore sorber method. Below in

Table C.33 mass quantities from four sorbers are shown.

Table C.33 BTEX Mass Quantities by Station

	4/3		4/26		6/22		9/24	
Station	**BTEX (ug)**	**Days in Water**	**BTEX (ug)**	**Days in Water**	**BTEX (ug)**	**Days in Water**	**BTEX (ug)**	**Days in Water**
CSTDOWN	1.25	57	0.25	23	0.31	94	0.15	15
CSTUP	2.07	57	1.87	23	0.25	94	0.23	15
MDWDOWN	0.38	57	0.25	23	0.12	94	0.05	15
MDWUP	1.7	57	0.37	23	0.25	94	0.3	15
MINNDOWN	0.97	57	3.28	23	0.17	94	0.25	15
MINNUP	3.36	57	0.64	23	0.91	94	0.2	15

Table C.34 shows BTEX concentration results with highest concentrations in the water column being found during spring break up.

Table C.34 BTEX concentrations by Station

	4/3		4/26		6/22		9/24	
Station	BTEX (ug/L)	Days in Water	BTEX (ug/L)	Days in Water	BTEX (ug/L)	Days in Water	BTEX (ug/L)	Days in Water
CSTDOWN	2.09	57	0.51	23	0.21	94	0.63	15
CSTUP	3.30	57	1.84	23	0.19	94	0.92	15
MDWDOWN	0.65	57	0.30	23	0.09	94	0.21	15
MDWUP	2.75	57	0.43	23	0.20	94	1.21	15
MINNDOWN	1.54	57	2.99	23	0.14	94	0.99	15
MINNUP	4.68	57	0.69	23	0.67	94	0.80	15

C.5.6 Temperature

The water temperature was recorded in 15 minute intervals using the YSI Sonde at each station. The results can be seen in Figure C.5 for the influent stations. Figure C.6 shows temperatures recorded at the effluent station for each sedimentation basin. The trend in the influent temperature is closely mimicked by the effluent stations and is expected.

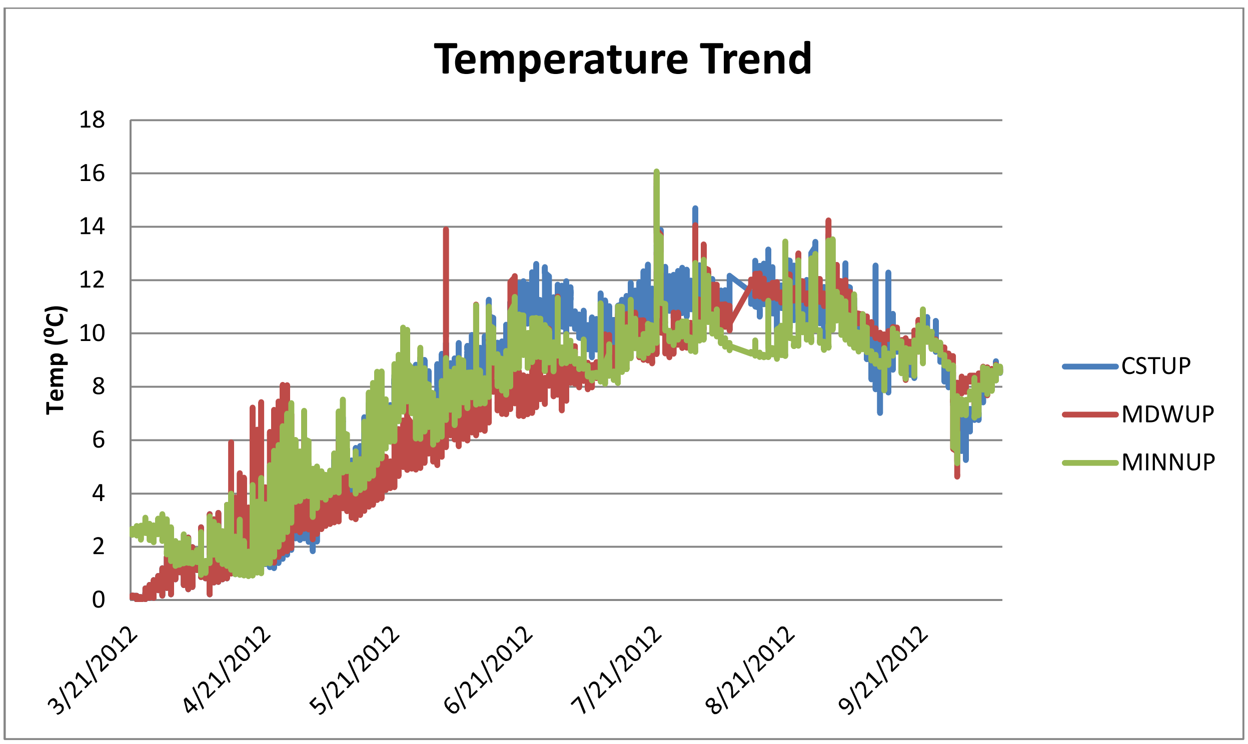


Figure C.5 Temperature at Influent Stations at each Sedimentation Basin

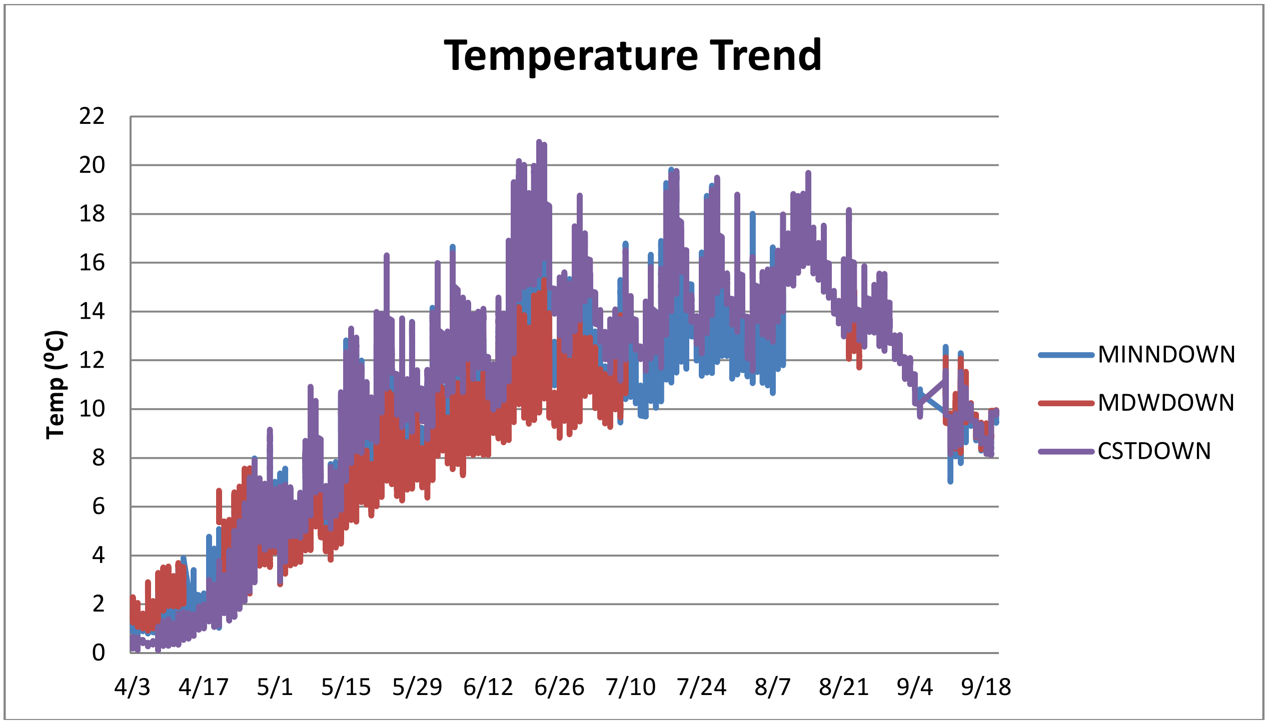


Figure C.6 Temperature at Effluent Stations at each Sedimentation Basin

The AWQS for aquaculture is that temperature may not exceed 20°C at any time. C St Basin effluent temperature was over this standard at a few points in June. Although stormwater is not directly regulated by the AWQS, the water from the effluent in some cases is flowing into streams and mixing. Therefore, comparison with AWQS is a good benchmark, but does not show any violations.

Another AWQS for aquaculture is for spawning areas in which temperature cannot exceed 13°C. In this case, a majority of the open water season temperatures where higher than this AWQS, but once again does not result in a violation.

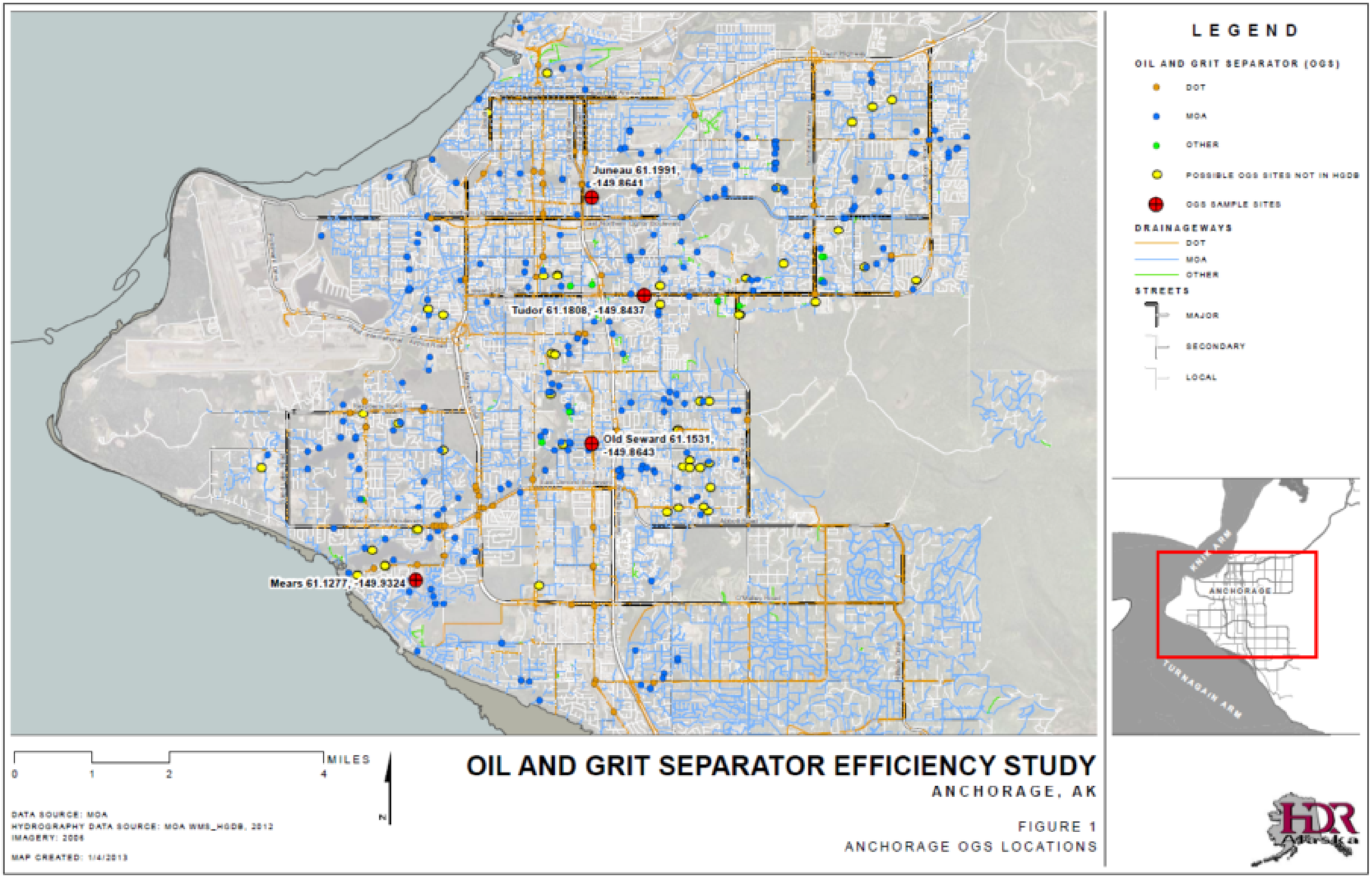
D. PROJECT DATA RECORDS

Project data records including the QAPP, original field data, lab records, chains of custody, and data files can be found under Appendix D and F on the CD entitled, "MOA WMS Sedimentation Basin and OGS Efficiency Study". This CD if not attached to the report can be obtained from the MOA WMS office in Anchorage, Alaska. The data structure is as follows:

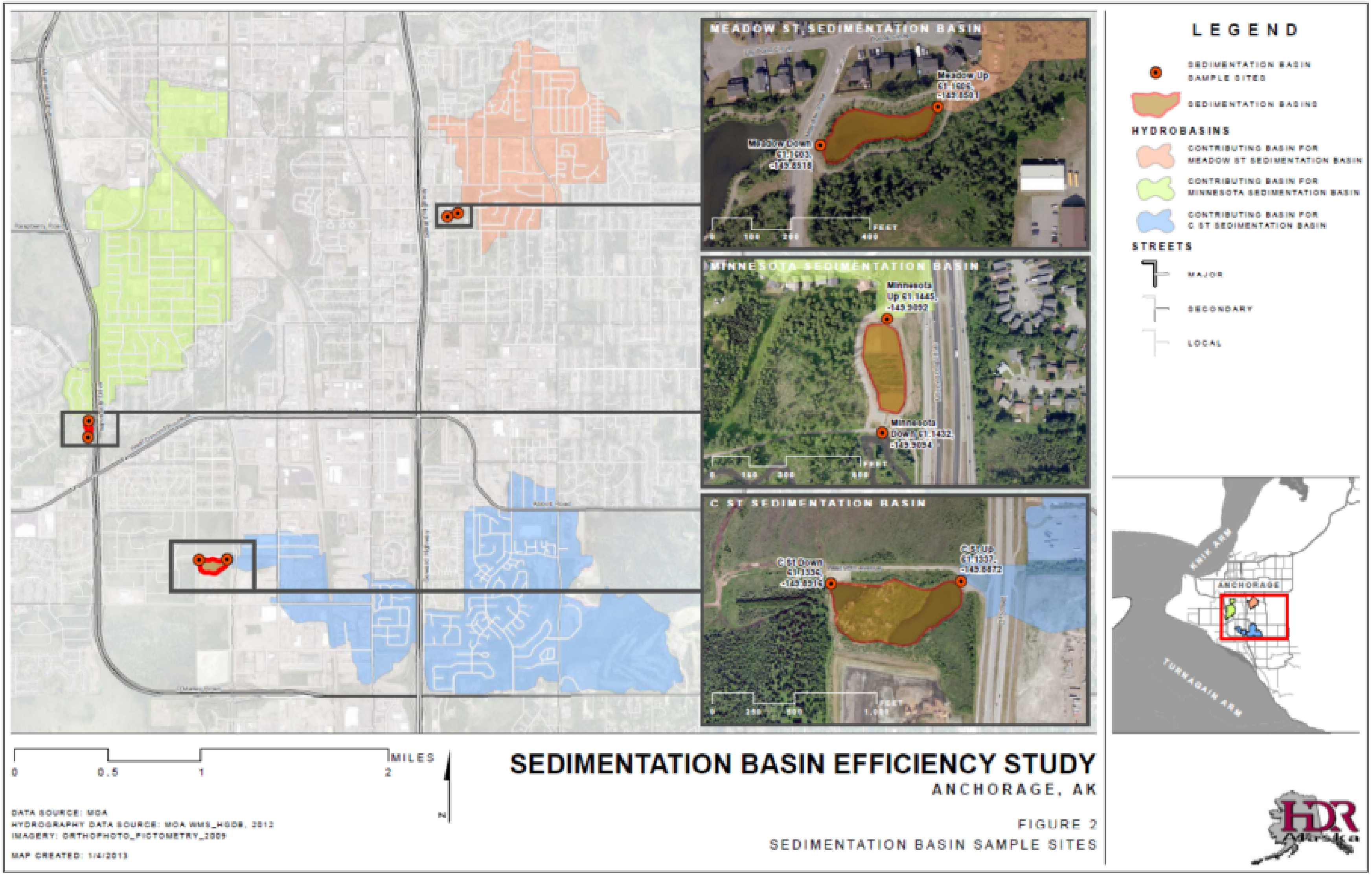
- D1. QAPP-The guiding document setting sampling and analysis protocols for this study.
- D2. Original Field and Lab Records- can be found in Appendix F.
- D3. Compilation of NWS climate record files, raw and SWMM-ready-can be found in Appendix F.
- D4. Compilation of all raw and validated data records and results-can be found in Appendix F.

E. PROJECT STATIONS AND EQUIPMENT INVENTORY

E.1 OGS Inventory and Locations Map



E.2 Project Station Maps and Location Descriptions (include GPS location data)



E.3 Project Stations Setup Instructions and Descriptions

This appendix addresses the sedimentation basin monitoring sites with a description of the instrumentation and set up for each station. The influent and effluent of 3 sedimentation basins for a total of 6 sites were monitored during the summer of 2012 and most but not all of the gear has been removed for storage. The gear remaining in the field as of 1-1-2013 is noted below.

At all sites the YSI 600 OMS V2 sondes were mounted inside a perforated 4" PVC enclosure which was clamped into a custom fabricated steel mounting bracket. These brackets were mounted upstream of the discharge weirs.

C Street and 97th Avenue Sedimentation Basin.

The site is protected by a locked gate and a fenced enclosure but ATV's gain access to the area along a muddy track from the west.

CSTUP - Influent site:

This monitoring site is located on the downstream end of the aluminum box culvert that passes under the sedimentation basin maintenance and access road

Installation includes:

- Fiberglass waterproof enclosure. Fiberglass to allow transmission of telemetry signal.
- HOBOlinked wireless communication and data storage module with battery and solar panel.
- HOBO Barometric pressure transducer
- A tipping bucket rain gage
- Atmospheric vented stage level recorder
- YSI 600 OMS V2 sonde with turbidity, vented stage recording, conductivity and temperature sensors. Internal data logger and batteries, connection/download cable and external battery adapter plug and 6 volt gel cell battery.
- A custom fabricated aluminum v notch weir constructed in two pieces bolted with concrete anchor bolts to the concrete headwall where this culvert empties into the upper pond.
- The sonde mount was attached to the bottom of the aluminum culvert with self tapping screws. A separate mount consisting of a short perforated poly tube was attached to the leg of the YSI sonde mount to secure the HOBO telemetry stage transducer.
- A stage gage was bolted to a steel angle iron that was screwed securely at both ends to the top and bottom of the culvert.
- A 4 inch x 4 inch wooden post was secured to the concrete wall on the west side of the weir also with concrete anchor bolts.
- An 8 foot steel pipe was extended above the wooden post to hold the rain gage and solar panel
- Cables were routed through flexible conduit.
- Brackets on the back of the weir secured the cable conduit, routed to the west side of the weir where it transitioned to the wooden post to the instrumentation box.

- A short piece of 1 inch PVC pipe was secured to the conduit underwater and served as a mount for the Gore Sorbers.

As of 1-1-2013 the following gear is still in place:

- Weir
- Staff Gage
- Wooden post, mast, enclosure, data logger and telemetry transmitter, solar panel, rain gage, barometric pressure sensor

CSTDOWN - Effluent site:

This instrumentation site was mounted on the upstream entrance to the outfall piping where the effluent from the sedimentation basin entered the storm drain system. The pipe entrance was covered with a steel grate that had to be modified to allow access to this site. This access grate will need to be rebuilt once the study site is abandoned.

Installation included:

- YSI 600OMS V2 Sonde with turbidity, vented stage recording, conductivity and temperature sensors. Internal data logger and batteries, connection/download cable and external battery adapter plug and 6 volt gel cell battery.
- Custom aluminum weir cut round to fit across the opening to the 4 foot diameter effluent pipe. Weir was attached with steel I-bolts secured with self tapping screws to the culvert and extending through the weir where nuts could be tightened pulling the weir against the face of the culvert where it was sealed with a foam rubber gasket made from neoprene pipe insulation.
- Steel post and 10 inch x 10 inch weatherproof steel enclosure to contain the download cable termination and external battery. Post was secured to the steel grate structure near the top and attached to the steel culvert flair at the bottom.
- A stage gage was bolted to the steel post.
- Download cable was routed through flexible conduit to the steel enclosure and secured with cable ties, conduit brackets and hose clamps.
- Sonde mount was attached to the culvert flair pan directly above the weir with self tapping screws.
- A short piece of 1 inch PVC pipe was secured to the conduit underwater and served as a mount for the Gore Sorbers.

As of 1-1-2013 the following equipment is still in place:

- Steel enclosure on steel post.
- Staff Gage

Minnesota Sedimentation Basin.

The site is protected by a locked gate off the southbound Minnesota Drive exit ramp to Dimond Blvd.

MINNUP - Influent site:

This monitoring site is located on the downstream end of the 4 foot diameter PEP Culvert above the Sedimentation basin.

Installation included:

- YSI 600 OMS V2 Sonde with turbidity, vented stage recording, conductivity and temperature sensors. Internal data logger and batteries, connection/download cable and external battery adapter plug and 6 volt gel cell battery.
- Custom aluminum weir cut to fit across the opening to the 4 foot diameter pipe. Weir was attached with steel bolts passing through the weir and into the end of the plastic pipe where nuts could be tightened pulling the weir against the face of the culvert where it was sealed with a foam rubber gasket made from neoprene pipe insulation.
- A staff gage was bolted to the upstream side of the weir.
- 10 inch x 10 inch weatherproof steel enclosure to contain the download cable termination and external battery. Enclosure was bolted directly to the side of the plastic pipe with galvanized steel bolts.
- Sonde mount was attached to the bottom of the culvert directly above the weir with self tapping screws.
- Download cable was routed through flexible conduit to the steel enclosure and secured with conduit brackets.
- A short piece of 1 inch PVC pipe was secured to the conduit underwater and served as a mount for the Gore Sorbers.

As of 1-1-2013 the following equipment is still in place:

- Steel enclosure.

MINNDOWN - Effluent site:

This instrumentation site was mounted on the downstream end of the basin outfall piping where it passed under the Campbell Creek bike path and emptied into Campbell Creek. Installation included:

- YSI 600 OMS V2 Sonde with turbidity, vented stage recording, conductivity and temperature sensors. Internal data logger and batteries, connection/download cable and external battery adapter plug and 6 volt gel cell battery.
- Custom aluminum weir cut round to fit across the opening to the 4 foot diameter effluent pipe. Weir was attached with steel I-bolts secured with self tapping screws to the culvert and extending through the weir where nuts could be tightened pulling the weir against the face of the culvert where it was sealed with a foam rubber gasket made from neoprene pipe insulation.
- 4 inch x 4 inch wood post and 10 inch x 10 inch weatherproof steel enclosure to contain the download cable termination and external battery. Post was secured to the culvert flair structure on the east side of the culvert
- Sonde mount was attached to the inside of the culvert directly above the weir with self tapping screws.

- Download cable was routed through flexible conduit to the steel enclosure and secured with conduit brackets.
- A short piece of 1inch PVC pipe was secured to the conduit underwater and served as a mount for the Gore Sorbers.

As of 1-1-2013 the following equipment is still in place:

- Steel enclosure on wooden post.

Meadow Street Sedimentation Basin.

The site is accessible off the east side of Meadow street north of 68th Avenue. Only the east pond was instrumented.

MDWUP - Influent site:

This monitoring site is located on the downstream end of the 4 foot diameter CMP Culvert above the Sedimentation basin near the north east end of the pond.

Installation included:

- YSI 600 OMS V2 Sonde with turbidity, vented stage recording, conductivity and temperature sensors. Internal data logger and batteries, connection/download cable and external battery adapter plug and 6 volt gel cell battery.
- Custom aluminum weir cut to fit across the opening to the 4 foot diameter squashed CMP pipe. Weir was attached with steel I-bolts passing through the weir and attached to the inside of the CMP pipe where nuts could be tightened pulling the weir against the face of the culvert where it was sealed with a foam rubber gasket made from neoprene pipe insulation.
- A staff gage was bolted to the upstream side of the weir.
- 10 inch x 10 inch weatherproof steel enclosure to contain the download cable termination and external battery. Enclosure was bolted directly to the side of the CMP pipe with galvanized steel bolts.
- Sonde mount was attached to the bottom of the culvert directly above the weir with self tapping screws.
- Download cable was routed through flexible conduit to the steel enclosure and secured with conduit brackets.
- A short piece of 1 inch PVC pipe was secured to the conduit underwater and served as a mount for the Gore Sorbers.

As of 1-1-2013 the following equipment is still in place:

- Steel enclosure and wood post.

MDWDOWN - Effluent site:

This instrumentation site was mounted in the east pond directly above and slightly to the south of the permanent concrete broad crested outfall weir.

- YSI 600 OMS V2 Sonde with turbidity, vented stage recording, conductivity and temperature sensors. Internal data logger and batteries, connection/download cable and external battery adapter plug and 6 volt gel cell battery.

- 4 inch x 4 inch wood post and 10 inch x 10 inch weatherproof steel enclosure to contain the download cable termination and external battery. Post was secured to the southeast inside corner of the concrete weir.
- A staff gage was mounted on the south east corner of the concrete weir structure
- Sonde mount was cantilevered into the pond with a special steel mounting bracket. This bracket was bolted to the top of the concrete weir with concrete anchor bolts.
- Download cable was routed through flexible and rigid conduit along the front face of the weir to the steel enclosure and secured with conduit brackets.
- A short piece of 1 inch PVC pipe was secured to the conduit underwater and served as a mount for the Gore Sorbers.

As of 1-1-2013 the following equipment is still in place:

- Steel enclosure on wooden post.
- Staff Gage

E.4 Weir Formulas, with Sources Cited and Tagged to Field Equipment/Sites

E.4.1 C ST Up Weir Equation

The CSTUP weir had an 150° angle and the equation used was the Kindsvater-Carter Equation (U.S. Bureau of Reclamation 2001)

Q (cfs) = 4.24 Ce Tan (ø/2)(H +K)^5/2

Ce was determined from the following formula:

Ce =.607165052-(.000874466963)*ø +(6.10393334 x 10^-6)*ø^2

And K was derived from:

K = 0.0144902648 - (.00033955535)*ø +(3.29819003 x 10^-6)*ø^2 -(1.06215442 x 10^-8)*ø^3

Stage discharge relationships are shown in Figure E.1. See Appendix E for further analysis.

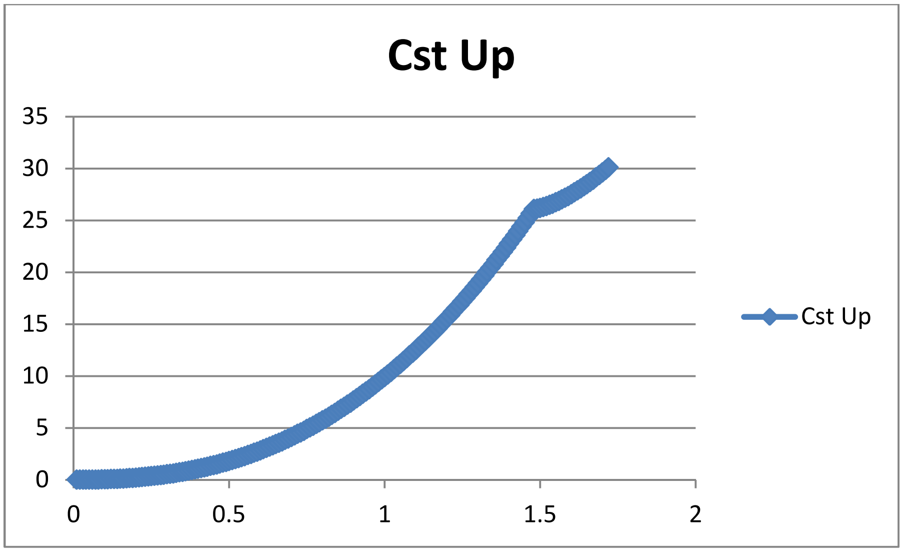


Figure E.1 CSTUP Weir Equation for Discharge Graph

E.4.2 Meadow Down Weir Equation

Meadow Down weir is a broad-crested weir that was installed during construction of the sedimentation basin and is a two stage weir. The formula is as follows:

Q = Cd *Cv *2/3*√(2/3g)*bc*h^1.5
Cv =1 for pond backed weirs
Cd = .93 + .1*h/L
Q = (.93+(.1*H/.66667))*(2/3)*((2/3*32.174)^.5)*6.17*(H^1.5)
H or h = the height above the weir of upstream pond, Hmax is 6 1/4"
L = weir crest width 8"
bc = width of weir (6.17' BELOW H= .5') (20' ABOVE H = .5')

This formula is based on the British Standard 3680, 1969. The stage discharge graph of this weir is shown in Figure E.2

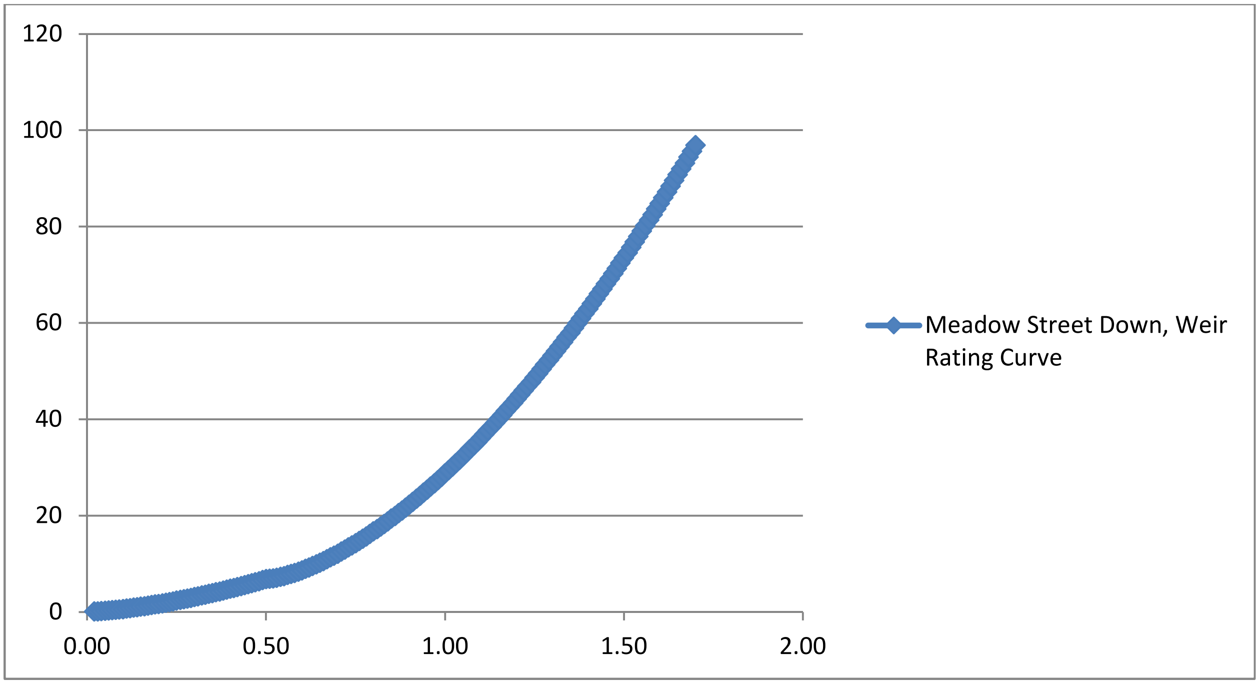


Figure E.2 Meadow Street Down, Weir Rating Curve

E.4.3 All Other Site Location Weirs

All remaining site locations were fitted with 90° V-notch weirs and discharge values can be calculated using the following formula:

$$\mathbf{Q} = 2.49H^{2.48}$$

The following graph, Figure E.3, shows the discharge rating curve for these weirs:

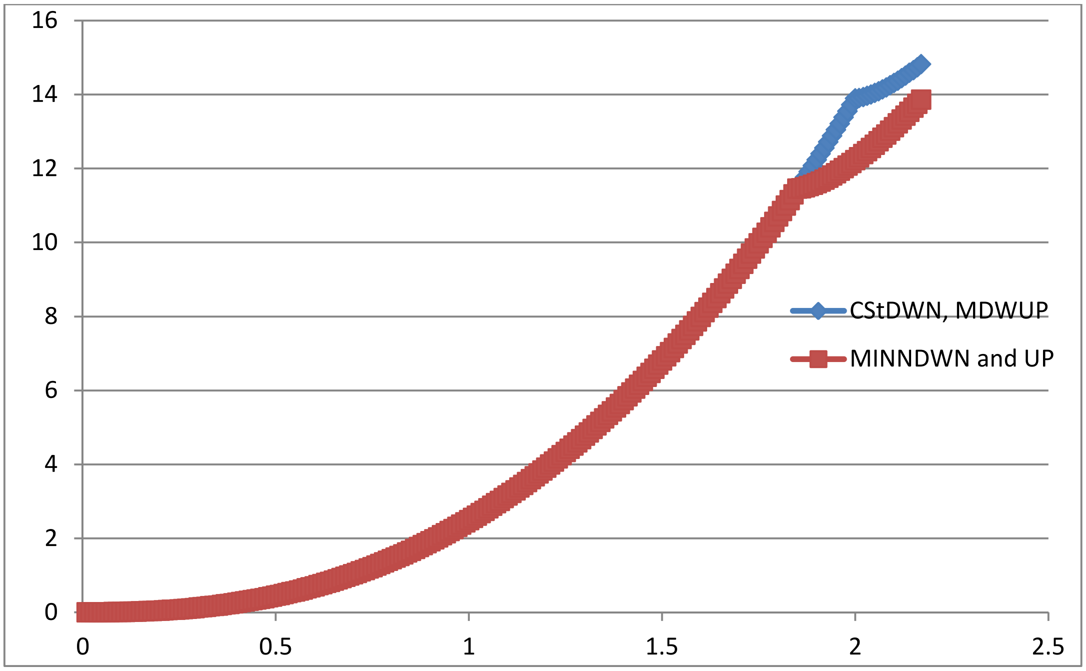


Figure E.3 Rating Curve for 90° V-notch Weirs

For further information as to how the curve was created please see Appendix E.4.

E.5 Equipment Maintenance Practices Descriptions

Equipment used for this study include six YSI Sonde 600 OMS V2 measuring water depth, temperature, conductivity, and turbidity. For in-situ field testing of pH and DO a YSI 556 multiprobe or a Hanna combo pH & EC meter.

Calibration of each instrument for specific parameters were completed according to manufacturer's instructions and with proper calibration solutions when necessary. The YSI Sondes were calibrated in office before being placed into structures at each sampling location. The calibration was for conductivity and turbidity. Conductivity was calibrated using 1413 µS/cm and for turbidity a three point calibration was conducted using 0 NTU, 126 NTU, and 1000 NTU. Calibration instructions can be found below.

Once YSI Sondes were in place at sampling locations, the water depth was set according to a staff gage present at each location that was previously set when weirs were installed. The Sondes and the housing were scrubbed clean due to iron deposits forming on the outside. This was done to prevent bias in turbidity readings. At the beginning of the sampling season, calibration checks were conducted on the Sondes to determine how often calibration would need to be conducted. Drift was not occurring therefore, calibration checks were moved to every two weeks and eventually to every three.

During calibration checks, turbidity and conductivity were checked using calibration standards and recalibrated if necessary. Calibration was recorded in calibration logs and can be found in Appendix D.

The YSI 556 multiprobe and Hanna combo probes were used for point in time data during storm events. Depending on which meter was being used, it was calibrated in the office for pH using a three point calibration and DO using the saturation method before going to the field to capture storm data. They were calibrated to the manufacturer's instructions found below.

E.5.1.1 YSI Sonde 600 OMS V2 Calibration

Carefully immerse the probes into the solution and rotate the calibration cup to engage several threads.

Connect the Sonde to the 650 MDS with the cable attached to the sonde.

From the sonde main menu select 2-Calibrate

Select the parameter to calibrate

Conductivity

Place 1413 µS/cm solution in the calibration cup to ensure the conductivity probe will be completely covered with solution.

Let set for at least one minute before calibration

Select Conductivity from the main menu of the 650 MDS

Enter 1413 into the screen asking for the calibration solution value, hit enter

The current reading of the solution will appear on screen. Record this value in the calibration log as the pre-cal reading.

Press enter. This will calibrate the sensor to the solution. Press enter again to get back to the main menu. Take a final reading before removing the solution

Turbidity (3 point calibration)

5. Ensure the turbidity sensor and wiper are clean.
6. Insert sensor with cage attached into the 0 NTU calibration solution
7. From the main menu of the 650 MDS, 2-Calibrate, and then Turbidity
8. Enter 0 in to the screen and press Enter.
9. Allow the reading to stabilize and record the value in the pre-cal reading on the calibration log. Press Enter. Allow to stabilize and record post cal reading on calibration log.
10. Remove the probe and cage from the calibration solution
11. Insert sensor and cage into 126NTU calibration solution and enter 126 into the screen as the second solution. Press Enter
12. Allow the readings to stabilize and record the pre-cal reading. Press Enter, allow to stabilize, and record the post cal reading in the calibration log.
13. Remove sensor and cage from calibration solution. Rinse with tap water and insert sensor and cage into 1000 NTU calibration solution.
14. Enter 1000 into screen and press Enter.
15. Allow to stabilize, take reading, and record in calibration log.
16. Press Enter, allow to stabilize, and recording in calibration log as post-calibration reading.
17. Press Enter, this should take you back to the main menu.

E.6 Field Equipment Inventory Including Invoices, Makes, Models, and Manuals

This appendix contains a listing of the 2012 MOA Sedimentation Basin monitoring study field equipment.

1- HOBO link weather station, data logger and wireless transmitter. Weather station contains a tipping bucket rain gage, solar panel, barometric pressure transducer, stage level sensor, and fiberglass enclosure.

- 7- YSI 600 OMS V2 sondes, All currently in working order but in need of routine maintenance. Batteries have been removed and compartments dried. Units should be returned to the manufacture for servicing prior to reuse. Each sonde comes with a padded case, maintenance kit, turbidity probe w/mechanical wiper, conductivity, temperature and vented stage level sensor and software package.
- 1- handheld YSI 650 MDS meter for direct attachment to the YSI sondes. This meter is used for downloading, field calibration and other setup functions in replacement of a computer connection.
- 7- download vented cables lengths in varying lengths.
- 6(or 5)- external battery adapter plugs for use with YSI sonde cables
- 6 or 7- 6 volt gel cell lead acid batteries for use with YSI sondes and adapter plugs
- 1or 2 - 6 volt gel cell battery chargers
- 5 -aluminum weirs for gauging flow
- 5 -10” x 10” weather proof steel enclosures
- 2 -2’ diameter x 90° V notch weirs, purchased but not used
- 1 -3’ x 90° V notch weir, purchased but not used at Juneau Street
- 6 -staff gages of various lengths for different installations.
- 7 -PVC sonde enclosures
- 6 -steel sonde mounting brackets
- Misc hardware, mounting brackets and conduit for cable routing.

E.7 Field Equipment and Instrumentation Cleaned, Labeled and in Job Boxes

This equipment and instrumentation is currently stored at the HDR remote storage unit off Arctic Blvd. It will be eventually transferred back to WMS for long term storage and/or reuse on other projects.

F. DATA REFERENCES

Data compilation and analysis completed in support of the results of this study can be found in the following file structure on the CD entitled, "MOA WMS Sedimentation Basin and OGS Efficiency Study" located at the MOA WMS office in Anchorage, Alaska:

Appendix F1 Historic Weather Data

1. *Historic Rainfall Statistics* - This excel sheet outlines the SYNOP statistical data for the historic NWS data 1962 to 2011
2. *Historic SYNOP Rainfall Precipitation Analysis*– This is the SWMM source files used for the SYNOP analysis for the historic NWS data 1962 to 2011
3. *NWS weather data Instructions* – this is the associated instruction file from NWS explaining the original DSI 3240 format the data is provided in.
4. *Original NWS hourly Precipitation data 1962-2011* – this is the original DSI 3240 file format file from NWS for precipitation data from 1962 to 2011.
5. *Historic SYNOP User Input File* – This file is the user input file format for the SWMM ready file. It converts the DSI 3240 file format into the SWMM required format.
6. *OGS 90th Percentile Historical Intensity Analysis* – a statistical analysis of hourly summer rainfall records from 1962 to 2010 to develop the appropriate OGS design intensity.

Appendix F2 Historic Hydrography Data

1. *Project Basin Landcover Development* – This file identifies the land cover areas and types for each of the contributing basins for the project sedimentation basins.

Appendix F3 Historic Sediment Loading and Washoff Data

1. *Sediment Transport for Curb and Gutter Systems* – This file analysis the washoff for a particle sizes along a 300 ft section of street.
2. *Interactions between Catchbasin and Street Cleaning in Urban Drainages and Sediment Transport in Storm Drainage Systems* – This a study critical shear velocity, shear stress and shield's parameter.

Appendix F4 Project Basin Data

1. *Field Notes for Sedimentation Basins* – This file addresses the field notes taken for each of the sedimentation basins to confirm relative geometric dimensions for project basins.
2. *Weir Equations* – This table develops the rating curves for inlet and outlet weirs for each of the project monitoring sites. This allows us to convert depth to discharge.
3. *Sedimentation Basin asbuilt Analysis Folder* – A .dwg file with associated x-referenced files used to approximate basin geometries.

Appendix F5 Project Weather Data

1. *2012 Rainfall Statistics*– this spreadsheet outlines the 2012 SYNOP statistics and storms using the rain gauge data from the project installed rain gage at the C street sedimentation basin.
2. *2012 SYNOP User Input File* – this is the SWMM formatted input file. It displays the summation of rainfall volume in inches for each hour.
3. *2012 SYNOP Rainfall Precipitation Analysis* – is the SWMM source files used for the 2012 SYNOP analysis using the 2012 UIF.txt file.

Appendix F6 Project Sampling Data

1. *Field Forms* – field notes taken during field sampling.
2. *Field Photos* – Photos taken during sampling events.
3. *Lap Reports* – Digital versions of original lab reports.
4. *Chain of Custody* – Documents indicating the chain of custody for the field samples

Appendix F7 Project QC Data

1. *Original (raw) Continuous Data* – Digital copies of the original continuous data files.
2. *Sampled and Continuous Validated Data* – worksheet identifies validated field sampled data and continuous data.

Appendix F8 Project Regression Worksheets

1. *Multiple Linear Regressions for Project Basins* – this worksheet aids in determining relationships between TSS, NTU and discharge.

Appendix F9 2012 Performance Worksheets

1. *C Street 2012 Hydrograph Storm Definition* – this worksheet developed the set of rainfall/runoff events for the C Street project basin with precipitation and hydrograph data.
2. *Meadows 2012 Hydrograph Storm Definition* - this worksheet developed the set of rainfall/runoff events for the Meadow project basin with precipitation and hydrograph data.
3. *Minnesota 2012 Hydrograph Storm Definition* - this worksheet developed the set of rainfall/runoff events for the Minnesota project basin with precipitation and hydrograph data.
4. *DCM Sedimentation Basin Performance* – This file calculated the project sedimentation basin removals using criteria defined the Municipality of Anchorage Design Criteria Manual.
5. *2012 Sedimentation Basin Probabilistic Performance with modified HLR and Volume Ratios*– This calculates the project sedimentation basin performance for each site using the effective surface area and effective and effective total basin volume.

6. *2012 Sedimentation Basin Probabilistic Performance* – This calculates the project sedimentation basin performance for each site using the geometric surface area and geometric total basin volume.
7. *C Street Basin Hydraulic Analysis*– this worksheet calculates the nominal detention time, mean detention time, peak detention time, N, λ and e values for the C Street project basin based on project gathered data.
8. *Meadows Basin Hydraulic Analysis* – this worksheet calculates the nominal detention time, mean detention time, peak detention time, N, λ and e values for the Meadow project basin based on project gathered data.
9. *Minnesota Basin Hydraulic Analysis* – this worksheet calculates the nominal detention time, mean detention time, peak detention time, N, λ and e values for the Minnesota project basin based on project gathered data.
10. *MOA Treatment Train Analysis* – This worksheet analyzes, in general, the removal rate of individual components owned and operated by MOA including: catchbasins, OGS and sedimentation basins.
11. *Meadows Spring Removal Efficiency* – This worksheet calculates the removal efficiency during spring for the Meadows project basin.
12. *Meadows Summer Removal Efficiency*– This worksheet calculates the removal efficiency during summer for the Meadows project basin.
13. *Minnesota Spring Removal Efficiency*– This worksheet calculates the removal efficiency during spring for the Minnesota project basin.
14. *Minnesota Summer Removal Efficiency*– This worksheet calculates the removal efficiency during summer for the Minnesota project basin.
15. *C Street Spring Removal Efficiency*– This worksheet calculates the removal efficiency during spring for the C Street project basin.
16. *C Street Summer Removal Efficiency*– This worksheet calculates the removal efficiency during summer for the C street project basin.

Appendix F10 Mean Performance Worksheets

1. *Historic Sedimentation Basin Probabilistic Performance* – this worksheets calculates the project basins performance under mean historical precipitation conditions using the effective surface area and effective total basin volume.

Appendix F11 Recommended Design Method Worksheets

1. *Basin N versus Delta Performance Figure*– this image depicts the basin geometry and associated N-value based on Persson et. al.
2. *Design Recommendations for Sedimentation Basins* – this worksheet roughly analysis hypothetical design recommendation scenarios based on the current C street geometric dimensions.
3. *NJCAT Testing Protocol* – this document refers to the national standards for OGS testing.

4. *Current MOA 2 yr 6 hr storm 20micron removal OGS* – this worksheet looks at the removal efficiency of a Stormceptor® at different flow rates regarding the 20 micron particle size.

G. PROJECT TEAM

2012 Sedimentation Basin and OGS Evaluation at Anchorage, Alaska

Watershed Management Services, Municipality of Anchorage

Document ID: WMP APr13001

Table G.1: Project Team and Responsibilities

Name	Project Role	Primary Responsibilities
WMS, MOA		
Scott R Wheaton	Project Scientist	Project management, project design, performance analysis design, systems analysis and systems performance analysis, off-hours sampling
HDR Alaska		
Bill Spencer, PE	Project Engineer	Project management, project design guidance development, OGS performance analysis, field logistics management, off-hours sampling
Cynthia Milligan	Lead, Water Quality Analysis Statistician	Sampling coordinator, QC and data validation, water quality analysis, regression analysis, project data management and archive
Jacques Annandale, EIT	Lead, Design Performance Analysis	Storm analysis, pollutant transport analysis, sedimentation basin performance analysis, design guidance development
Mark Doner	Data Analyst	Data analysis
Jodie Anderson	Data Analyst	Data analysis
Nick Schlosstein	Environmental Scientist	Instrumentation and field sampling
Dan Campbell, EIT	Environmental Scientist	Instrumentation and field sampling, data analysis
Alena Gerlek	Environmental Scientist	Instrumentation and field sampling
Lynn Spencer	Environmental Scientist	Field sampling
Patrick Blair	Water Resources Scientist	Station installation and maintenance
Todd Heyworth	Engineering Technician	Station installation
Erin Begier	Technical Editor	Report production