Hydraulic and Hydrologic Analyses of the May Creek Channel Restoration Project

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King County

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EXECUTIVE SUMMARY

May Creek in May Valley routinely flows out of channel inundating adjacent pastures and wetlands during the wet season. At the downstream end of the valley, the natural landscape constrains stream flows back into channel controlling flow rates leaving the valley before entering the ravine. This feature, coupled with the flat pasture lands, are reasons why flooded areas in the valley can take several days to sufficiently drain; returning to usable pasture lands. Combine this with a frequent occurrence of small storms and portions of the pasture lands expectedly remain unusable for much of the wet season with frequent undesirable inundation continuing through spring and into the summer months. Given these conditions, the proposed project focuses on areas upstream leaving the natural constricting features unaltered.

A study was conducted for May Creek in May Valley to evaluate stream channel capacity for existing and proposed conditions. This report contains hydrologic and hydraulic analyses (H&H) used to help optimize project design. Analyses include: assessment of channel capacity for existing and proposed designs, assessment of flood frequencies and durations, assessment of channel competency to mobilize fine sediments and improve lifespan of the project, and assessment of impacts to erosivity in the ravine downstream of 148th Ave SE. Two types of models were used, HSPF and HEC-RAS. HSPF is an U.S. EPA hydrologic watershed model used extensively in the Puget Sound region. The original model used was developed for the May Creek Current and Future Conditions report (King County 1995). This model was updated with more current meteorology and channel routing to improve understanding of stream responses with longer periods of precipitation record and more accurate hydraulics. The HEC-RAS model used was last modified by Otak in 2006. For this study, new land survey work was completed in 2010 to update channel geometry and enhance resolution specific to this project study area, supporting simulation of several proposed channel restoration activities to reduce frequency of flooding without significant downstream impacts.

At the lowest point in channel capacity under existing conditions, it is estimated that May creek begins to flow overbank at approximately 6 cubic feet per second (cfs). This flow rate is below the estimated mean annual flow rate of 9 cfs. Thus for most of the wet months, small portions of the pasture susceptible to these minor exceedences will remain inundated. Additionally, a tributary (Long Marsh Creek) historically deposits large gravels from a mostly forested subbasin in May Creek just upstream of a footbridge (approximately at river mile 4.6). These gravel deposits are large enough to backwater May Creek upstream for a couple thousand feet. This backwater condition facilitates the recruitment of more fines and vegetation litter that decays into organic muck. This muck then allows for an increase of vegetation encroachment of the channel further reducing channel capacity.

The project study proposes an excavation of the channel between 148th Ave SE and approximately 2000 feet upstream to an elevation of 308 ft (NAVD 88). Additionally, dense vegetation choke points downstream of 148th Ave SE will be thinned to reduce impediment of low flows exiting the valley heading to the ravine. Hydraulic analyses estimate that post project channel capacity will be increased from 6 cfs to approximately 50 cfs before overbank flows begin. This improvement will effectively reduce most small storms from flooding the pasture areas. However, this channel improvement is still below the magnitude of an annual storm, thus May Valley is still expected to flood annually, but with shorter duration. This change in low flow hydraulics frees up overbank storage for larger storms, such that results of this study estimate that storm events greater than the 10-year will either remain the same or marginally reduce in peaks. Storm events between the 1-year (approximately 10% increase) and the 10-year (approximately 1% increase) slightly increase, with all estimated increases or decreases calculated within model accuracy of a calibrated hydrologic model.

Durations of flows near the 2-year (i.e. 200 cfs) and above are essentially the same. Durations of flows at 100 cfs again are nearly the same with an estimated difference in durations of approximately 400 hours over a 60 year period (525,960 hours, 0.08%). The higher the flow rates the less difference in durations to be expected.

Sediment mobility was also evaluated to estimate expected lifespan of the project. Channel bottom sediments in the project area are comprised mostly of silty fines and organic muck. With this type of channel bottom, it's estimated that a shear stress of 0.01 pounds per square foot is required to move sediment downstream. A mean annual flow rate (i.e. 9 cfs) was selected to evaluate success of the project given that flows at or above the mean annual level occur during most of the year, thus minimizing the possibility of any significant recruitment of fines or vegetation re-establishing in the channel. Post project, estimates of shear stress at 9 cfs are at or above 0.01 psf except downstream of 148th Ave SE in the wetland. There at low flows, deposition is expected to occur similar to existing conditions. These results signify that given the management of gravel deposition from Long Marsh Creek and ability to mobilize fines in May Creek, post project conditions should continue into the future with minimal deposition of fines reducing intended channel capacity.

Based on the sediment transport study conducted downstream in the ravine (King County 2009), channel sediment mobilizes approximately at 233 cfs (refined from original flow rate estimates of 275 cfs). This estimate along with marginal changes in durations of flows (maximum difference at 100 cfs with 0.08%), suggest no significant downstream impacts in the ravine resulting from proposed project designs. It is acknowledged that there is a level of uncertainty in estimates of stream channel sediment mobilization thresholds that could be lower. Given the maximum estimated difference in durations (at 100 cfs) is approximated to an annual average increase of 7 hours during the course of a year; these effects would likely be undetectable in the ravine.

1.0. INTRODUCTION

May Creek valley experiences out of bank flooding on a routine basis every wet season lasting from several days to weeks at a time. The stream course is essentially in a bowl for approximately 2100 feet (river mile 4.6 to 5.0) between a footbridge upstream of Parcel #0223059005 property down to Long Marsh confluence—just upstream of another footbridge. Long Marsh is primarily a forested basin with steep gradients. This characteristic gives the tributary the ability to deposit gravels large enough that May Creek is not capable to redistribute gravels downstream. Thus stream bed elevations at this location rise as more gravels are deposited. This accumulation then backwaters May Creek upstream causing more deposition of fines and decaying vegetation—ultimately reducing conveyance capacity and increasing frequency of valley flooding.

Like any natural stream system, larger but less frequent flow rates perform work on the stream banks and bed. Downstream of the valley, May Creek drops into a ravine where channel forming processes are expected. A recent sediment transport study was conducted at three locations in the ravine between Coal Creek Parkway and 148th Ave SE (King County 2009) characterizing conditions capable of causing erosion in the ravine. Those results are used to evaluate effects of this proposed study.

1.1 Study Goals

The goal of this study is to evaluate channel capacity for different alternatives in the valley area to maintain flow rates near 1-year flood return interval by showing a reduction in frequency and duration of flooding. The reduction in duration of flooding is intended to affect only the most frequent, smaller storms, therefore unlikely to have any significant impact to the larger storms capable of eroding downstream conditions. Additionally, the proposed conveyance improvement should also be sustainable by passing through silts and retarding buildup of fines.

In order to perform these types of analyses, a combination of techniques was necessary to evaluate detailed hydraulics and hydrology. Two types of models were used to perform the analyses, HEC-RAS for hydraulics and HSPF (Bicknell 2005) for hydrology. Both models used were adapted from existing models and updated to reflect current conditions. HEC-RAS (USACE 2005) was used to evaluate channel conveyance capacities and flooding inundations, while HSPF was used to provide statistical measures of durations and magnitudes of storm events.

1.2 Study Extent

While the extent of the proposed channel improvements extend from river mile 4.31 up to river mile 4.99 (yellow highlight in Figure 1), it was necessary to extend the boundary conditions to support the ravine erosion analysis and include the lower portions of the HEC-RAS hydraulic model down to the Coal Creek Parkway, river mile 3.59 (model extent shown as cross-sections in green in Figure 1). Similarly, the watershed model used encompasses the entire basin as shown in light red in Figure 1.

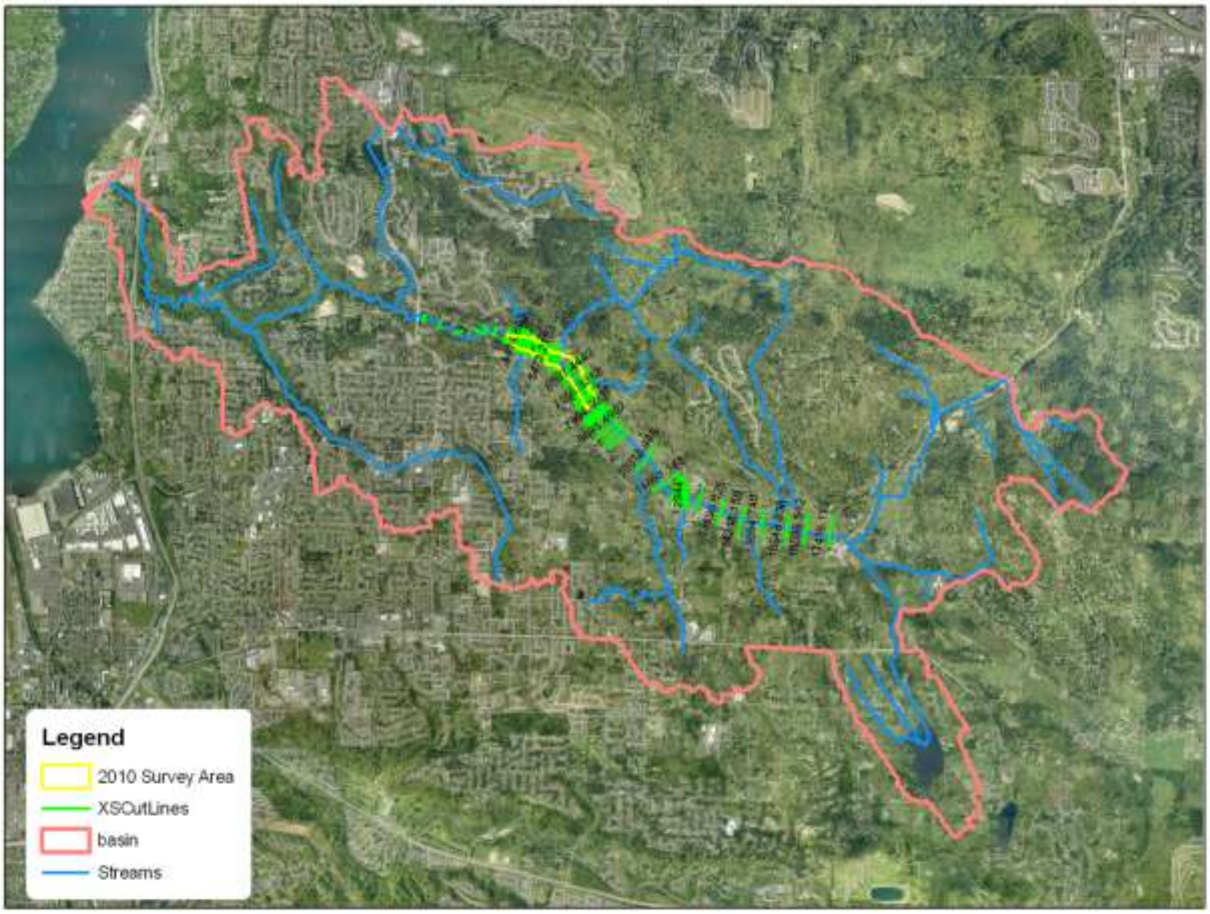

Figure 1 Locator map of study area in May Creek basin.

2.0. MODEL SYSTEM DESIGN

Model design was dependant on available information and intended goals of the project. Modeling of the May Creek system included a hydrologic (HSPF) and hydraulic model (HEC-RAS). The hydrologic model used was developed for the *May Creek Current and Future Conditions* report (King County 1995), while the HEC-RAS model was based on multiple modifications over time (King County 1995, Entranco 2002, Otak 2006) as projects occurred, with the most recent modifications performed by Otak in 2006.

2.1 Objectives

The model system setup was designed to address May Creek Capital Improvement Project restoring hydraulic capacity with these objectives:

1. assess channel capacity for existing and proposed designs,
2. assess changes in flood frequencies and durations,
3. assess channel competency to mobilize fines in May Creek through the valley, and
4. assess impacts to erosivity in the ravine downstream of 148^{th} Ave SE.

2.2 System Overview

In order to evaluate the stochastic nature of stream hydrology, it was necessary to perform a deterministic evaluation of the flow rates in the ravine and valley. Using the backwater computational abilities of HEC-RAS, channel routing tables (FTABLES) were created to provide a detailed characteristic of reaches in May Creek in HSPF. Then using HSPF, hourly continuous stream flow data are simulated through the May valley and ravine. By simulating continuous hydrologic conditions for multiple decades (i.e. 60 years), the sequencing and permutations of selecting shapes and magnitudes of storm events are not needed. This framework then allows for a comprehensive durational analysis of exceedances of flow rates that inundate the valley and exceedances of flows above the incipient motion threshold in the ravine.

2.3 HEC-RAS Model Setup

An existing model was used as a starting point for updating existing channel geometry with recent survey data collected in January 2010. Outside of the surveyed area, existing model definitions were used. Additionally, there was the intent to use the same stationing for location of cross-sections as was previously defined in the model within the surveyed area with cross-sections added where recent survey data suggested a change in topography that may not have been present in the previous modeling efforts. This included a denser set of cross-section stationing to better encapsulate undulations of the stream profile where adverse slopes between segments were common or where vegetation choke points are occurring..

The existing model domain started a short distance downstream of Coal Creek Parkway, to two-thirds of a mile upstream of where May Creek crosses May Valley Road at S.R. 900 (a little over 4 miles in total).

2.3.1 Survey Data

King County recently surveyed much of the lower valley starting at Parcel #0323059038 property (approximately 950 ft downstream of 148th), to approximately 640 ft upstream of Parcel #0223059005 lower footbridge—approximately 3800 ft of stream length, during the month of January 2010. This surveyed area coincides with river miles: 4.266 through 4.99 (Figure 2). To further extend the model cross-sections from valley wall to valley wall, ground elevations using LiDAR data were used. Given the comprehensive extent of the field survey work, the addition of LiDAR was more for visualization rather than included in any of the hydraulic computations. The one exemption in the recent survey data were any bridge geometries upstream of 148th street (including 148th Street bridge). For these structures, existing geometry in the HEC-RAS model was used (see Table 1 for longitudinal stationing).

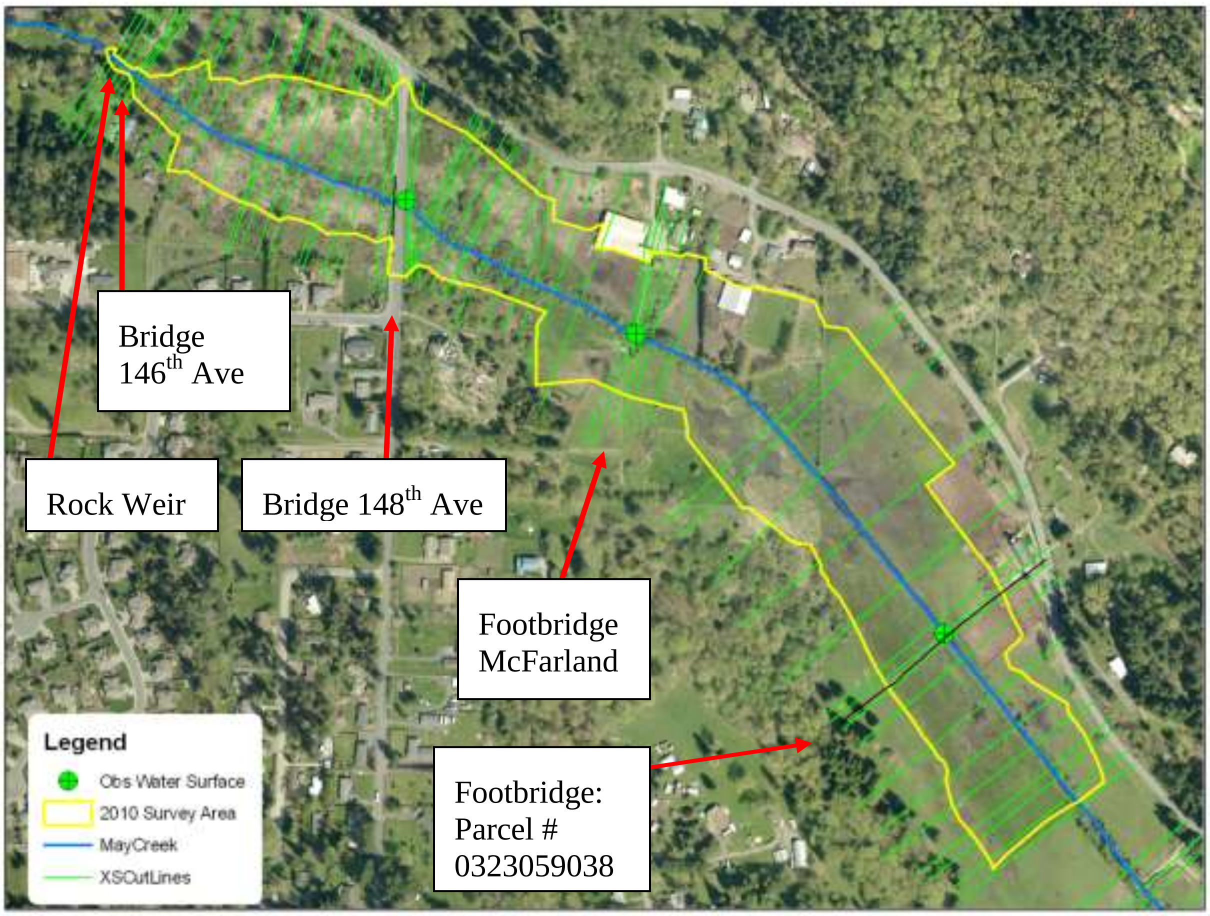

Figure 2 Extent of January 2010 King County Survey

Table 1 Stationing for structures in the HEC-RAS model

Station	Description
7.07	Bridge- May Valley Road
6.95	Bridge- Renton-Issaquah Road

5.87	Bridge- 164th Avenue SE
5.04	Footbridge- Upstream of Parcel #323059038
4.868	Footbridge- Parcel #323059038
4.612	Footbridge- McFarland's
4.455	Bridge- 148th Avenue SE
4.28	Bridge- 146th Avenue SE
4.265	Weir- Partial rock weir
4.114	Bridge- 143rd Avenue SE

2.3.2 Flow Rate Change Locations

In a stream system where tributaries occur or where attenuations from in-channel and overbank storage volumes reduce peak flow rates, a defined water surface profile may have changing flow rates associated to a set of cross-sections in the model. These flow rate changing locations in the hydraulic model were derived from two methods; either using observed gauge flows or results from continuous hydrologic model (HSPF). Statistical type flows such as mean annual, or 2 yr, etc., are based on outputs of the HSPF model, while any specific flow rate events evaluated were based on gauge data.

In the hydraulic model, there were five defined inflow points starting near the headwaters as the upstream inflow down to where two lateral tributaries drain into the wetland on Open Space 803540, west of 148th Ave SE. River mile stations for the flow change locations defined in the model are:

- River mile: 7.605 (defined as catchment outlet NFK), head waters of May Creek
- River mile: 7.05 (defined as aggregation of catchment outlets: NFK, EFK, and LKC), confluence of North Fork, East Fork, and Lake Kathleen, at SR-900
- River mile: 6.943 (defined as catchment outlet MVM), local drainages feeding to downstream of 164th Ave SE
- River mile: 5.277 (defined as catchment outlet MVL), drainages leading to 148th Ave SE
- River mile: 4.388 (defined as catchment outlet CCP), drainages leading to Coal Creek Parkway.

As an example, the mean annual water surface profile is defined using the stationing from above. Flow rates start at the headwaters with 4.6 cfs, and accumulate to 13.6 cfs entering into the ravine.

Table 2 Example of Flow Change Locations in HEC-RAS for mean annual flow rate

Station	mean annual

	(cfs)
7.605	4.6
7.05	10
6.943	8.4
5.278	8.6
4.388	13.6

2.3.3 Channel Roughness

Defined channel roughness followed previous modeling efforts developed by Otak. Essentially, channel reaches with substantial reed canary grass or collections of willow tree root systems were simulated with a channel roughness decreasing with increasing flow rates (Table 3). Otherwise, channel roughness in continuously choked reaches has a constant channel roughness of 0.07 and where channel was assumed clear, a roughness of 0.04.

In addition to channel roughness, obstructions were used represent effective blockages either from dense clusters of willow trees, or heavy mats of canary reed grass on the banks.

Table 3 Vertical varying roughness by flow rate

	Manning's n		
Flow Rate (cfs)	Left Bank	Channel	Right Bank
5	0.080	0.089	0.080
10	0.070	0.081	0.070
25	0.065	0.060	0.065
50	0.065	0.051	0.065
75	0.055	0.047	0.055
100	0.055	0.045	0.055
125	0.055	0.042	0.055
150	0.050	0.040	0.050
175	0.050	0.039	0.050
200	0.050	0.037	0.050

	Manning's n		
Flow Rate (cfs)	Left Bank	Channel	Right Bank
250	0.050	0.036	0.050
300	0.050	0.035	0.050
350	0.050	0.034	0.050

2.4 Hydrology

A numerical hydrologic model (HSPF) developed for the 1995 *May Creek Current and Future Conditions Report* was used to simulate the hydrologic regime for a 60 year period. To generate this long period of record, the National Weather Service Sea-Tac metrological station was used for precipitation, and the Washington State University Puyallup station was used for evapotranspiration (ET). The period of record simulated was from water year 1949 through water year 2008 (10/1/1948 – 9/30/2008).

2.4.1 Precipitation

Precipitation is processed to hourly intervals, while the ET is processed to daily increments. However in the lower Puget sound basin, the Cascade foothills topography create an orographic effect of increasing precipitation the further east and closer to the mountain range. As a result, the observations made at Sea-Tac station needs to be translated to the May Valley basin. There are any number of ways of doing this, one typical way is to scale precipitation using mean annual ratios of Sea-Tac to any local data in the basin. In general, this will provide a means for representing annual runoff volumes, but the scalar can be greatly divergent for a given season (e.g. over estimate storms in the winter and under estimate in the summer). This technique was used in the original Conditions report model. However, for this project a slightly more sophisticated technique was used to better preserve the individual seasons (i.e. by month). In May Valley, there were two local precipitation stations used to scale the Sea-Tac data (King County station 37u for the lower parts of the valley, and 37v for the upper elevations of the valley. The Sea-Tac data were then scaled on a monthly basis using linear regressions with a constant of 0.0 for each month, such that zero precipitation at SeaTac will be zero precipitation in May Valley. This allows for closer approximation of seasonal variability. Thus, in the HSPF model where one would use a scalar to adjust the Sea-Tac precipitation, the scalar is kept at 1.0 since the scaling was done prior to the model run. Monthly Scalars are listed in the table below.

Table 4 Monthly scalars to transpose SeaTac precipitation to May Valley.

Month	SeaTac to 37V	SeaTac to 37U
January	1.172	1.044
February	1.150	1.096

Month	SeaTac to 37V	SeaTac to 37U
March	1.252	1.197
April	1.270	1.150
May	1.555	1.378
June	1.311	1.283
July	1.785	1.438
August	1.187	1.163
September	1.462	1.272
October	1.352	1.158
November	1.178	1.069
December	1.112	1.064

2.4.2 FTABLES

FTABLES are user defined channel routing tables characterizing the relationship between stage, surface area, storage volumes, and flow rates. Four of these FTABLEs were modified to reflect the hydraulics modeled using HEC-RAS. Using the multiple flow rate profiles defined in HEC-RAS ranging from mean annual flow rates to 100-year flood frequencies, a series of cross-sections were used to define the transient storage HSPF utilizes for kinematic wave routing. For every cross-section in HEC-RAS the stage and wetted area can be highly distinct, thus an average was developed for each of the four catchments in HSPF. The groupings of cross-sections per catchment are listed below:

- RM 3.5 though 4.451 were used for HSPF catchment CCP- FTABLE 100
- RM 4.53 through 5.49 were used for HSPF catchment MVL- FTALBE 80
- RM 5.69 through 5.86 were used for HSPF catchment MVM- FTABLE 70
- RM 5.87 through 6.84 were used for HSPF catchment CFD- FTABLE 60.

The depth and flow rates are weighted averages using the downstream channel length defined in HEC-RAS. Surface areas and storage volumes are summed up for each group of cross-sections defined above. While the overall differences are minor, this was performed for each geometric scenario and inserted into the HSPF scenarios for durational analyses.

2.4.3 Refined Estimate of Stream Flow Events for Phase II Sediment Study

During the sediment study time period, there were two significant storm events that occurred, December 2007 and January 2009. Recorded stream flows during those events were determined to be unreliable (see Phase II, May Creek Sediment Transport Study). Additional investigation into estimating the magnitude of those two events was instructional to better understanding the sediment mobilization that occurred during the two events.

Flows at stations: 37A, 37B, May2, and 37G were evaluated for the two defining storm events (December 2007, January 2009) used to estimate incipient motion, 37H was not installed until WY 2010. At stations May1 and May3 no flow estimates were done, only stage was recorded (see Figure 3 for locations). Because it has been reported of active erosion/deposition influencing water levels at 37B, during the January 2009 event there is more uncertainty for flow estimates. Additionally, using a scaling method to synthesize records at 37B, Anchor estimated the peak flow rates for December 2007 and January 2009 to be very similar (339 and 348 cfs, respectively). However, upon further investigation the estimate used for the December 2007 event was based on a peak at 37A not appropriate for transposition to monitoring station 37B.

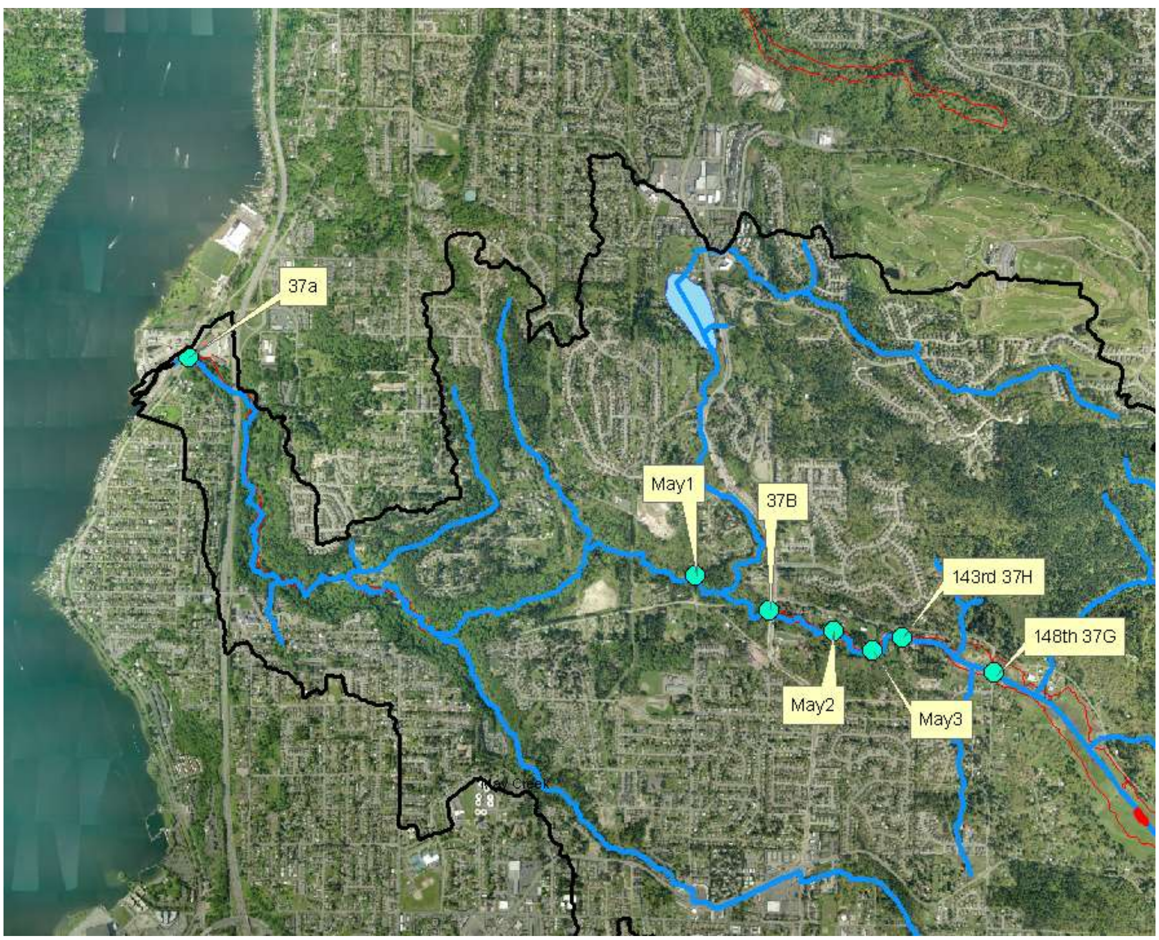

Figure 3 Gauge monitoring locations

The peak flow for the December 2007 event at 37A was near 600-cfs; however, that peak clearly occurs prior to the peaks measured upstream at the various continuous recording stations.

Moreover, there is a considerable area of highly developed landscape draining to 37A below 37B. This drainage area with relatively high amounts of impervious would respond to an event significantly faster than the upper May Valley basin. This is illustrated by overlaying the various hydrographs for the December event (Figure 4). Thus, even though the daily peak flow rates for the various stations do occur within the same day, the peak event at 37A used should be just below 500 cfs--rather than the near 600-cfs for that event.

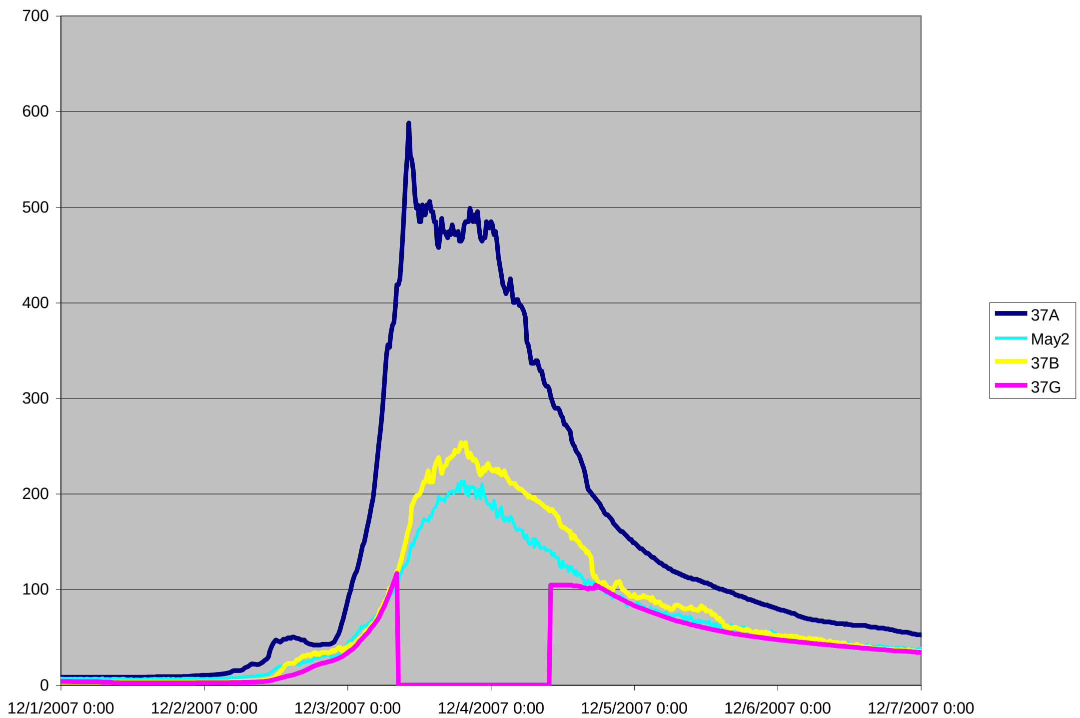

Figure 4 Flow rates for the December 2007 event

Conversely, while the fast response of the lower drainage areas presents themselves in the January 2009 event (Figure 5), that local maximum is less than the daily maximum coincident with the other stations. Thus the daily peaks used for the January 2009 event should be near 600-cfs at 37a (as was previously used). While the citation for the basis of the censoring the multiple years of continuous stream gauge records for 37a and 37b is provided, two apparent actions were taken in pre-processing the data: 1) data were split into high flow and low flow events, and 2) some periods of record were filtered for use. Given the survey results at monitoring station May2, there were minor changes in channel geometry between the beginning and the end of the sediment study. Therefore, the continuous water level measured at that station was assumed to remain consistent throughout the sediment study period with possibly small adjustments to the associated flow rates. Thus all else being equal, a greater depth at that location would coincide with greater flows—assuming no downstream conditions influence the gauge. Reviewing the stages at May2 (Figure 6 and Figure 7) for those two events, January 2009 was observed to have a stage approximately 0.5-ft higher than the December 2007 event. Thus, it is assumed the flows during the January event were greater than the 2007 event.

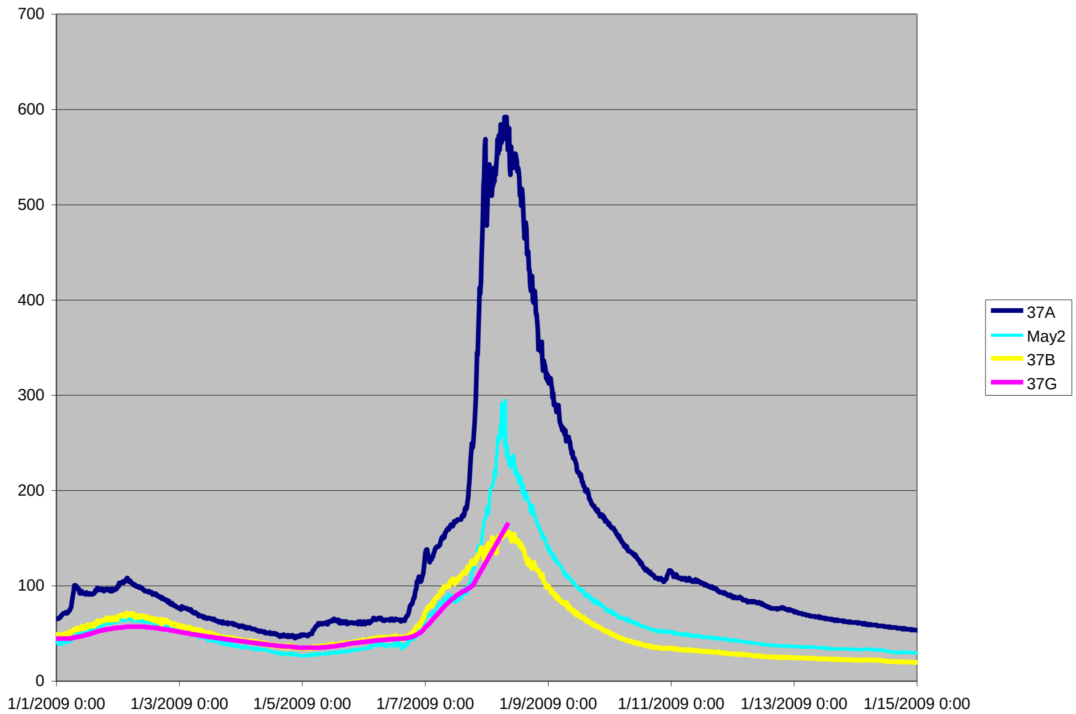

Figure 5 Flow rates for the January 2009 event

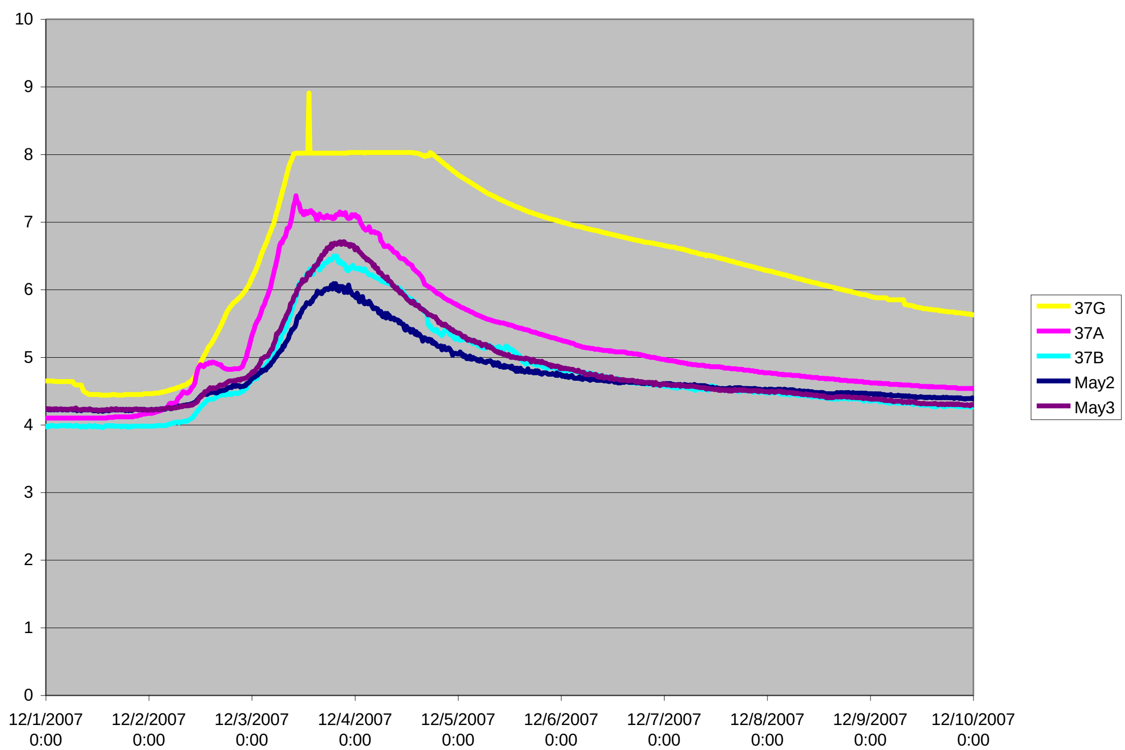

Figure 6 Stages for December 2007 event. Note the obvious data errors in station 37G.

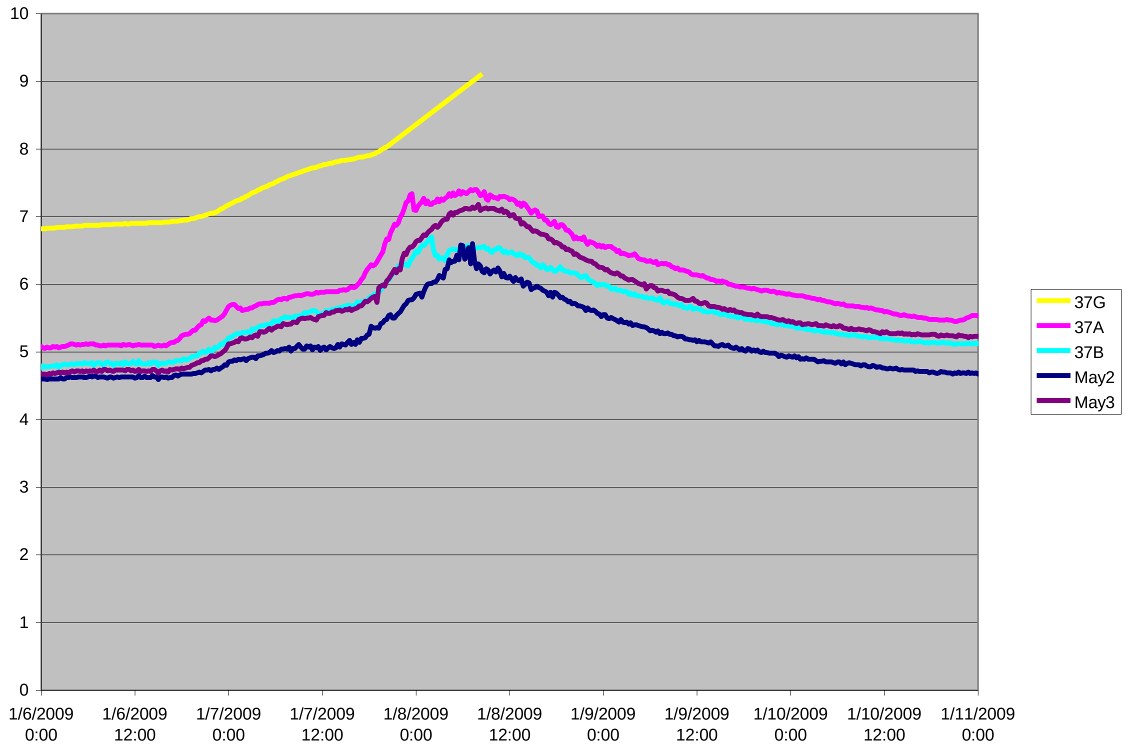

Figure 7 Stages for January 2009 event. Note the missing data for 37G.

In lieu of investigating Anchor's approach on using a subset of data and estimate an independent peak flow for 37b, a moderately different method was performed but for preliminary review adequate for comparison to Anchor's approach. Peak daily flow rates (based on mean-hourly) were assembled from November 1, 1998 through February 5, 2009 to be similar to Anchor absolute start and end dates—Ten plus years of data were used, and any days with missing data in either gauge were disregarded for this analysis. Then a robust regression (LOWESS) was performed on the entire set of peak daily data (subsequently, the same date ranges were used as in Anchor—the LOWESS results remained the same). In short, the LOWESS regression provides a sophisticated method for performing regressions applicable for linear and non-linear data making it not necessary to separate high and low storm events (Helsel and Hirsch, 2002). In addition to estimating the most likely value for 37B, secondary regressions were performed on the positive and negative residuals of the primary regression to estimate a range of possible values based on the primary regression (Figure 8).

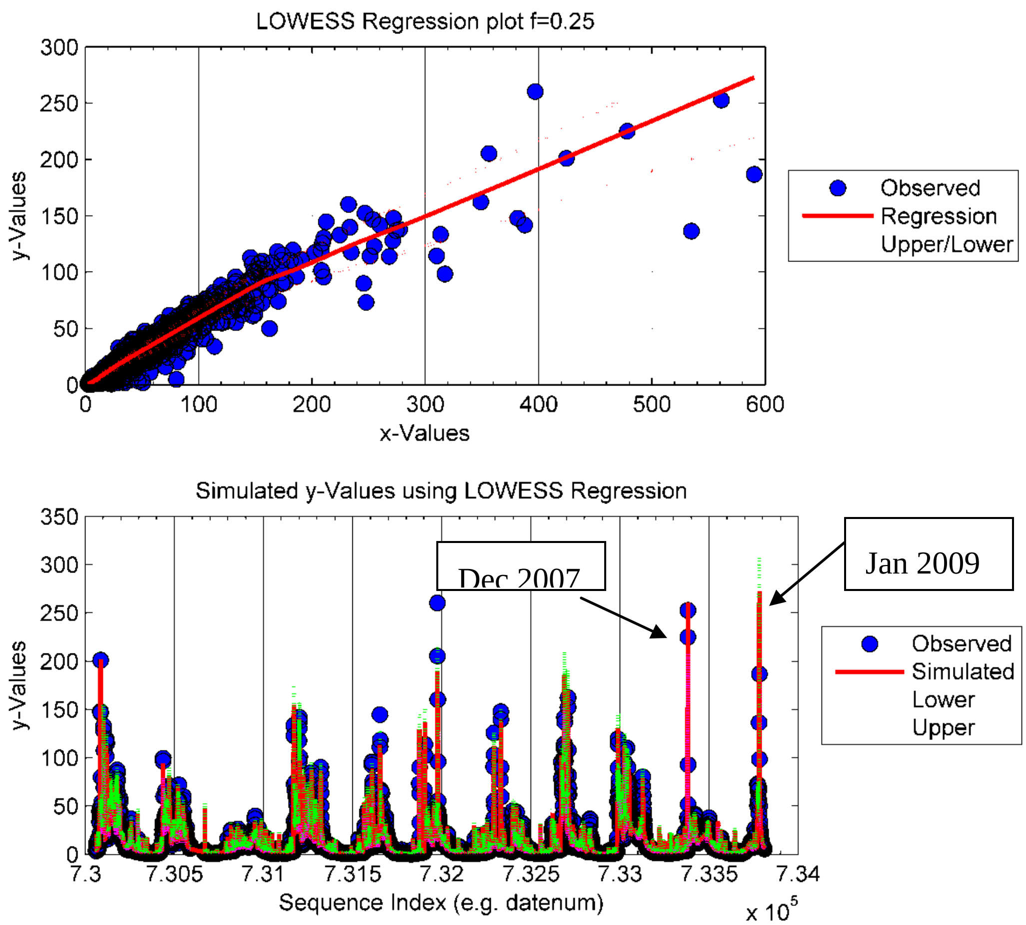

Figure 8 Robust Regression (LOWESS- Locally weighted scatter plot smoothing) on Daily Peak Flow Rates from 11/1/1998 through 2/5/2009. X-axis = 37A, Y-axis = 37B. Lower graph is a simulation of 37B using the regression and observed with time on the x-axis, and flow rate on the y-axis.

Results from the regressions estimates peak daily flows at 37B to be **233-cfs** (with a possible range of 198 cfs to 260 cfs) for December 2007 (Figure 9) and **272-cfs** (with a possible range of 218 cfs to 310 cfs) for January 2009 (Figure 10). Notably, this revised estimate more closely matches the critical shear stress presented in the sediment transport study based on observed data. Additionally, performing the same analysis but not including data after January 13, 2006, the flow estimates at 37B are estimated to be 240 cfs and 280 cfs (rounding to the nearest 10 cfs) for the 2007 and 2009 events. These are based on linear extrapolation of the LOWESS regression results since the magnitude of the peaks for the 2007 and 2009 events did not exist in the reduced dataset.

Comparing the simulated flow rates using the robust regression to observed at gauging station 37B, there are good correlations, r-square's, and slope's when using all data greater than 100 cfs and for data greater than 100 cfs but excluding gauge records after January 13, 2006 (Table 5). A perfect fit would have a coefficient of 1.0 for each of those statistics and an intercept of 0.0. As such, the robust regression slightly under predicts observed.

Table 5 Accuracy of robust regression for simulated versus observed for gauging station 37B using linear regression statistics, with observed on the x-axis for slope.

Dataset	Pearson	r-square	Slope	Intercept (cfs)
All Data greater than 100 cfs	0.87	0.75	0.91	10.1
Greater than 100 cfs and excludes data after 1/13/2006	0.88	0.78	0.82	14.8

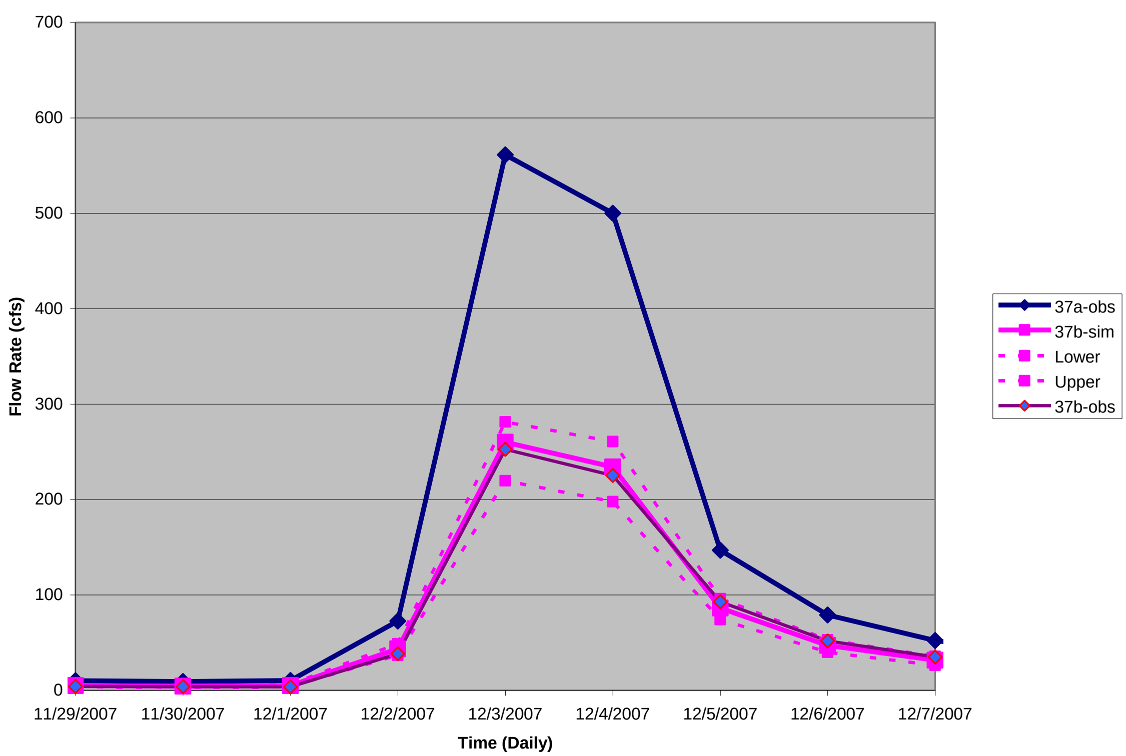

Figure 9 December 2007 Hydrograph of event

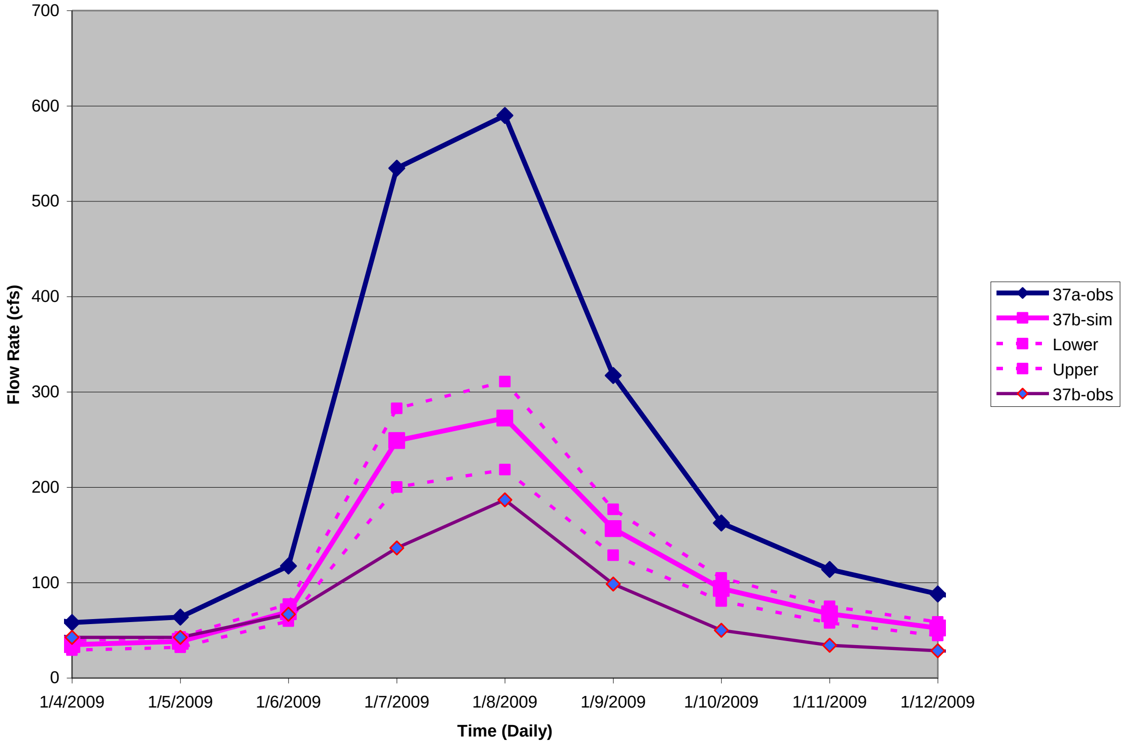

Figure 10 January 2009 Hydrograph of event

It is worth noting that although it was discussed previously about the faster response for drainages below 37B, no effort was made on a storm event basis to match up peaks that might have shifts within the same day; which is similar to the method previously done by the sediment study.

One other comparison was performed using the HEC-RAS model developed for this project. At the May2 station, it was noted in the May Creek Sediment Transport Study, that the model was estimating approximately 1-ft higher than observed after calibration. This was based on the flow rate of 340+ cfs for those two storm events. An attempt was made to reconcile where the monitoring stations were in the HEC-RAS model and known stations and locations in the ravine. Stationing between the HEC-RAS model and assumed known locations of the gauging did not reconcile; thus, matching up where the documented elevations are at the study sites and cross-sections in the model was not possible at this time.

2.4.4 Model Validation

Edge of water was surveyed on two different days, January 8, 2010 and January 22, 2010. The January 8 survey was preceded by a small storm on January 5 cresting at 42 cfs as estimated at 148th Ave SE bridge (KC Gauge 37G). Flows computed from the gauge during the survey on January 8 were approximately 29 cfs. The second survey occurred after a larger storm that began to recede January 16. The peak flow rate using the same stream flow gauge for that event was estimated to be 66 cfs. During the January 22 survey of edge of water, flows were estimated to have receded to a flow rate of 13 cfs as measured at KC gauge 37G (see Figure 12).

Survey data for the January 8 event were transposed to the nearest cross-sections (RM 4.8612 and 4.8285) just downstream of Parcel #0223059005 footbridge and the January 22 survey work was taken upstream of the Parcel #0223059005 footbridge (see Figure 11 and Table 6 below for more detail).

Figure 11 Edge of water survey shown in light blue lines.

Table 6 Water surface observations, elevations in NAVD88.

Station	Date	Left Bank	Right Bank	Avg.	37G Flow Rate	WS Profiles	Diff.
4.992552	1/22	314.27	313.85	314.06	13 cfs	313.00	-1.06
4.988154	1/22		314.20	314.20	13 cfs	312.99	-1.21
4.9749	1/22	314.35	314.04	314.20	13 cfs	312.98	-1.22
4.949	1/22	314.08	314.14	314.11	13 cfs	312.94	-1.17
4.937	1/22	314.17	314.14	314.16	13 cfs	312.91	-1.25

Station	Date	Left Bank	Right Bank	Avg.	37G Flow Rate	WS Profiles	Diff.
4.912	1/22	313.97	314.19	314.08	13 cfs	312.89	-1.19
4.8612	1/8	313.95		313.95	29 cfs	313.29	-0.66
4.8285	1/8		313.41	313.41	29 cfs	313.27	-0.14
4.868	1/14			314.15[1]	64 cfs	313.97	-0.18
Station	Date				Field Measured Flow Rate	WS Profile	Diff
4.868	3/30			313.54[1]	40 cfs	313.60	+0.06
4.612	3/30			313.97[1]	48 cfs	313.47	-0.50
4.455	3/30			312.04[1]	48 cfs	311.89	-0.15

[1]Elevation is based on tape down from top of footbridge.

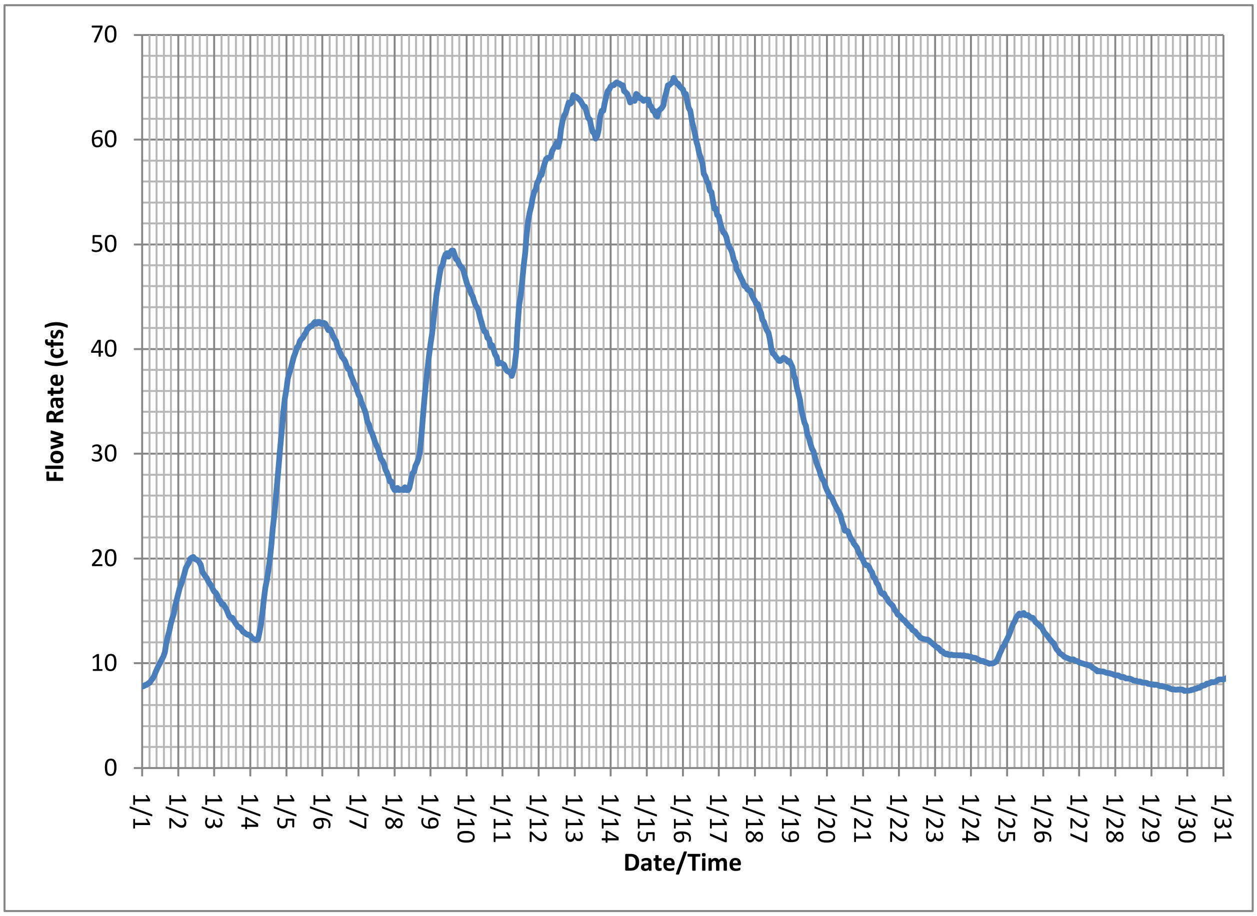

Figure 12 January 2010 Hydrograph for King County Gauge 37G (148th Bridge)

Reviewing the surveyed water surface elevations shows that for those two dates, the elevations are virtually the same with the exception of the lower most point a half foot lower (associated with the 29 cfs date) than the rest. This does present an inconsistency between the calculated flow rates and expected water surface elevations. With flow rates on January 22 less than half of what was estimated for January 8, one would expect the water surface elevations to be lower on January 22, when in fact they appear the same or higher with half the associated flow rate.

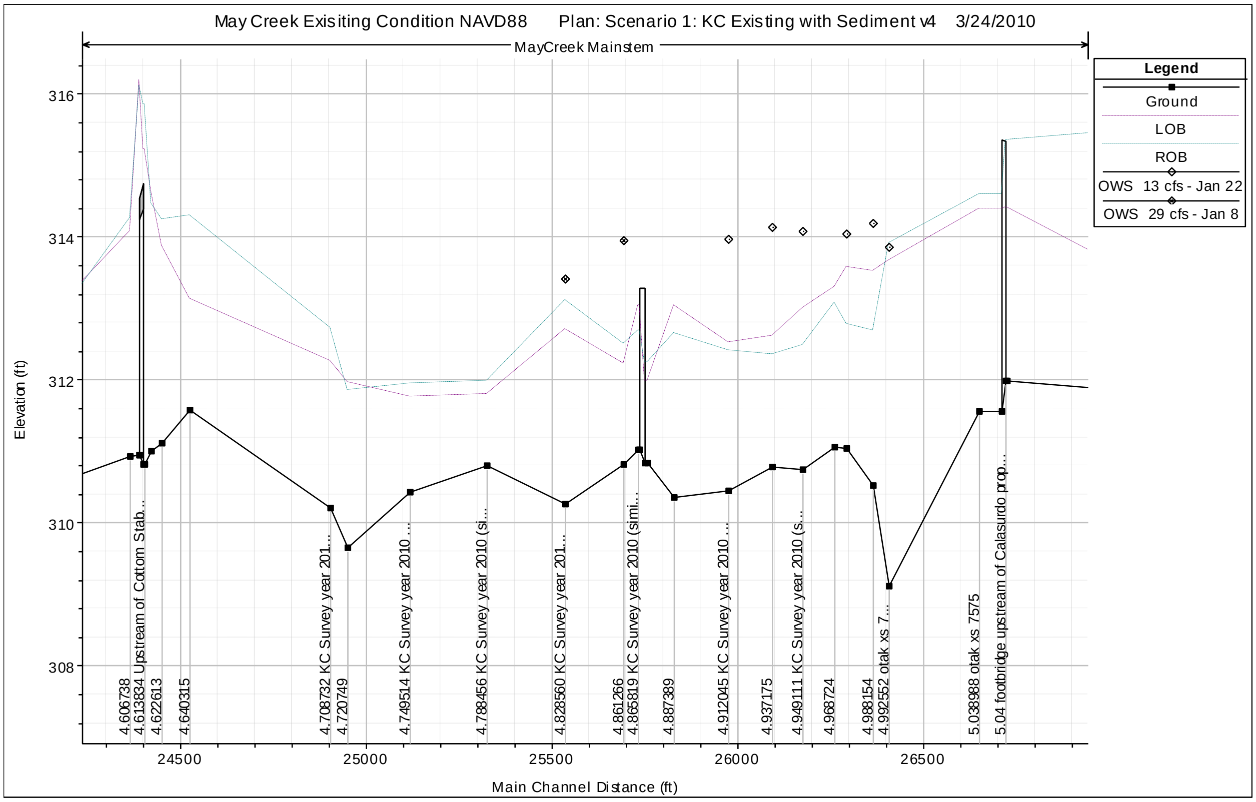

Figure 13 Profile of observed water surface elevations for January 8 (downstream of footbridge) & 22 (upstream of footbridge) with left and right bank elevations (LOB, ROB) plotted.

In addition, in the area where edge of water was taken, overbank ground elevations are lower than bank elevations effectively creating a bowl outside the channel. This is consistent for about 570 feet (RM 4.88 through 4.99). The observed edge of water was either very near bank elevations (i.e. depressional area filled with water) or up to a half foot above assumed bank elevations (see Figure 14).

Given the combined circumstances of inconsistent water surface elevations relative to flow rates, and the overbank depressional areas, conveyance out of bank in the pastures likely will behave in a couple of different fashions. When flows are initially going over bank, the flow pathways will act like a branch in the stream with its own water surface profile until it rejoins the mainstem. Then as flooding waters increase, the whole valley acts as one conveyance. As the storm recedes, the overbank flooding areas begin to behave like a slow draining lake (see Figure 15). Each of these conditions has a different hydraulic characteristic that may yield these inconsistent out of bank water surfaces for a given estimated flow rate.

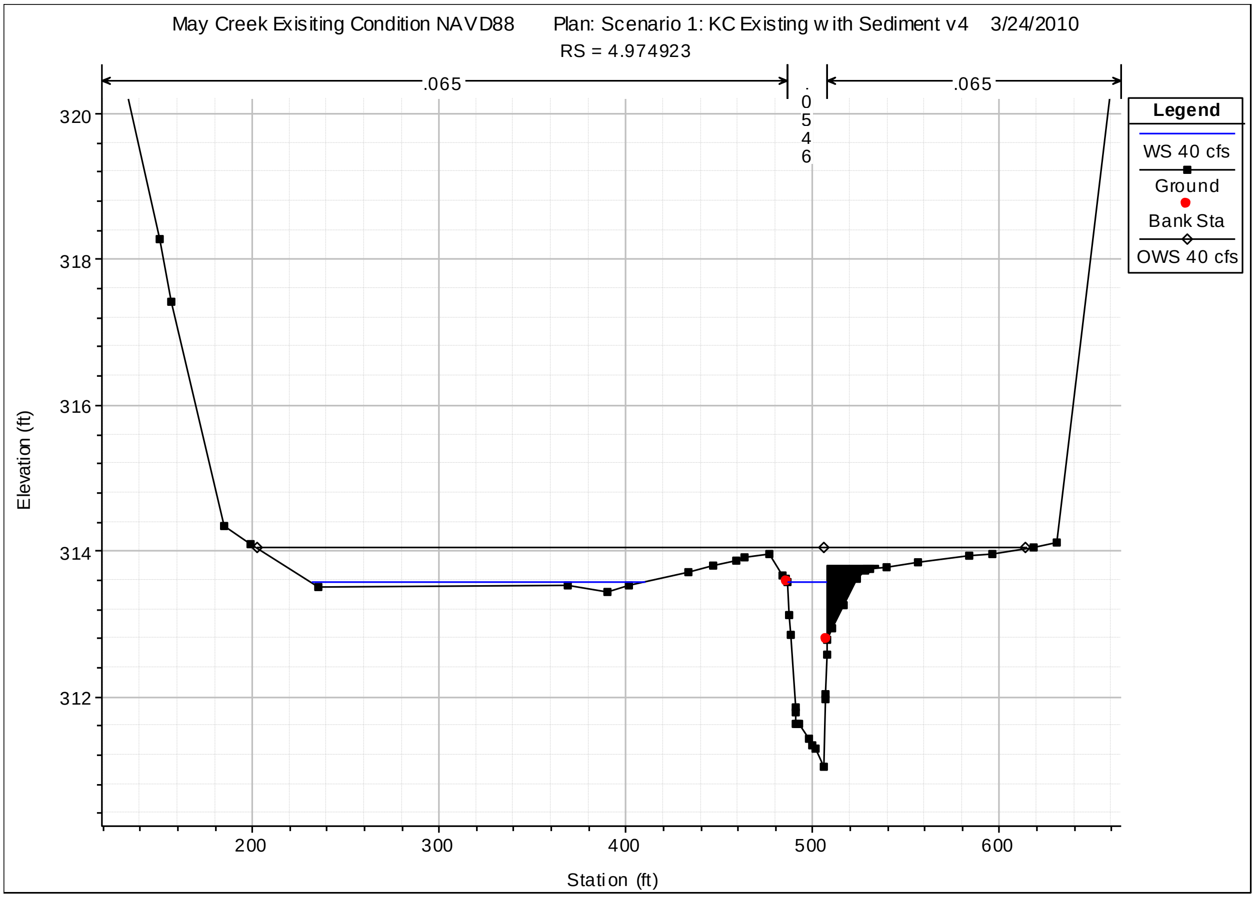

Figure 14 Example of cross-section where water surface elevation is same elevation as bank elevation (RM 4.974). Obstruction in cross-section is representative of dense canary reed grass on the banks.

Figure 15 Example of overbank flooding slowly draining back into channel after a storm from 7 days prior (with some minor precipitation 4 days prior). Photo taken 3/19/2010.

January 14, 2010 Water Surface Observation

A second observation of water surface was made during the larger storm event between the two survey dates on January 14, 2010 1:00 pm, at Parcel #0223059005 footbridge (tape down from top of bridge was used). Using the gauged flow rate at 148th Ave SE bridge of 64 cfs, the water surface profile was within a two tenths of a foot to observed (see Figure 16).

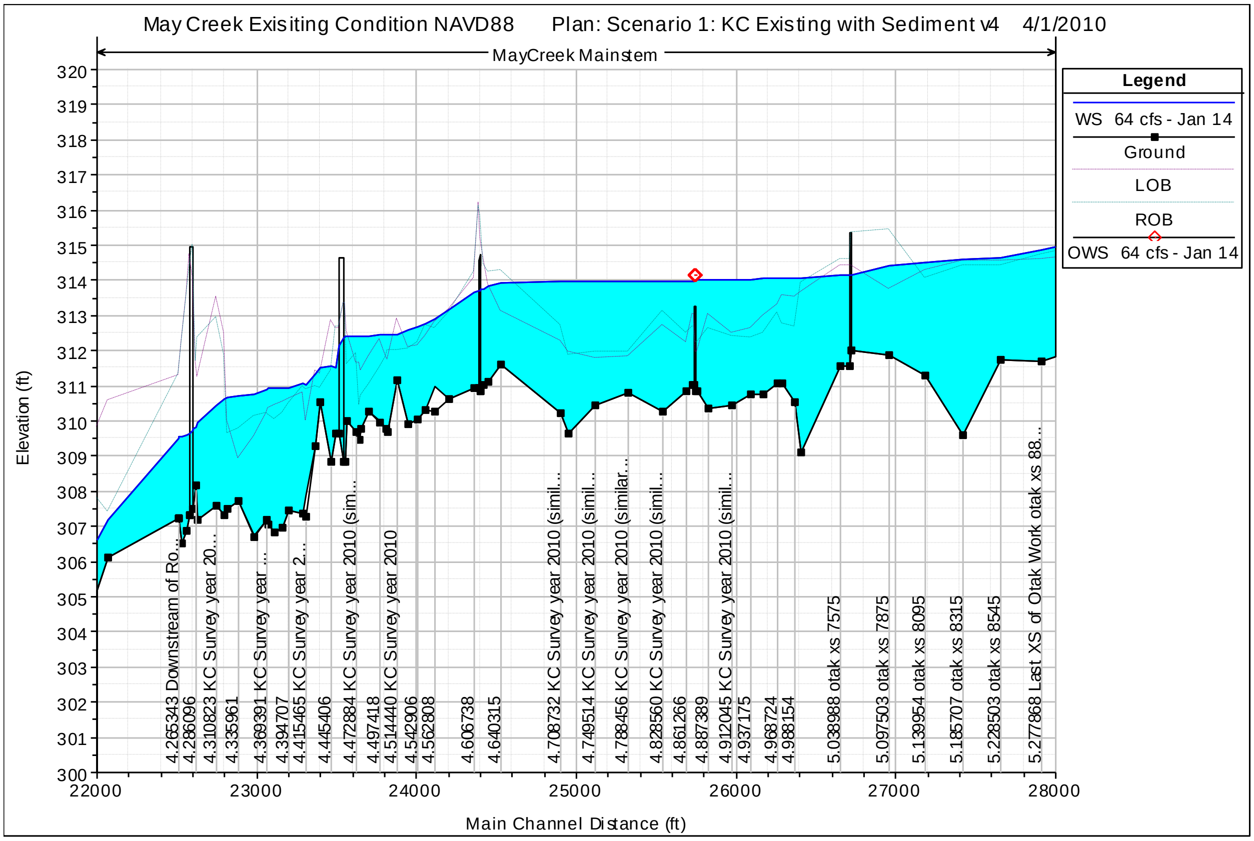

Figure 16 Observed water surface elevation for January 14, 2010 at 64 cfs at Parcel #0223059005 footbridge

March 30, 2010 Observed Water Surface Elevations

In an attempt to capture water surface elevations for smaller storm events, another site visit was conducted. Water surface elevations were obtained by taping down from top of bridge. Additionally, flow rates were estimated using standard methods and velocity meters at each water surface observation point. Observations were made at three locations, 148th Ave SE bridge, McFarland footbridge, and parcel #0223059005 footbridge with their respective estimated flow rates of 45 cfs (poor quality), 48 cfs (good quality), and 40 cfs (good quality). Long Marsh creek enters in upstream of the McFarland footbridge, hence the increase in flows at that measurement. Using these field measured flow rates, model accuracy validates with good accuracy with the greatest error equal to 0.50 feet. A water surface profile and observed water surface elevations is shown in Figure 17 below, and previously in Table 6.

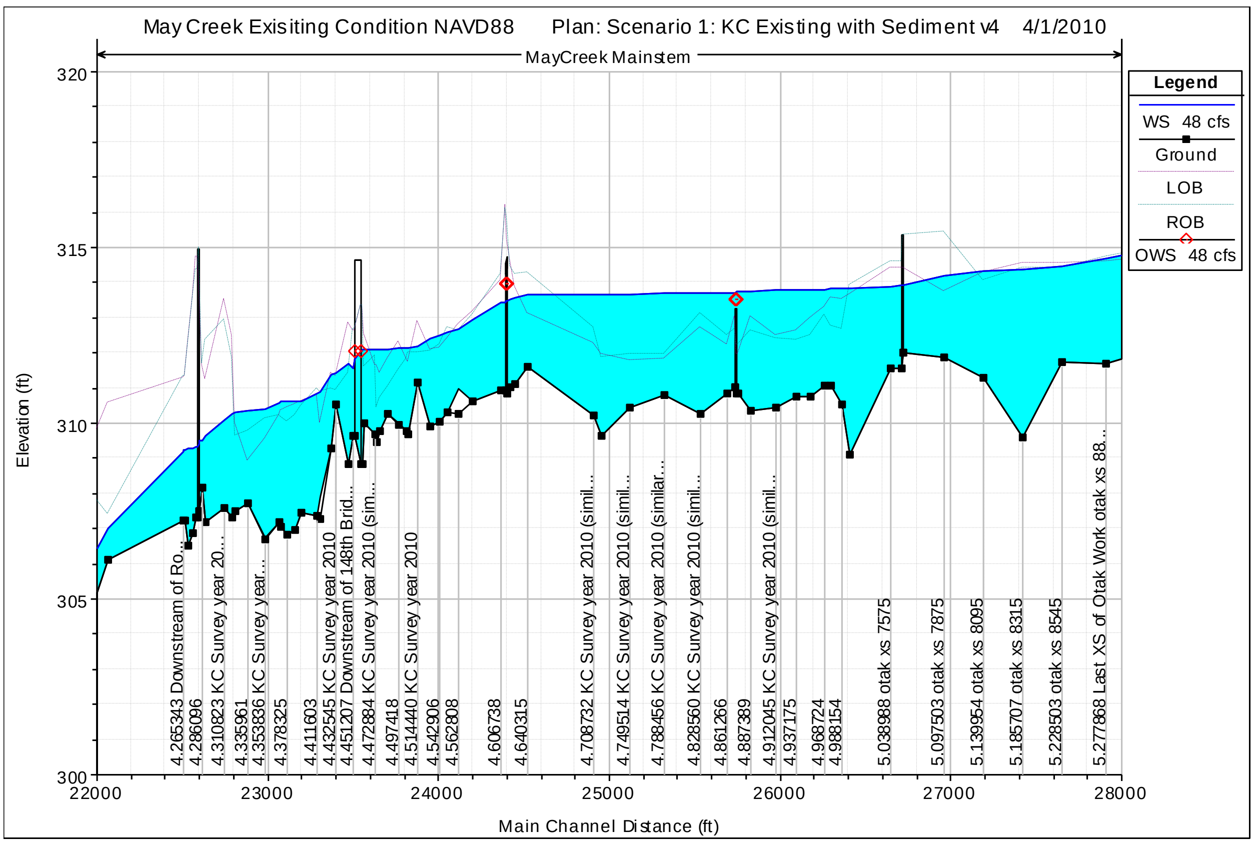

Figure 17 Observed water surface elevation for March 30, 2010 at 48 cfs at three bridges.

2.4.4.1 Validation Summary

This validation shows that the model under predicts water surface elevations for lower flows in the Valley floor anywhere from 0.06 feet to 1.3 feet (assuming calculated stream flows are accurate, but appear suspect) with most of the differences in the range of 1.0 feet and has better accuracy with higher flows (e.g. 48 and 64 cfs water surface observation). One hypothesis has been presented to partially account for the discrepancies, however there are multiple other plausible causes for the elevated water surfaces for low flows: some of which might be, error in gauge flow estimates at 37G, or unaccounted for choke points in the channel. At present, assuming the flow rate estimates are correct, it is very unlikely that water surface elevations could be as high as surveyed. To reproduce those elevations for those low flow rates, the McFarland footbridge would have to be nearly completely damned to backwater upstream that high. While it is not known if this may have happened, it again seems unlikely. Therefore, while the accuracy of the survey data is not in question, the combination of assumed accuracy in flow rates and the edge of water survey in flooded conditions appear to represent a set of conditions neither characterized in the model configurations nor explainable in their contradictions. Therefore those two dates of observations should not be considered part of the validation. Conversely, model accuracy seems to be quite good for larger storms with error less than or equal to 0.50 feet. However, further model validation is still being pursued at this time targeting storms in the range of 10 to 20 cfs.

3.0. SCENARIO DEVELOPMENT

Hydraulic scenarios were designed with changes in channel geometry representative to the three proposed channel improvements: 1) removal of choking vegetation points, 2) removal of choking vegetation points with increased sediment removal to an elevation of 308 feet, and 3) removal of choking vegetation points with some sediment removal to an elevation of 309 feet.

3.1.1 Model Geometry

Scenarios were designed to characterize existing and proposed conditions in the valley. The first scenario includes characterizing existing conditions, and three other scenarios were designed to evaluate increasing levels of channel modification to address the objectives. The naming of the scenarios are representative of the stepwise process of developing the geometry files to get from one scenario to the next in HEC-RAS rather than suggesting that multiple additional scenarios were evaluated but not presented in this report.

Scenario 1: Existing conditions

Survey work done in January 2010 included two channel bottom elevations: on top of soft sediment, and harder substrate assumed to be the more historical channel bottom. Existing conditions is meant to represent current channel geometry with channel bottom defined as on top of soft sediment.

Scenario 7: Removal of vegetation choke points

This scenario represents enhancements to the channel conveyance capacity by assuming Reed Canary grass is removed from the channel and banks, and Willow root systems are removed from the channel. The assumed channel bottom for this scenario and subsequent scenarios are based on the harder substrate sediment. This is based on the calculation of critical shear stress of silts in the channel and steam competency to mobilize the silts (see Section 4.0).

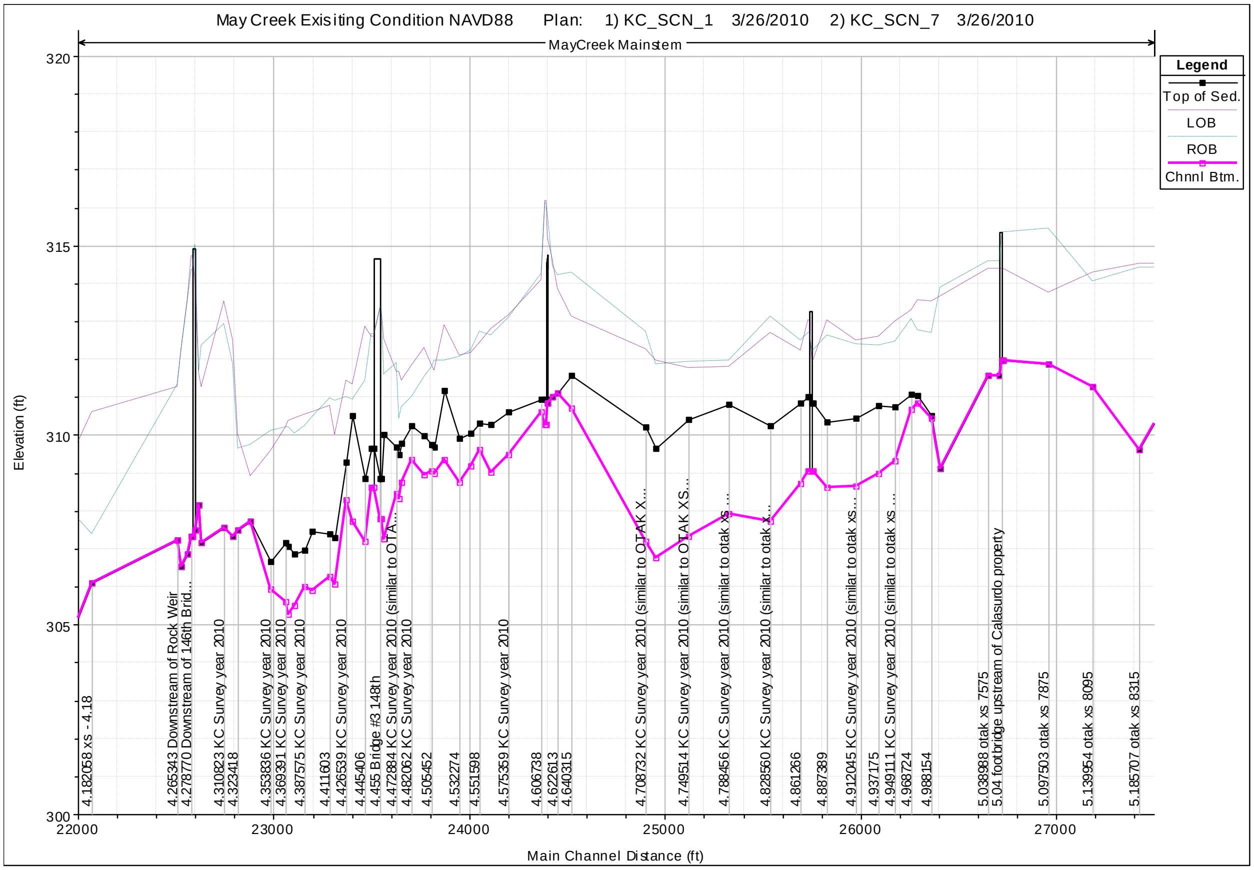

Figure 18 Scenario 7 longitudinal profile in study area. The black line is channel bottom using top of sediment (used in Scenario 1) and fuschia color line is profile of channel bottom to firm sediment.

Scenario 8: Removal of vegetation choke points and increased sediment removal

This is the most aggressive scenario with sediment removal assumed to occur between just downstream of 148th Ave SE bridge to the 125 ft upstream of McFarland footbridge—in total approximately 1025 ft at an elevation of 308 ft. The elevation of 308 ft was selected based on the apparent historical channel bottom at 148th Avenue SE bridge crossing.

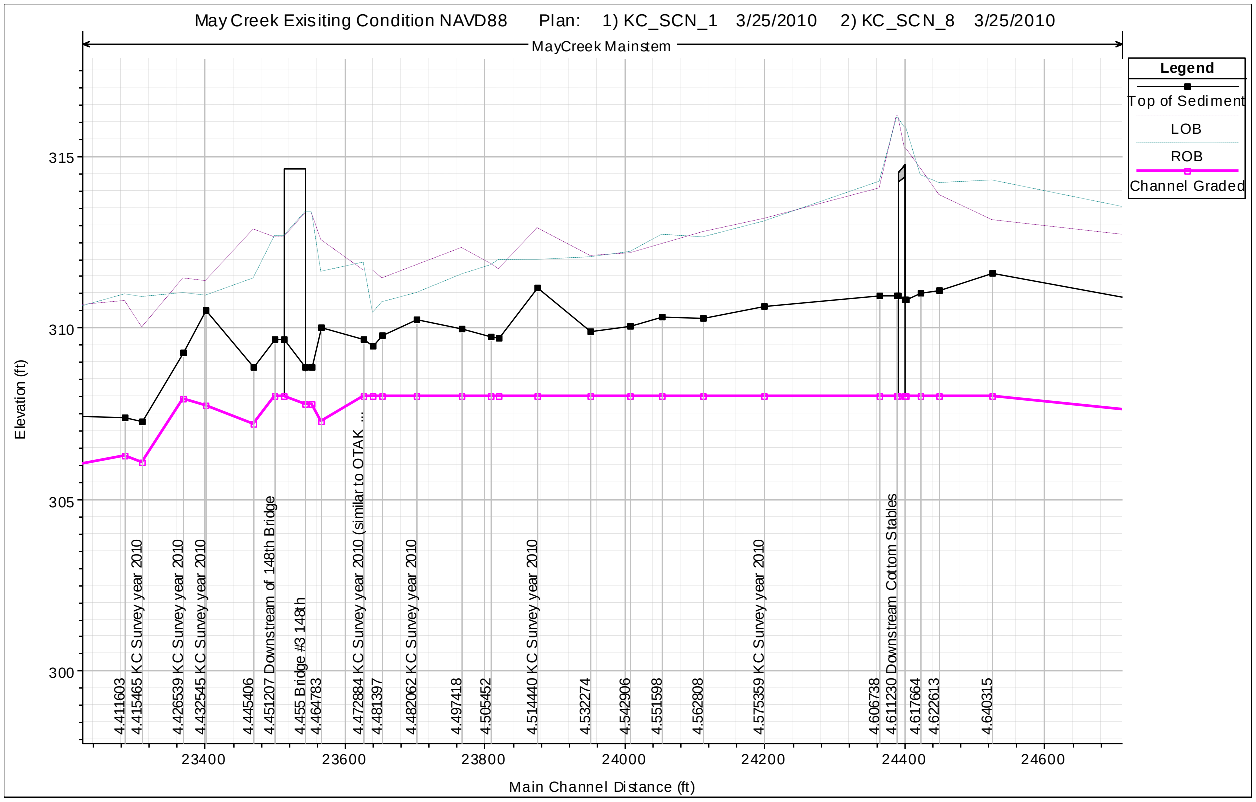

Figure 19 Scenario 8 showing existing conditions and proposed profile after sediment removal (308 ft) and flushing of silts.

Scenario 9: Removal of vegetation choke points and some sediment removal

Approximately 518 ft of sediment (125 ft upstream, 393 ft downstream) of McFarland footbridge is assumed to be removed to an elevation of 309 ft. This elevation was selected to be similar to historical channel bottom elevations leading down to 148th Avenue SE bridge crossing and to evaluate an intermediate alternative.

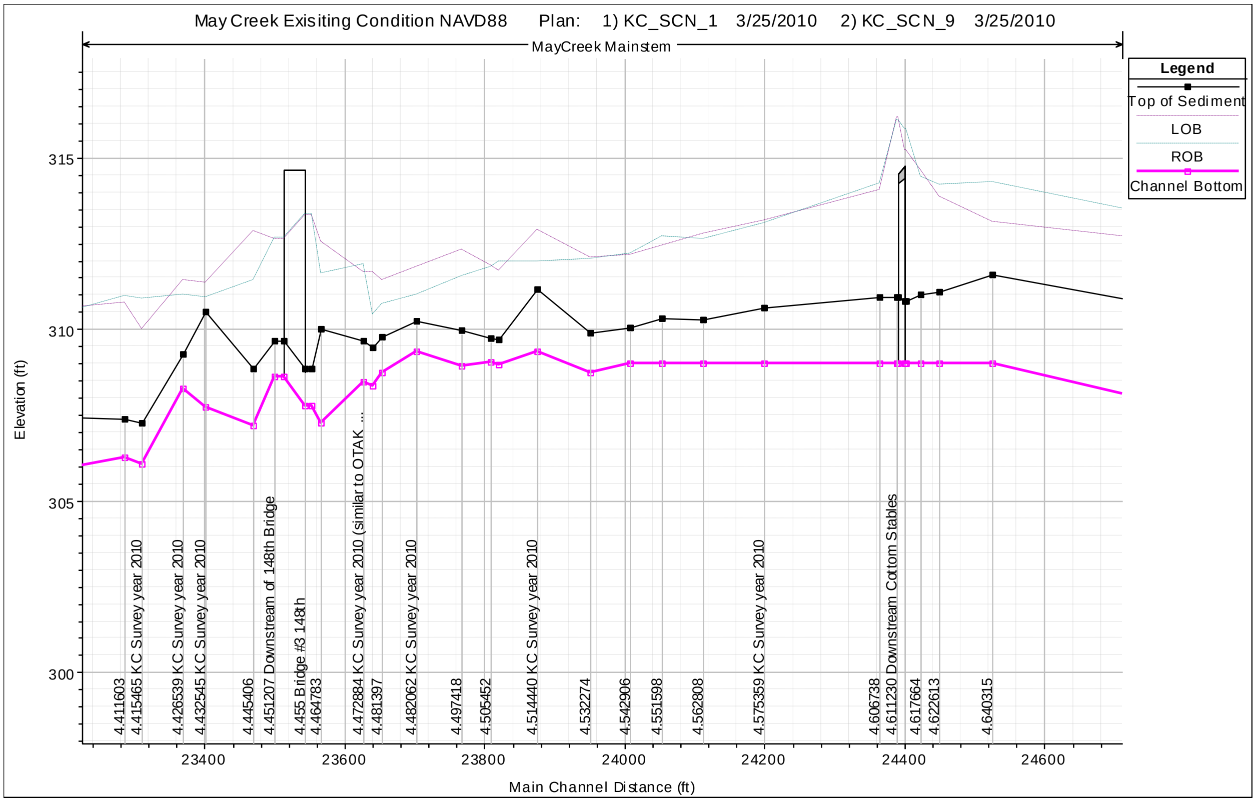

Figure 20 Scenario 9 showing existing conditions and proposed profile after sediment removal (309 ft) and flushing of silts.

3.1.2 Typical Channel Cross-Section

Typical Channel Cross-section within the sediment removal segments was simplified for this study to assume existing channel geometry with the bottom dropped to the proposed elevation. In final design, sections where excavation exceed 2 feet below top of sediment, channel banks will be given side slopes to prevent bank sloughing. This simplification represents a conservative side of expected as-built conditions given the addition of side slopes will slightly increase channel capacity.

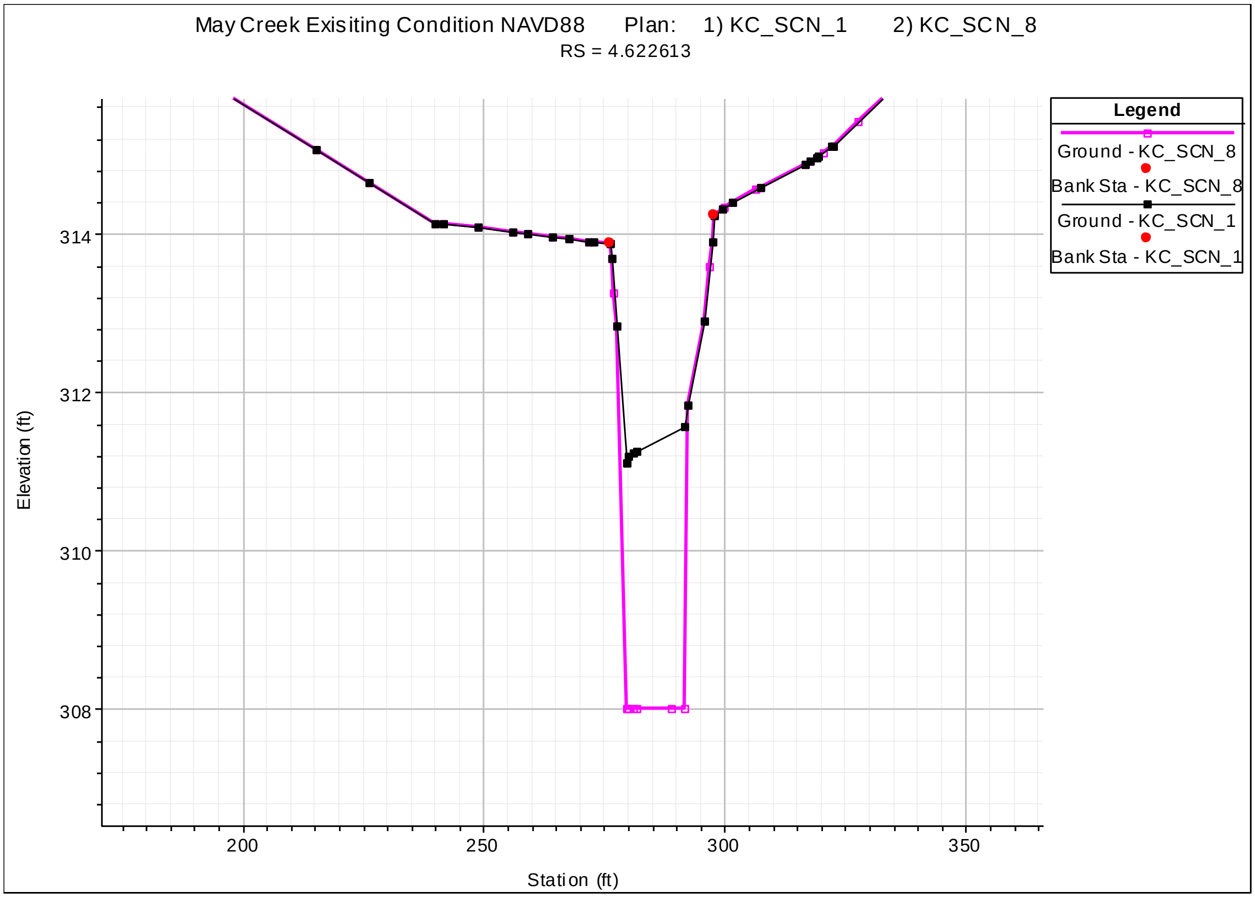

Figure 21 Typical Channel Geometry of existing (black line) and proposed (fuschia line).

4.0. SEDIMENT TRANSPORT

A particle size distribution was not performed for stream reaches in the valley floor; however, during field visits, it was noted that much of the channel bottom is extremely soft—in some places up to 3 feet of *muck*. Given the amount of bank vegetation and slow velocities, the soft sediments are likely made of up fine silts and organic matter from decaying vegetation. Given this condition, the particle size distribution (D_{50}) used for incipient motion in the valley channel was assumed to be 0.10 mm diameter (0.000328 ft). Aside from the percent of organic matter and possible colloidal conditions, incipient motion was calculated assuming the sediment is non-cohesive in nature and made up of mostly silt. While Shield's curve is nearly constant for substrate sizes larger than 5 mm (Re* ~ 400), it varies with smaller particle sizes. Thus it was necessary to compute the particle Reynolds number to obtain the Shields value (Guo 2002). Using a particle diameter of 0.1 mm, translates to a Re* approximately equal to 1.1, and Shields number of approximately 0.10 (Figure 22). Hence, the computed critical shear stress of less than 0.01 psf is estimated for silty fines.

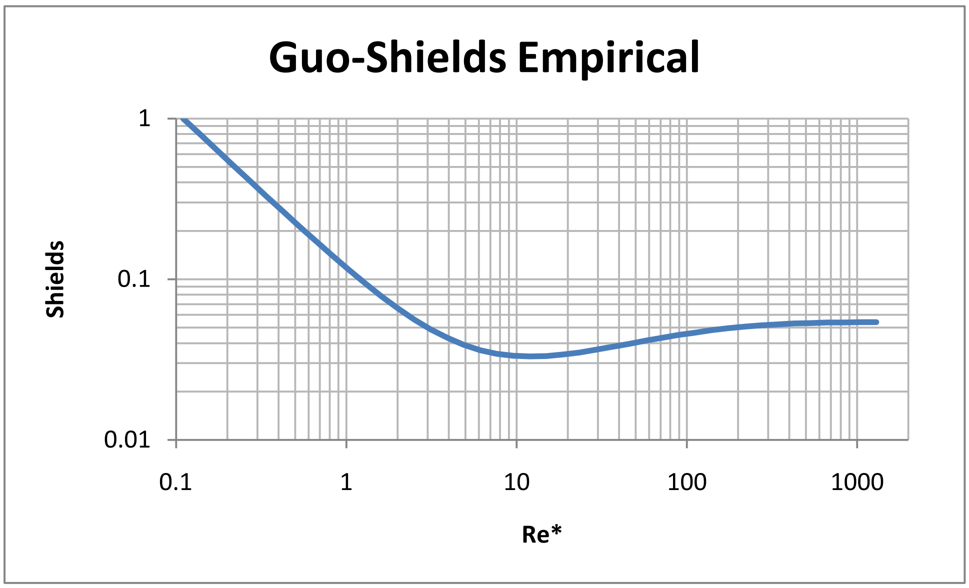

Figure 22 Guo-Shields Empirical Curve

In Hec-RAS, the shear stress is computed with the following formula:

$$\tau = \gamma RS ,$$

where R is the hydraulic radius, and S is the energy slope.

5.0. RESULTS

Summary of results include assessments of existing and proposed channel capacities, changes in flood frequencies and durations, stream competency to mobilize fines, and changes in erosion in the ravine.

Reviewing existing conditions, it is apparent where hydraulic controls are located in the system (based on available survey data). During mean annual flows (8.6 cfs through the study area), control points are vegetation choking points in the wetland downstream of 148th Ave SE bridge and mildly so upstream of 148th and gravel deposition where Long Marsh enters into May Creek at approximately river mile 4.64, just upstream of a footbridge. This high point of gravels controls the water surface elevation upstream approximately for 2000 feet to a footbridge located approximately at river mile 5.04 (Figure 23). Similarly for higher flows (e.g. 1 year event), Long Marsh again controls water surfaces upstream for the same reach length.

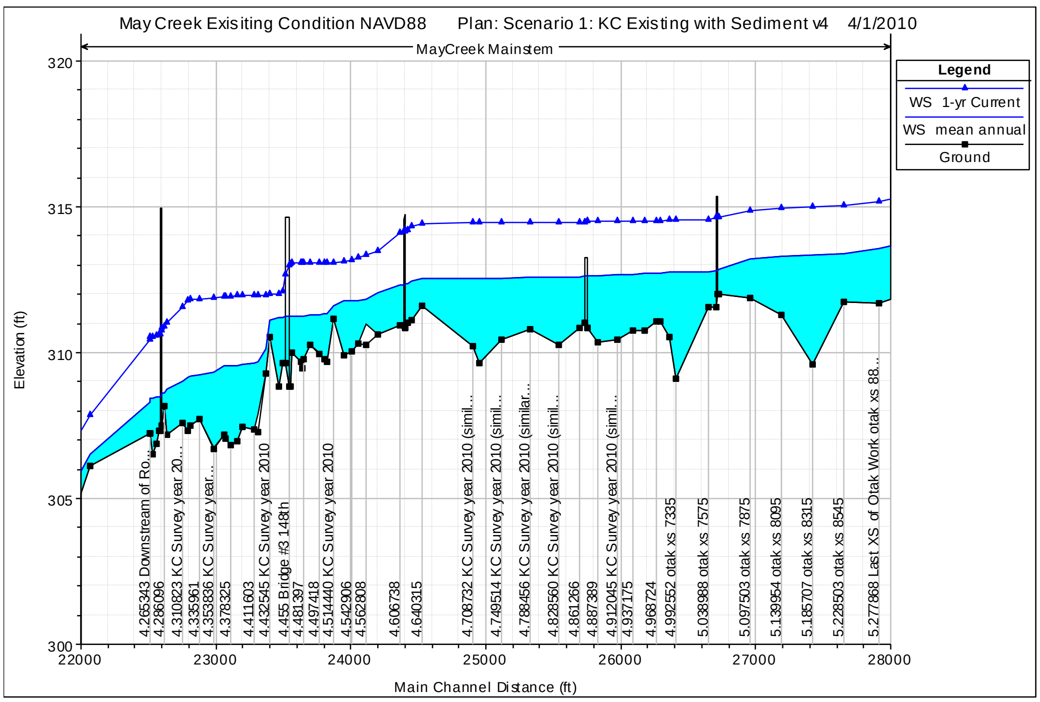

Figure 23 Scenario 1 (existing conditions) water surface profile for mean annual (filled in water surface) and Conditions Report 1 year event (blue line with symbols).

However, downstream of 148th model runs show a convergence of water surfaces for the same flow rates for pre (Scenario 1) and post (Scenario 8) project based on the transition from a valley to a ravine. This abrupt natural constriction changing from open wetland on valley floor to a well defined channel entering into the ravine become more controlling the larger the storm event. While water surface elevations may be lower for Scenario 8 in the wetland for the same flow rate, water surface elevations approaching 146th Ave bridge converge to the same elevation (Figure 24).

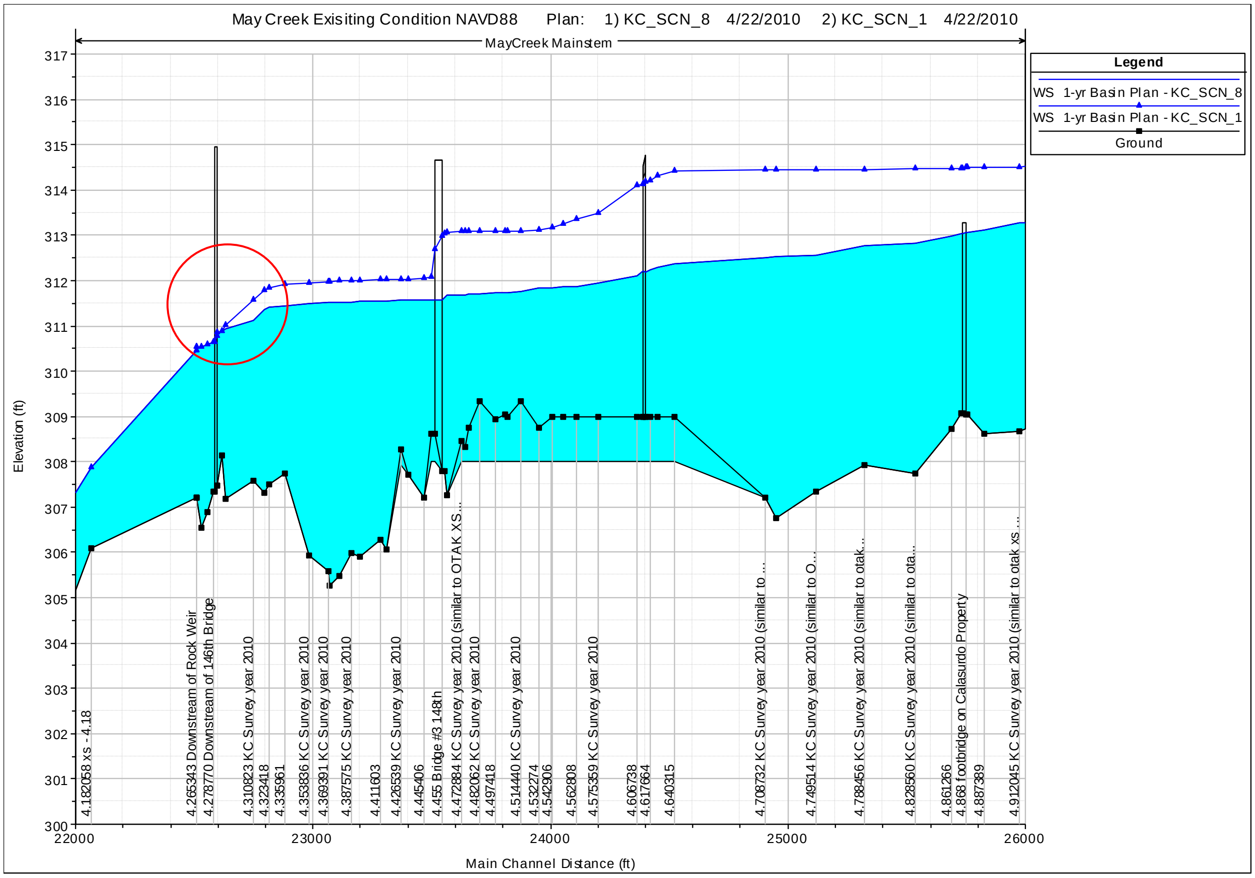

Figure 24 Water surface profiles for the 1-year (Conditions Report) for Scenario 1 and 8. Red circle highlights the convergence of profiles at 146th Ave bridge.

Secondarily, removal of vegetation choke points in the wetland show a few tenths change in water surface, but given the model accuracy and very small amount of lost storage, this natural land form downstream of the proposed restoration channel activities will greatly control potential changes in erosion in the ravine, and less control from the bridge at 146 Avenue SE.

In the following sections, three scenarios (plus existing conditions) were focused on for evaluations:

- Scenario 1—existing conditions,
- Scenario 7—removal of vegetation choke points,
- Scenario 8—removal of vegetation choke points with increased sediment removal, and
- Scenario 9—removal of vegetation choke points with some amounts of sediment removal.

As previously mentioned there are three main control points in the system under existing conditions: 1) the natural transition from valley to a ravine, 2) vegetation choking the channel downstream and upstream of 148th Ave. SE, and 3) sediment depositions upstream of 148th to the confluence of Long Marsh Creek. Each of the proposed scenarios improve in channel conveyance to varying degrees of success with Scenario 8 resulting with the ability to maintain waters in channel up to approximately 50 cfs for properties upstream of 148th Ave SE.

Estimated hydraulics for Scenario 8 elucidate that the control points effectively move down to the transition of the wetland to a channel entering the ravine for lower flows (Figure 25) and for high, infrequent flows (Figure 26). This characteristic supports the results of no increases in erosive flows to the ravine before/after the proposed project for the same flow rates.

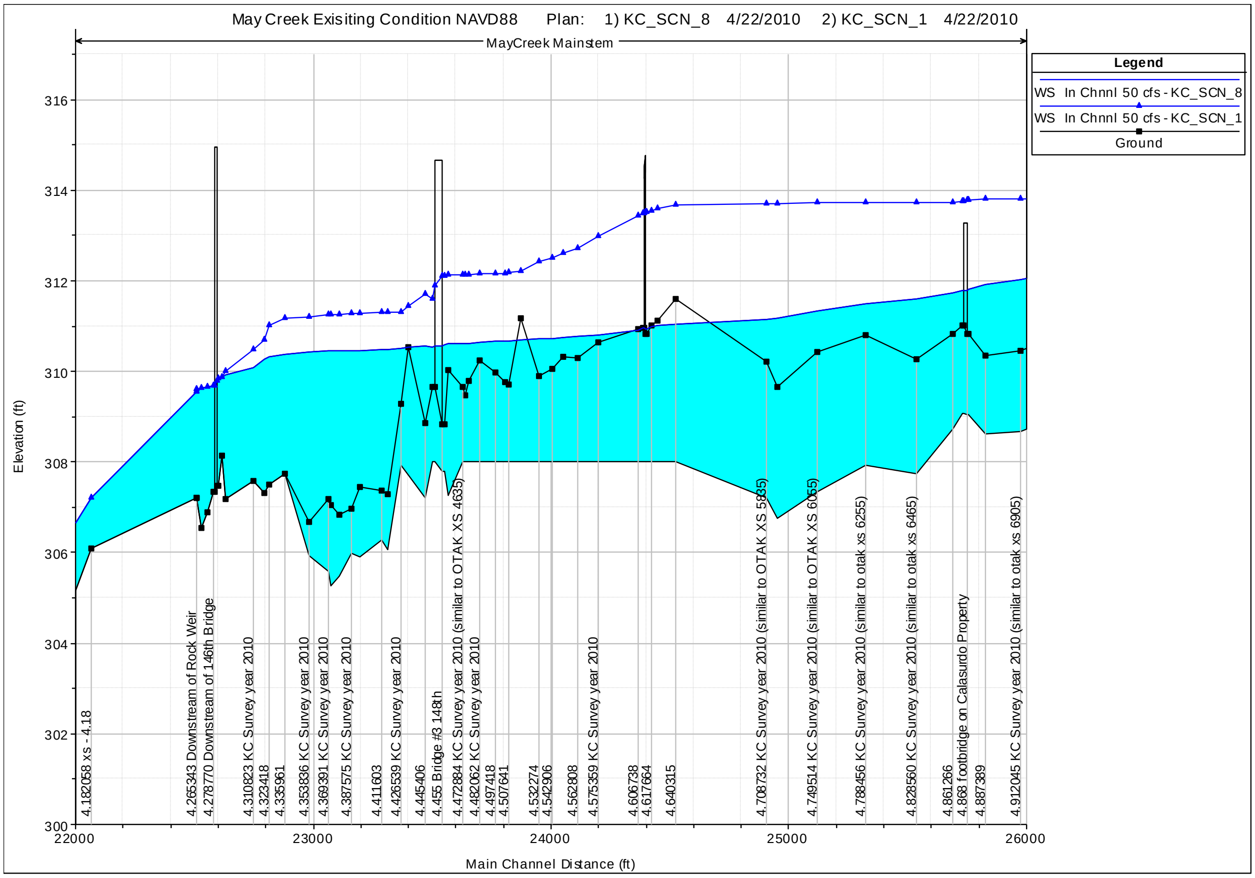

Figure 25 Water surface profiles at 50 cfs for Scenario 1 and Scenario 8.

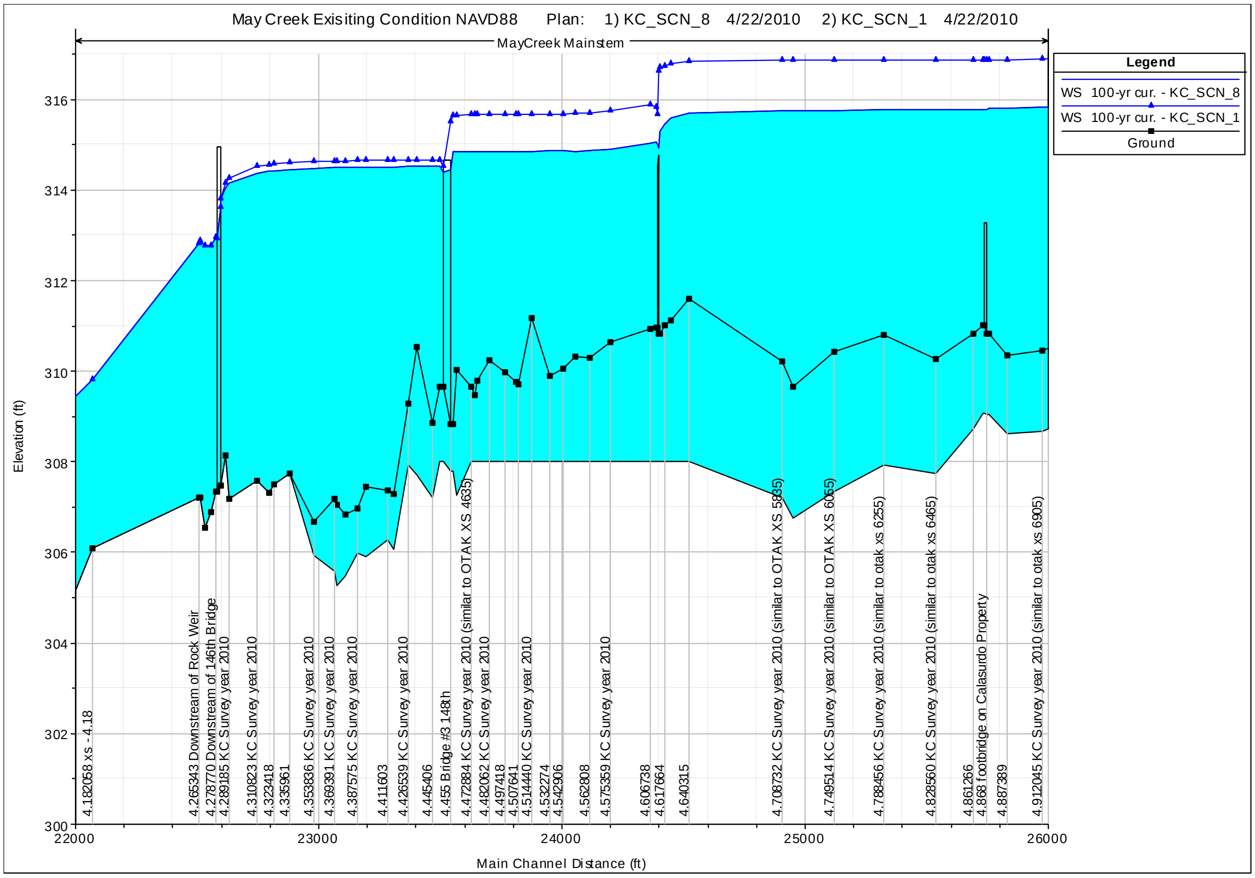

Figure 26 Water surface profiles for the 100 year flood event for Scenario 1 and 8.

Capacity at the bridge located at 146th Ave SE show that the 100-year (Conditions Report) flow rate does go overbank (left side looking downstream) under both existing and proposed conditions and at the same elevation. Again it is worth noting that under the proposed project, the magnitude of the 100 year return period decreases, water surface elevations post project will be less (Figure 27). Similarly, the bridge downstream at 143rd Ave SE shows to have capacity to pass the 100-year pre and post project (Figure 28) as well. It is worth noting that the bridge geometry used for 143rd Ave SE is based on previously existing geometry from the previous HEC-RAS model. A survey crew is scheduled to resurvey this bridge and confirm existing geometry from previous modeling efforts.

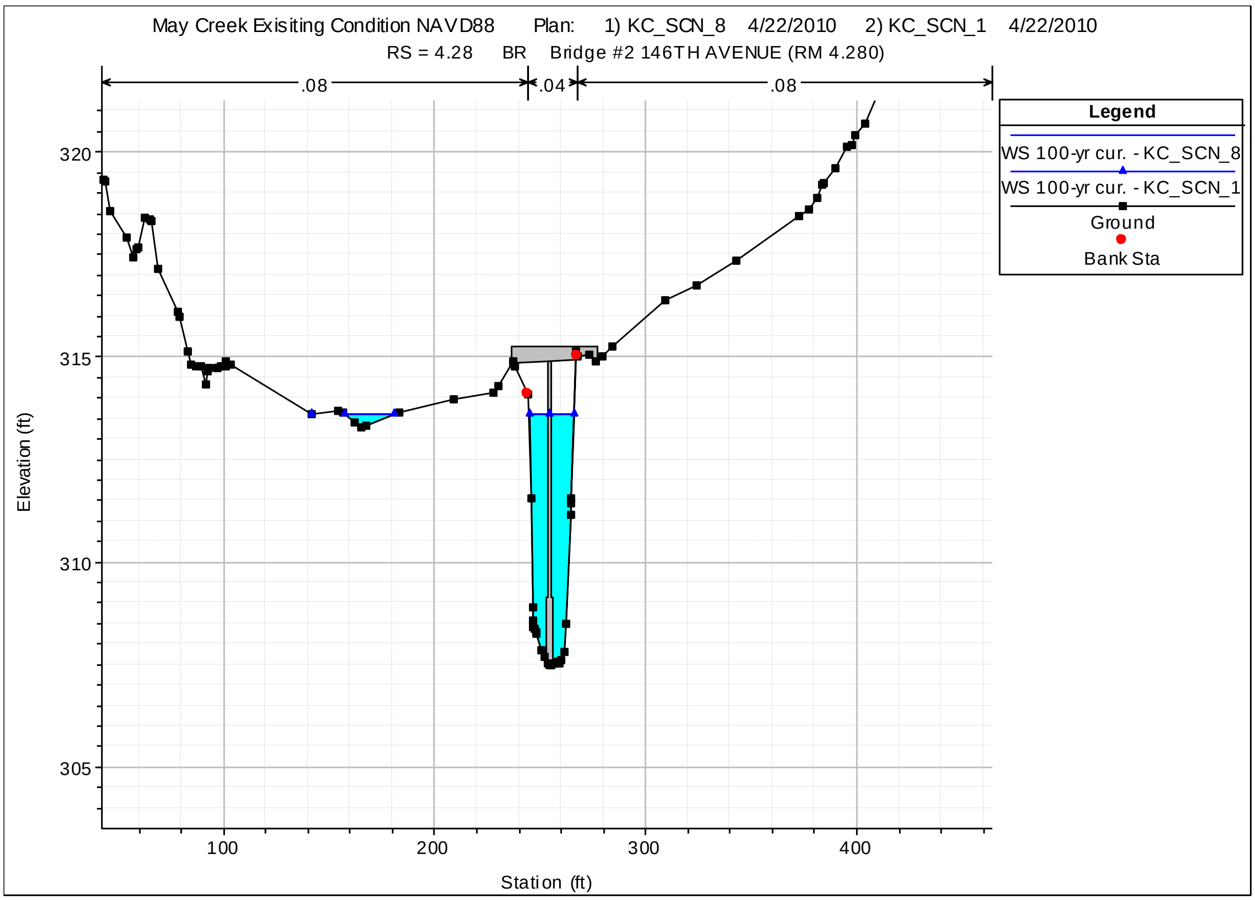

Figure 27 Water surface elevations (Scenario 1 and 8) for the 100 year (Conditions Report) at the 146th Ave bridge.

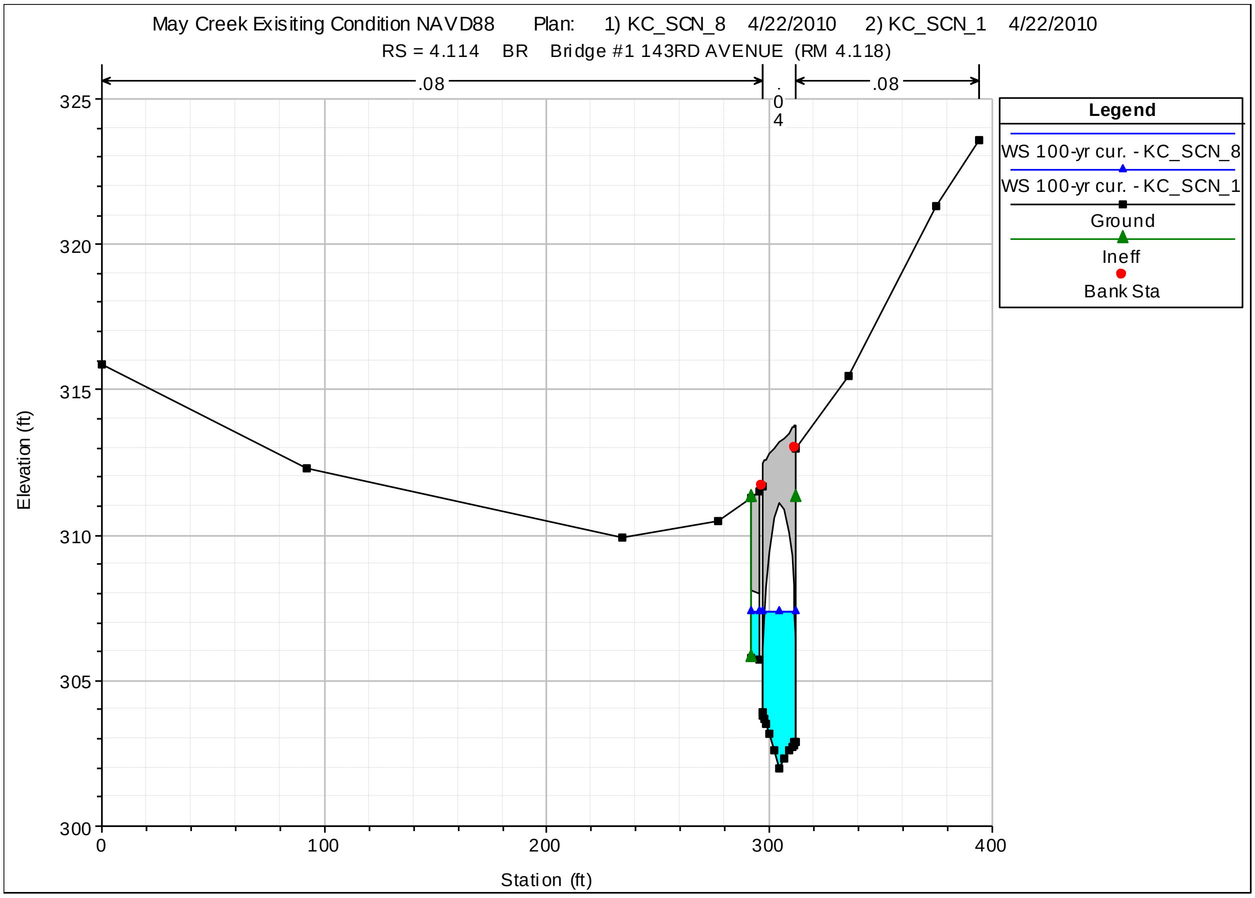

Figure 28 Water surface elevations for the 100 year return period (Current Conditions) at bridge crossing at 143rd Avenue SE for Scenario 1 and 8.

5.1 Improved Channel Conveyance

Hydraulic analyses using HEC-RAS estimate that Scenario 1 (i.e. existing conditions) flow rates as low as mean annual (8.6 cfs) over top banks inundating pasture lands (Figure 29). In fact, channel capacity estimates for a few sections show that flows can go out of bank at rates as low as 6 cfs for existing conditions (Figure 30) while Scenario 8 (later in this section) keeps flows in channel up to 50 cfs. Thus calculations for evaluating improved conditions in May Valley are based on this threshold of flows between 6 and 50 cfs, such that any improved conveyance capacity will reduce the frequency and duration of minor storm events flowing out of bank.

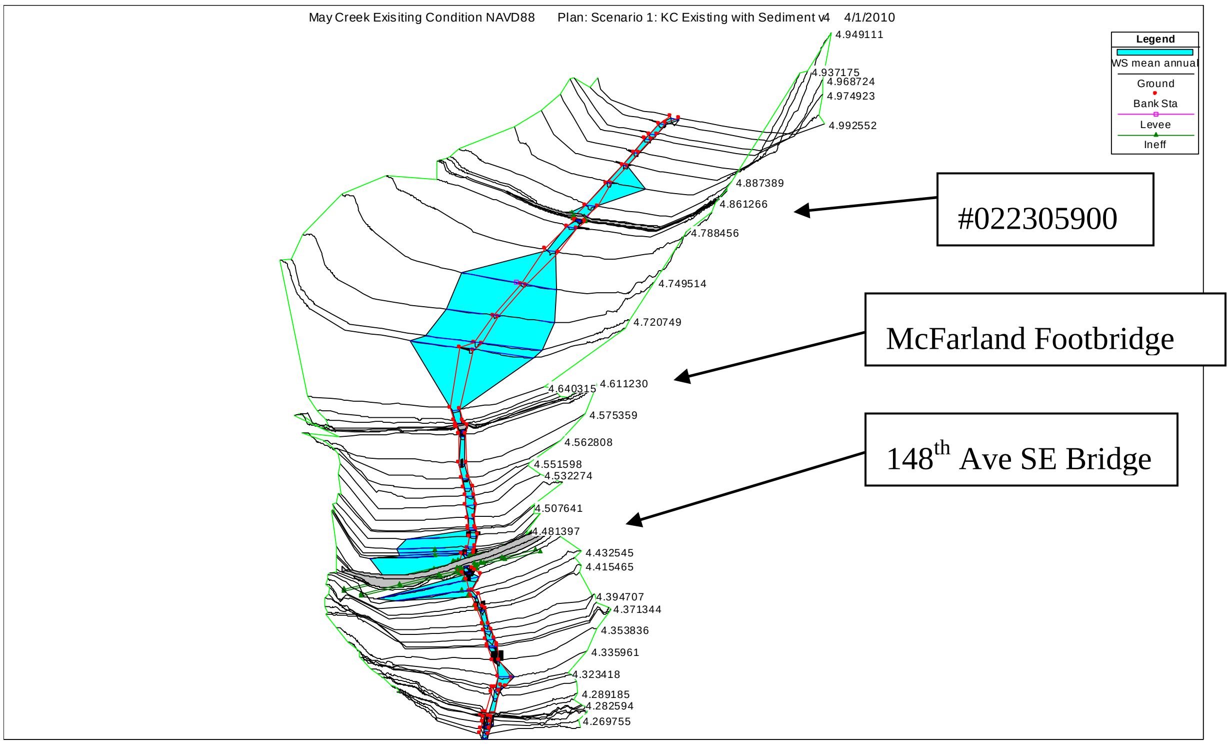

Figure 29 Perspective plot for Scenario 1 (Existing Conditions) mean annual flow rate (8.6 cfs at 148th Street)

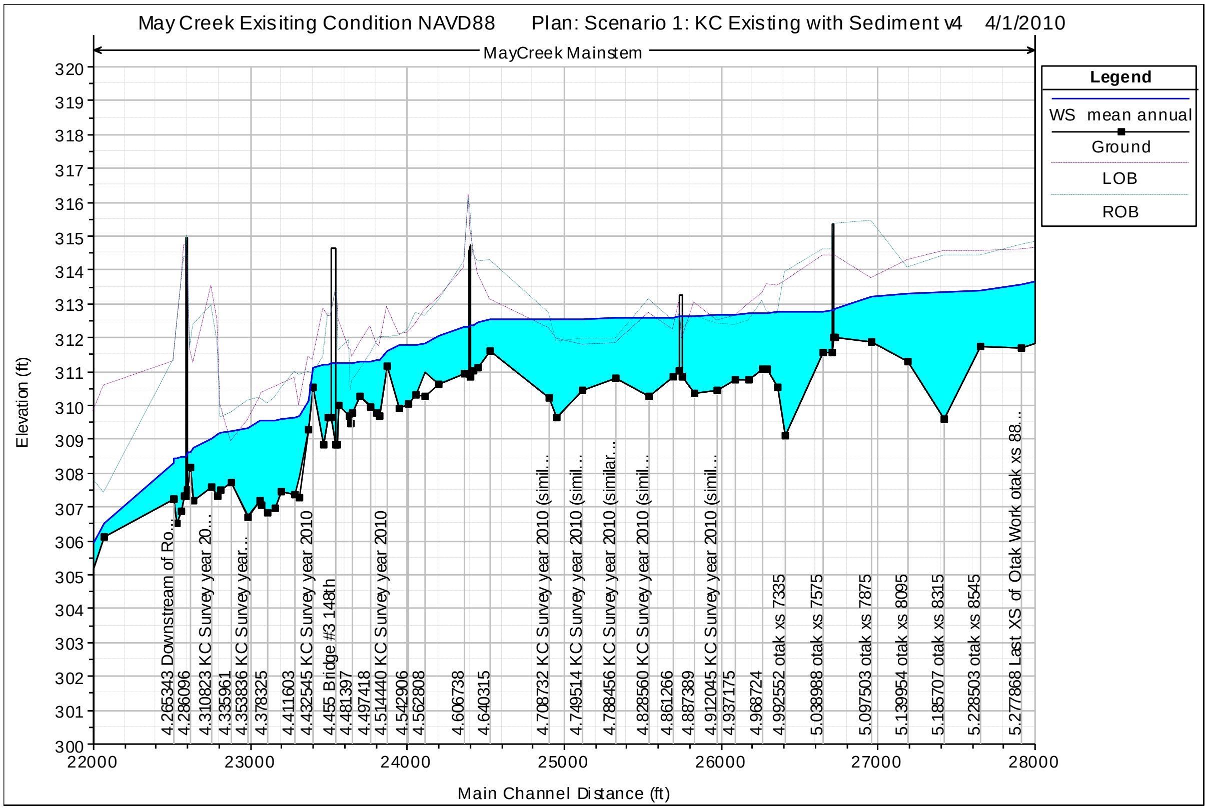

Figure 30 Longitudinal plot for Scenario 1 (Existing Conditions) water surface profile for mean annual flow rate.

Scenario 7

With complete removal of encroaching canary reed grass and Willow trees, flows are kept in bank between 148th Ave SE and at the footbridge just downstream of Long Marsh Creek (Figure 31). However, this is predicated on the fact that all the silty fines are flushed out of the system. Without that successful element, in-channel capacity will be greatly reduced and more representative of existing conditions (Figure 32). One of the most effective choke points to be removed is just downstream of 148th Ave SE. The combination of canary reed grass and Willow trees significantly reduce potential channel capacity at this location and a few others further downstream.

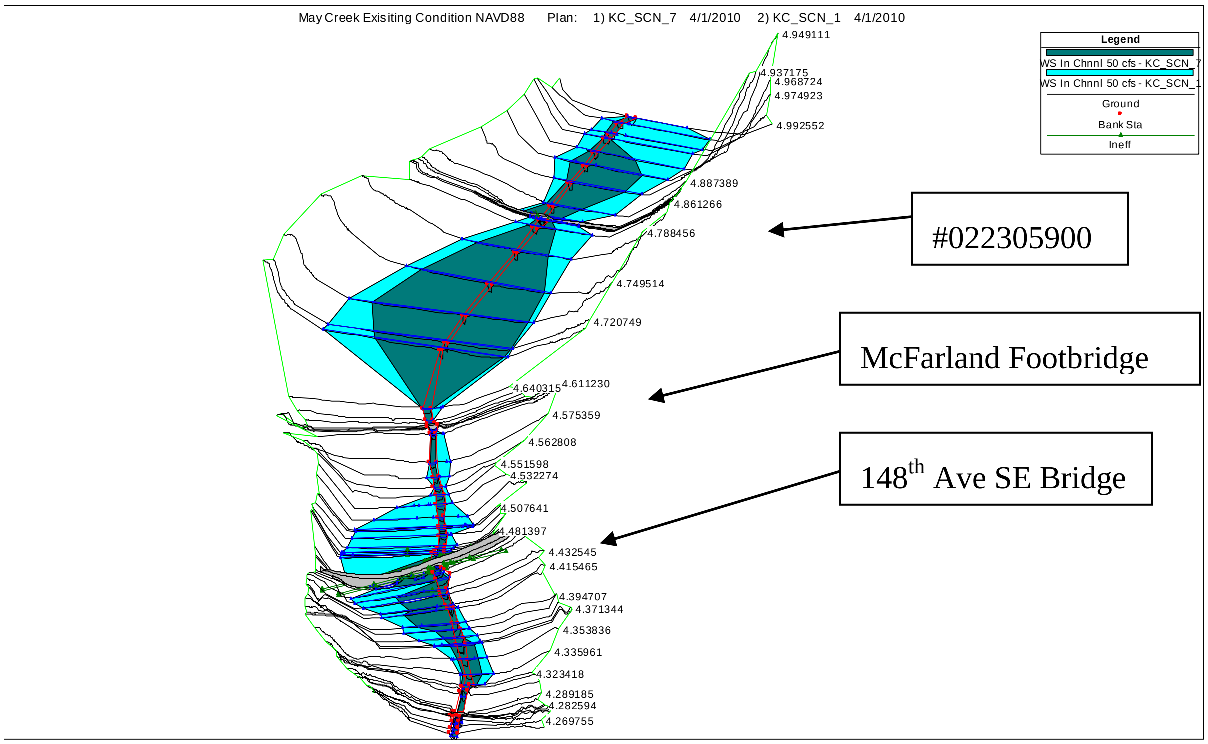

Figure 31 Perspective plot for Scenario 1 and 7 (vegetation removal) 50 cfs at 148th Street. Light blue are for existing conditions while dark blue are for Scenario 7.

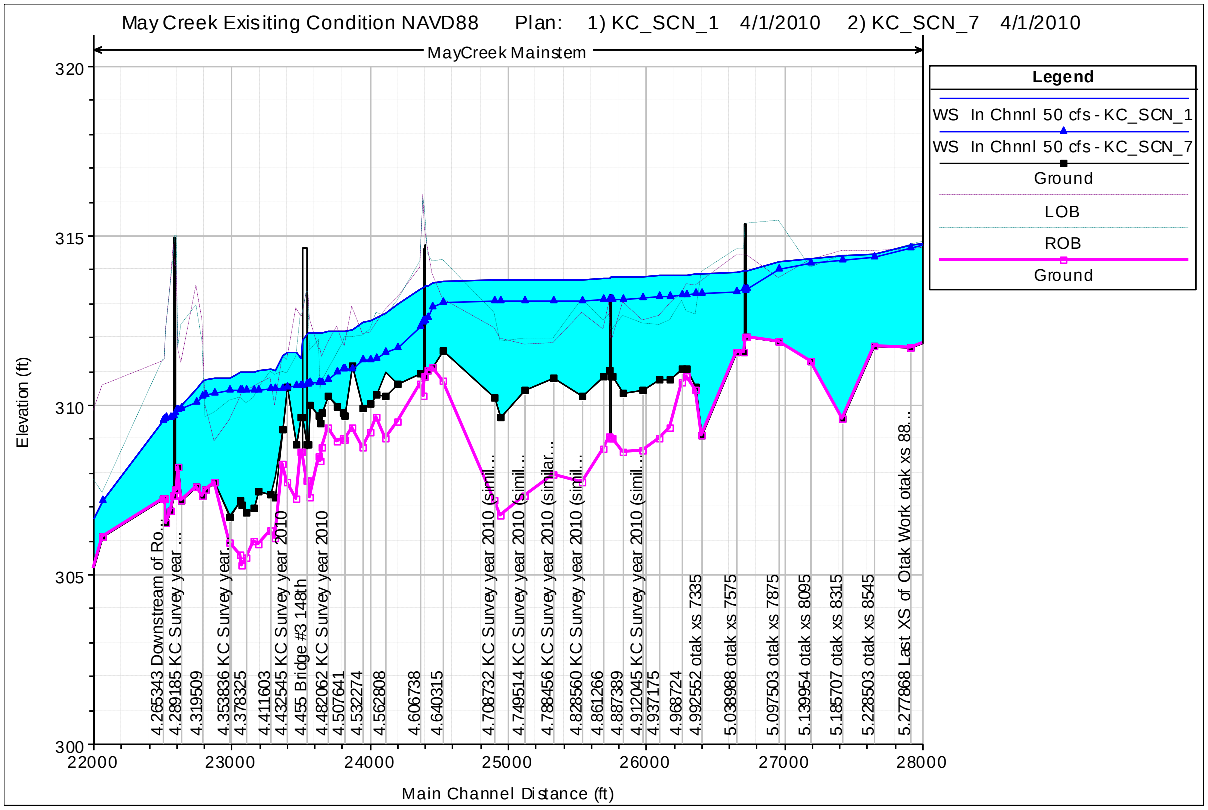

Figure 32 Water surface profile at 50 cfs for Scenario 1 and 7

With only a water surface reduction of half a foot through the study area, conditions upstream of Long Marsh are clearly unimproved given flooding still occurs at the mean annual flow rate (Figure 39). Furthermore, storm flows may begin to overtop downstream of Long Marsh depending on the magnitude of the event again not meeting project goals.

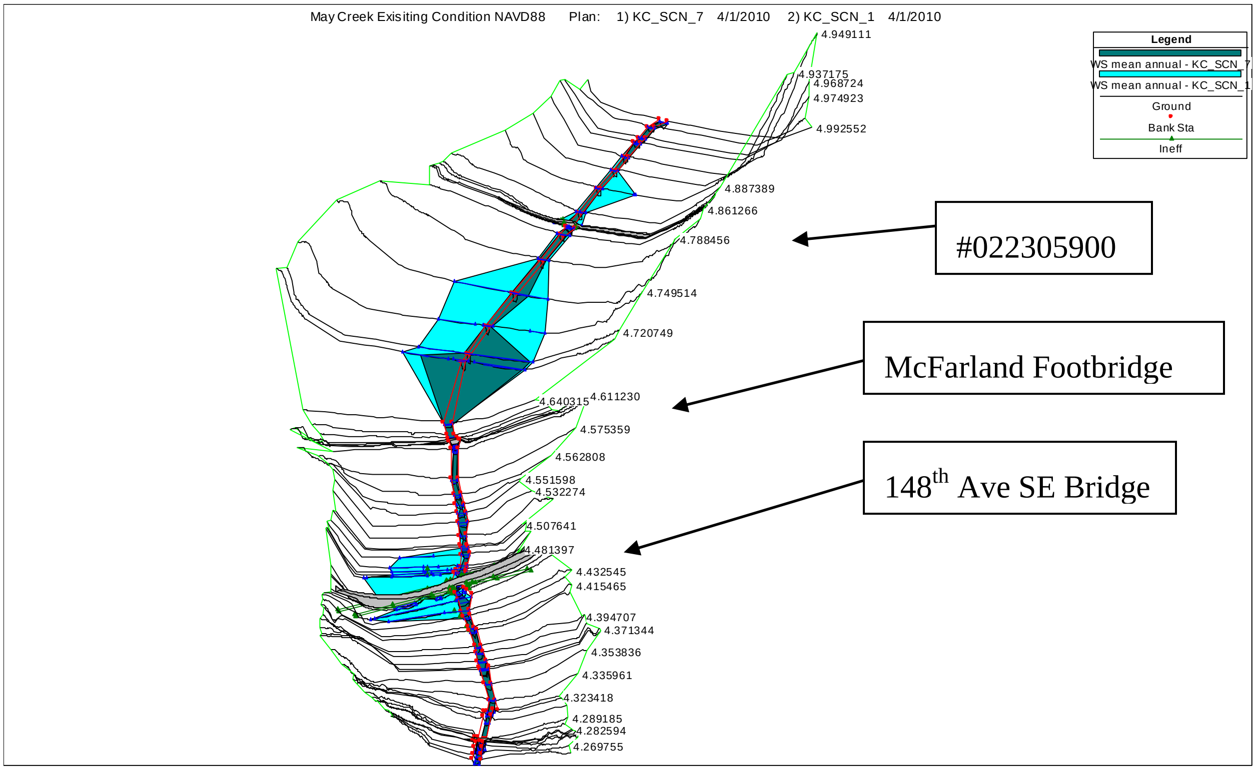

Figure 33 Perspective plot comparing Scenario 1 and 7 under mean annual flow rates

Scenario 8

Under the proposed project design, Scenario 8, with vegetation removal and sediment removal down to 308 feet for approximately 1000 feet, the conveyance capacity increases to approximately 50 cfs in the previous sections where 6 cfs would be out of bank. This improvement effectively mitigates storms near the 1-year magnitude (i.e. 61 cfs). Figure 34 shows that for 50 cfs, May Creek is over bank for existing conditions in the entire project area while for Scenario 8, flows are kept in-channel until the wetland area downstream of 148th Ave SE (shown in dark blue). Hydraulic controls in the system coalesce down to the outlet of the wetland entering into a well defined channel leading to the ravine, with a small difference in water surfaces through the wetland area as a result of removal of vegetation choke points (Figure 35). One of the primary choke points to be removed affecting upstream of 148th Ave SE is the combination of canary reed grass and clumps of Willow trees encroaching in the channel just downstream of 148th Ave SE—significantly reducing channel capacity.

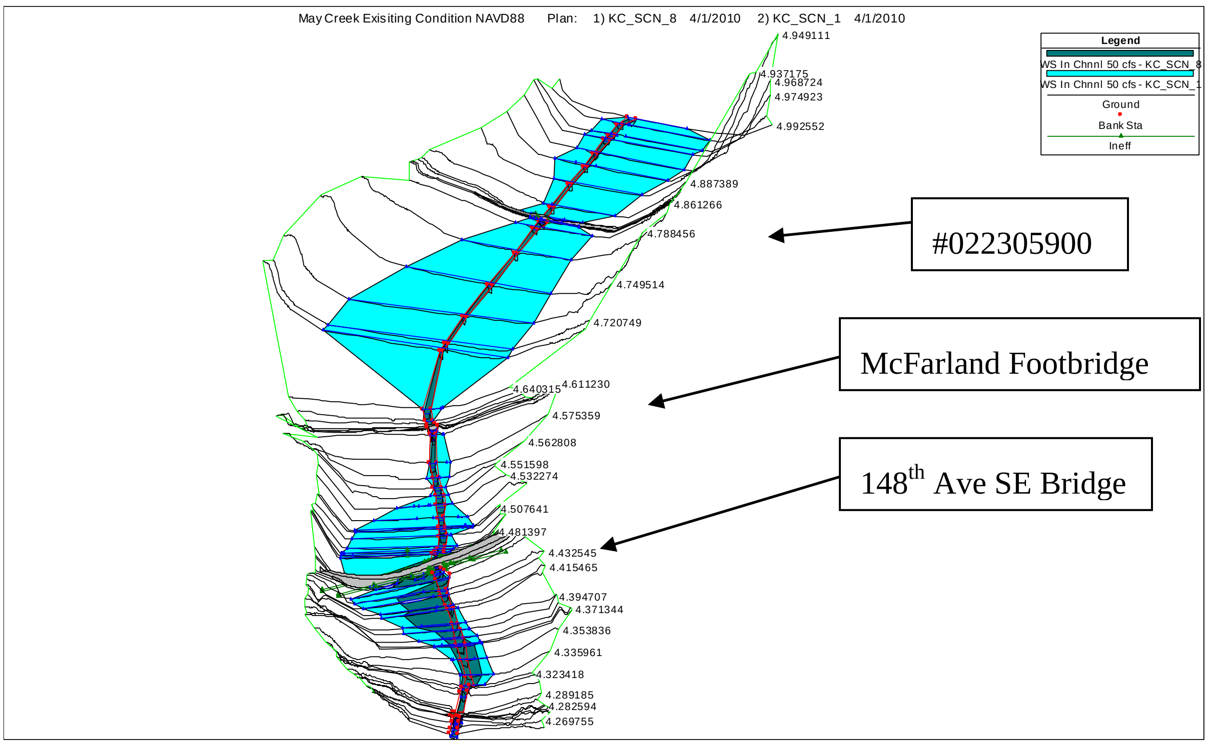

Figure 34 Perspective plot for Scenario 1 and 8 overbank flooding with 50 cfs at 148th Street. Light blue areas are inundated areas for Scenario 1 and dark blue are Scenario 8.

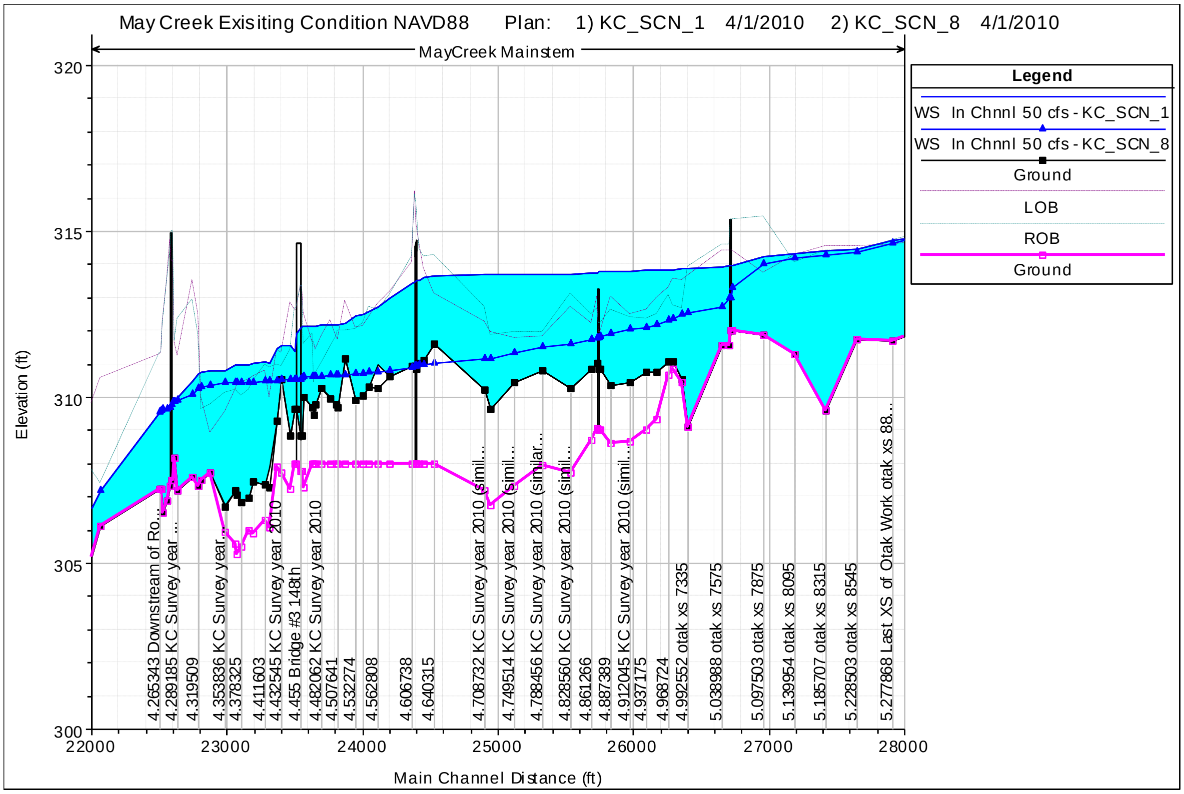

Figure 35 Water surface profile at 50 cfs for Scenario 1 and 8

In addition to lower flows, a review of higher flows was done. Focusing on downstream of the wetland through the bridge located at 143rd Ave SE, the same return period of flood frequency was used, but with their respective flow rates of 229 cfs (Scenario 1) and 240 cfs (Scenario 8) for the 2-year return period through the ravine. There is a marginal increase in water surface elevations downstream of 146th Ave SE of 0.08 ft and diminishes to 0.05 ft at 143rd SE (Figure 36). More importantly, the velocity changes are minimal as well with 0.10 ft/s at 146th Ave and 0.06 ft/s down at 143rd Ave SE. This marginal change in water surface elevations and velocities are essentially within the accuracy of the model validation. Thus, no effectively apparent significant changes between existing and proposed conditions at this flood frequency. Moreover, selecting any higher flow rates will result in even less differences between pre and post conditions.

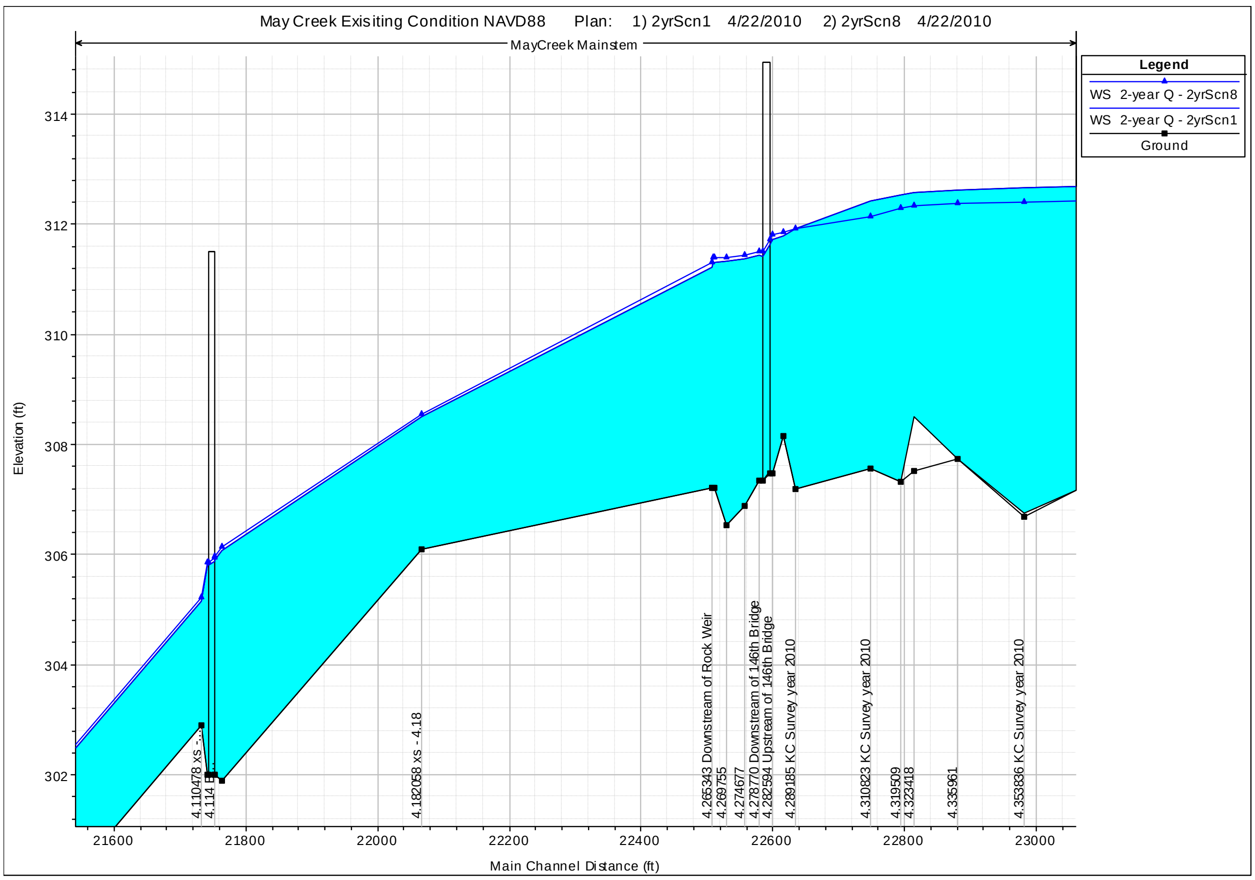

Figure 36 Water surface elevations for the 2 year return period (229 and 240 cfs) for Scenario 1 and 8.

Scenario 9

The alternative scenario of removing sediment and choking vegetation to an elevation of 309 feet (Scenario 9) is less effective at keeping flows in-channel at 50 cfs through the project area (Figure 37). However flows are estimated to stay within bank between 148th Ave SE and at the footbridge downstream of Long March creek confluence. However, this is primarily because of the element of removing vegetation choke points and the assumed flushing of existing silts with the increased velocities. Again, convergence of the water surface profiles converge at the natural control of wetland outlet (Figure 38). It is worth noting that flows are maintained in channel through the pasture areas up to approximately 40 cfs.

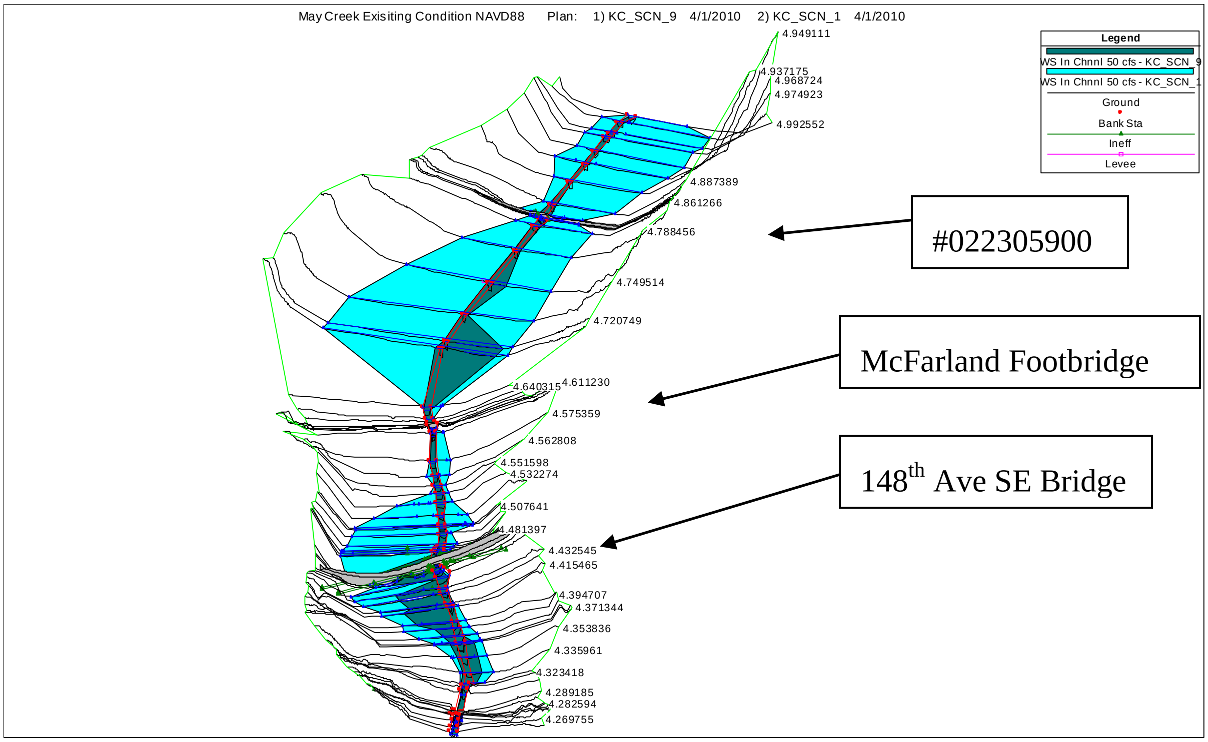

Figure 37 Perspective plot for Scenario 1 and 9 overbank flooding with 50 cfs at 148th Street. Dark blue areas are for Scenario 9, light blue are for Scenario 1 (existing conditions).

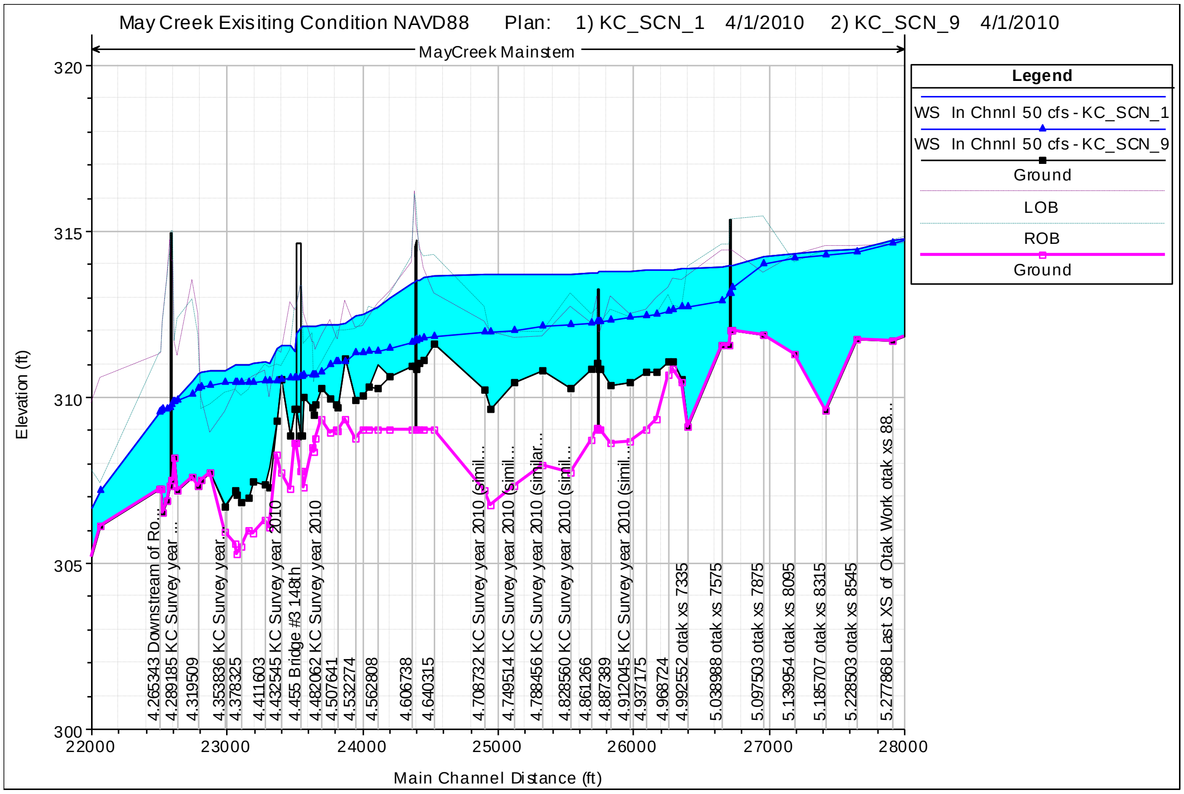

Figure 38 Water surface profile at 50 cfs for Scenario 1 and 9.

5.2 Updated Flow Frequencies

Original flood flow frequency estimates in the May Creek Current and Future Conditions Report were based on a 42 year period of record from water year 1949 through water year 1990, and a single scalar to translate SeaTac precipitation to May Valley conditions using Bulletin 17-B (USGS 1982). As previously mentioned, current hydrologic analyses include additional data through water year 2008 (18 years more data). This combined with the updated FTABLES for existing conditions, and flow frequencies have changed since the original analysis was done in the May Creek conditions report. Frequency analysis was done for two locations in the basin, flows draining through project area to 148th Ave SE bridge (Figure 39) and flows down to Coal Creek Parkway. This illustrates how estimates of flood frequencies are dependant on period of record in addition to magnitude of events and any small changes in frequency estimates should be viewed with that understanding.

Through the valley area, the magnitude of the 2 year flood frequency increases from 283 to 289 cfs (5 %). The increases in changes between Scenario 1 and Scenario 8 diminish to no change between scenarios at the 20 year event. In fact, flood events greater than the 20 year event decrease after the project. It is also worth noting that all the changes either increasing or decreasing are within the 95% confidence interval of existing conditions (Table 7). For convenience, the original May Creek Current and Future Conditions report flood frequencies are included in the table.

Table 7 Summary of flood frequencies for Scenario 1 and Scenario 8 for May Creek in the valley.

Return Period	Scenario 1			Scenario 8			Percent Difference	Conditions Report (1995)
	17-B	Upper	Lower	17-B	Upper	Lower		
1.01	53	64	41	61	73	48	16%	110*
1.11	94	108	80	104	118	89	11%	n/a
1.25	120	135	104	130	145	114	8%	n/a
1.67	163	182	145	173	191	155	6%	n/a
2	186	208	167	195	216	176	5%	165
5	283	326	252	289	329	259	2%	n/a
10	351	414	307	354	412	312	1%	285
20	417	503	359	416	496	362	0%	n/a
25	438	532	376	436	523	378	0%	n/a
40	483	595	410	478	581	411	-1%	n/a
50	504	625	426	498	609	426	-1%	413*
100	571	722	476	561	698	473	-2%	468

*Flows were interpolated and extrapolated from published 2, 10, and 100 year flow rates in the May Creek Current and Future Conditions Report.

Similarly for the ravine, these changes in flood events are nearly the same. The 2 year flood event increases from 229 cfs to 240 cfs (5%). Flood frequency magnitudes decrease starting around the 20 year event. Again, this estimated change is within the 95% confidence range of existing conditions (Table 8).

Table 8 Summary of flood frequencies for Scenario 1 and Scenario 8 for May Creek in the ravine.

Return Period	Scenario 1			Scenario 8			Percent Difference	Conditions Report (1995)
	17-B	Upper	Lower	17-B	Upper	Lower		
1.01	67	81	52	78	92	62	16%	141*
1.11	118	135	100	130	147	112	10%	n/a
1.25	149	167	130	161	179	142	8%	n/a
1.67	202	225	180	213	236	192	6%	n/a

Return Period	Scenario 1			Scenario 8			Percent Difference	Conditions Report (1995)
	17-B	Upper	Lower	17-B	Upper	Lower		
2	229	256	206	240	266	217	5%	208
5	349	400	310	355	404	319	2%	n/a
10	431	508	378	435	506	384	1%	357
20	513	618	442	512	609	446	0%	n/a
25	539	654	463	537	643	466	0%	n/a
40	594	731	506	590	716	507	-1%	n/a
50	621	769	526	615	751	526	-1%	514*
100	704	889	588	694	864	586	-1%	582

*Flows were interpolated and extrapolated from published 2, 10, and 100 year flow rates in the May Creek Current and Future Conditions Report.

Additionally with the proposed improved channel conveyance, attenuation of flows is marginally less thus slightly increasing flow frequencies on the lower end with the 1 year event increasing from 54 cfs to 61 cfs (Figure 40).

Flow frequencies in the ravine were similarly indifferent between existing (Figure 41) conditions and proposed (Figure 42).

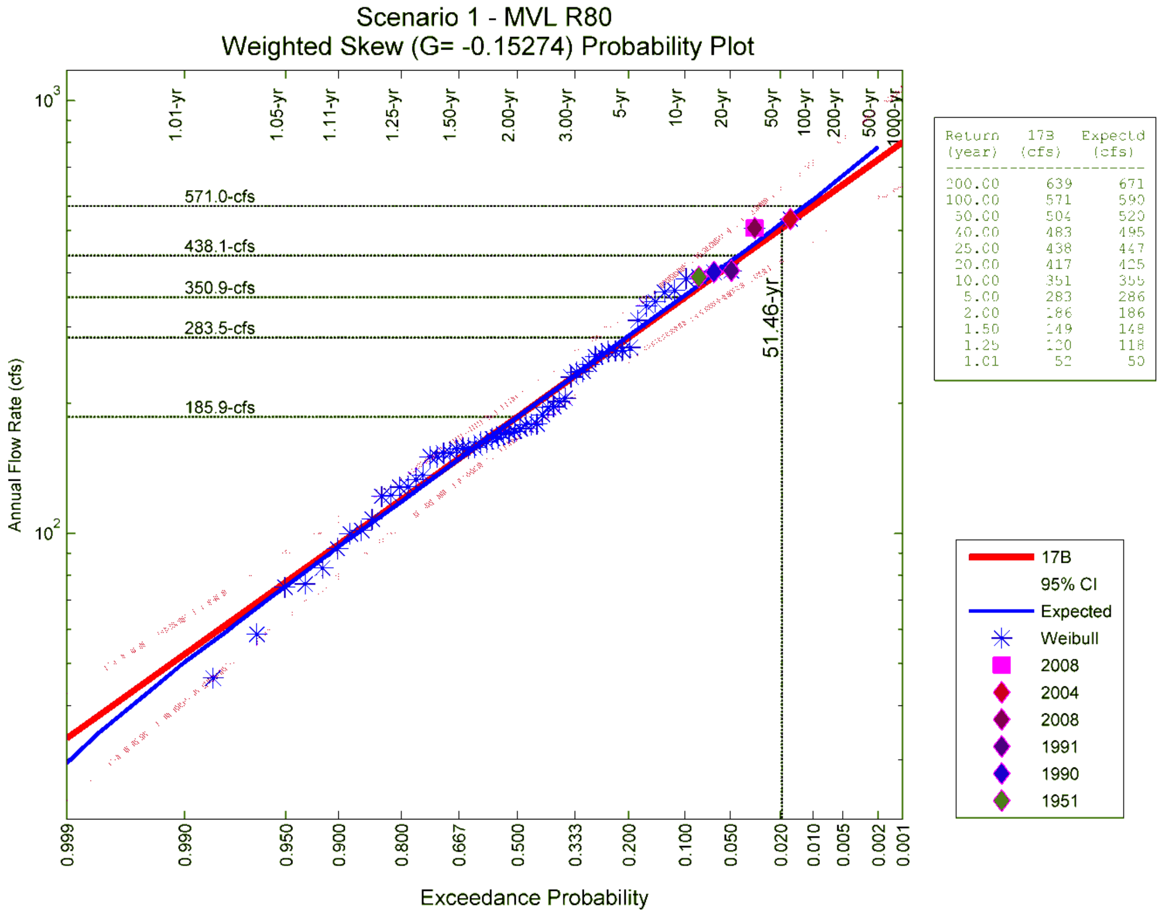

Figure 39 Flow Frequencies for existing conditions using USGS 17-B methodology for flows at 148th Avenue SE (catchment MVL)

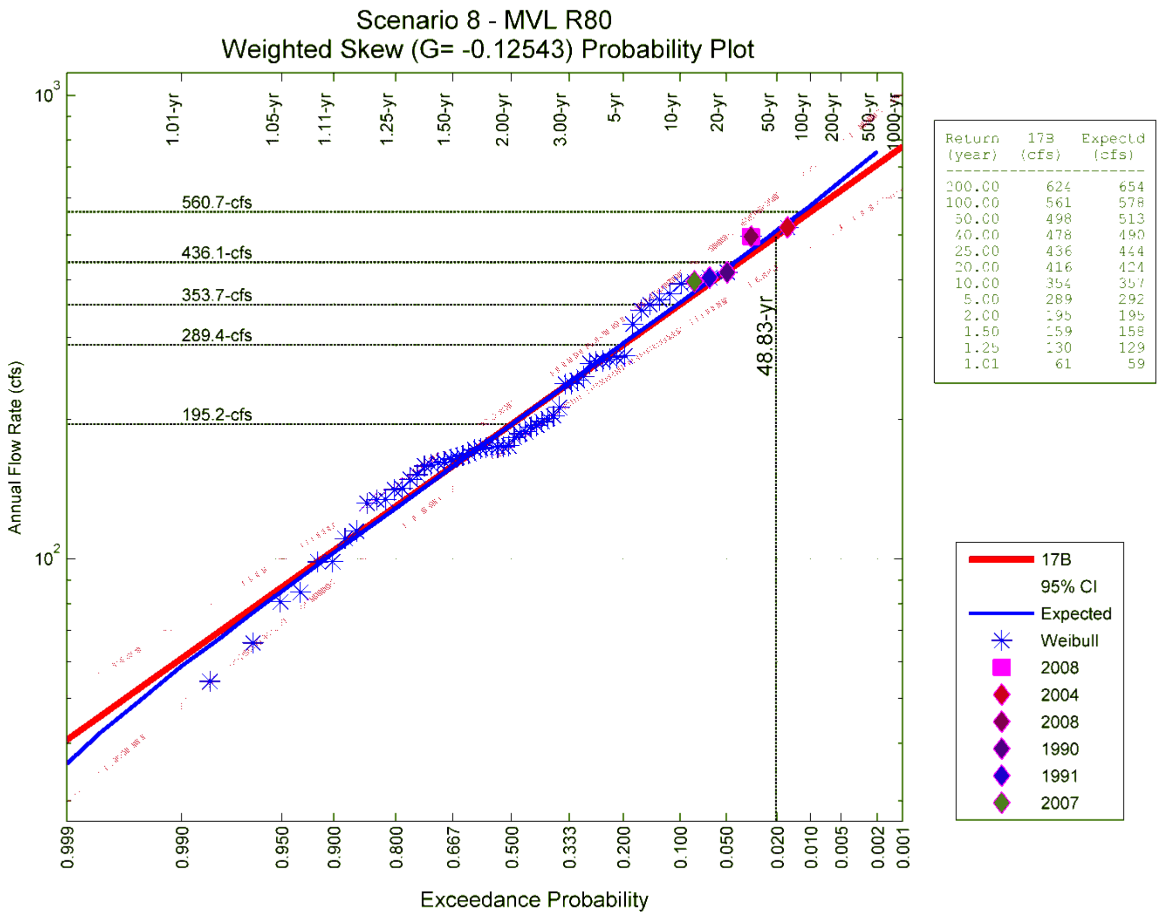

Figure 40 Flow Frequencies using USGS 17-B methodology for proposed project design (Scenario 8) for flows drainging to 148th Aveneue SE.

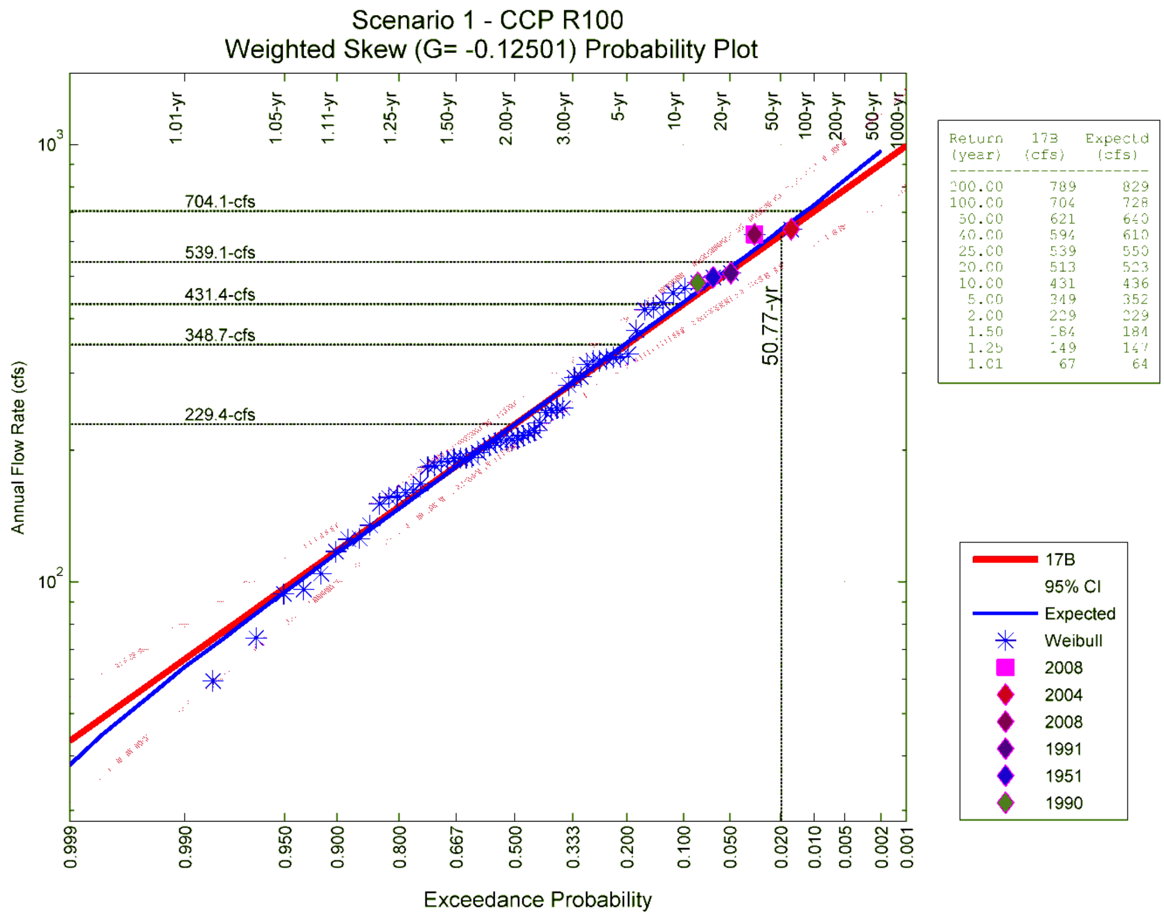

Figure 41 Flow Frequencies using USGS 17-B methodology for existing conditions at Coal Creek Parkway (catchment CCP)

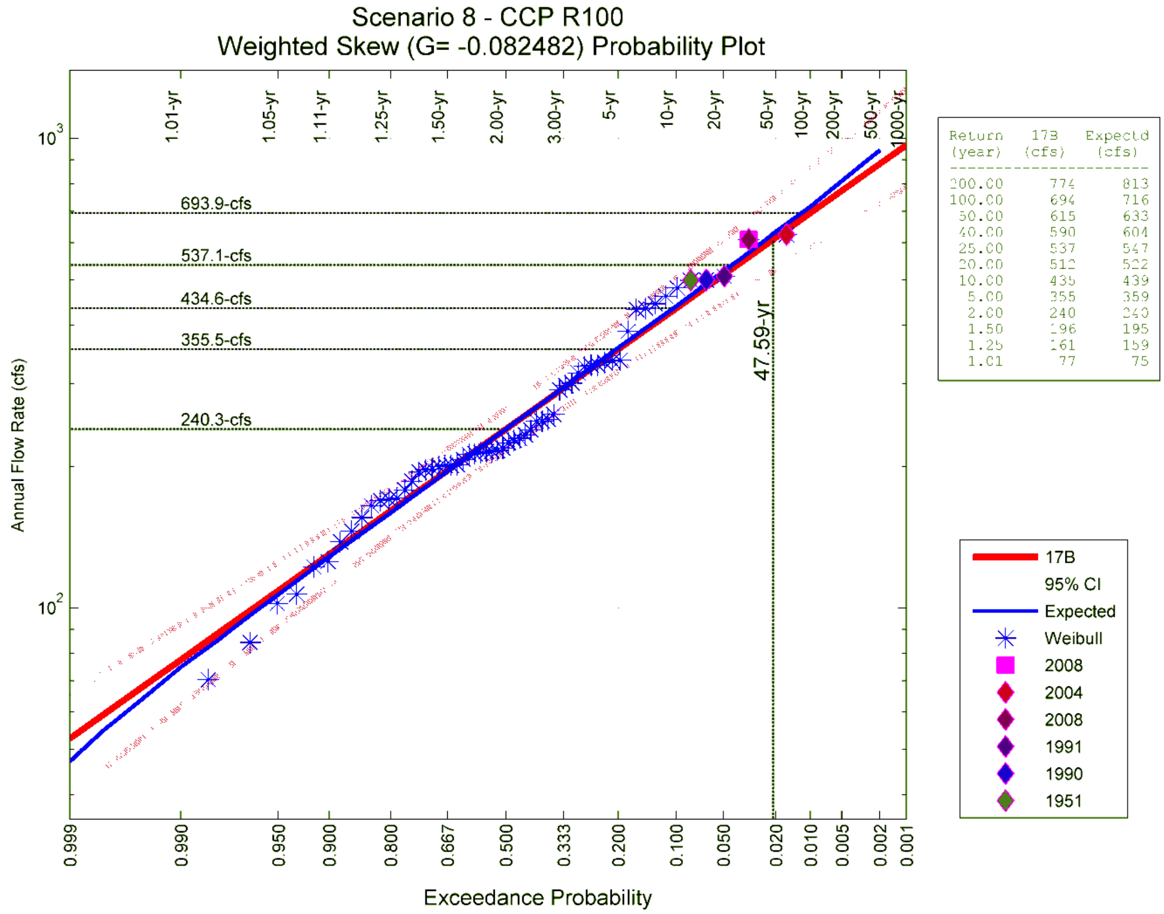

Figure 42 Flow frequencies using USGS 17-B methodology for proposed project design (Scenario 8) at Coal Creek Parkway (catchment CCP).

5.3 Reduced Duration of Flood Inundation for Smaller more Frequent Events

Reductions of durations in pasture flooding are quite substantive given the conveyance improvement from 6 cfs to 50 cfs. Using HSPF, a watershed model was run for a period of 60 years (water year 1949 through water year 2008) at one hour time steps for each scenario. Using the continuous output from the model, a durational analysis can be performed estimating the length of exceedances at any given threshold.

It is worth noting that these analyses are a simplification of actual conditions. These results do not take into account the time it takes for the flooded pastures to recede back into the channel after a storm event, thus actual reductions in durations of inundation will be less than presented here, but for the purposes of demonstrating improvements this method is valid.

Relevant for valley flooding, the two key thresholds of interest are 6 cfs and 50 cfs where flows are either out of bank or in-bank—post project. Under existing conditions, flows are forecasted to be over bank, on average, 24-percent of the time or roughly 3 months of the year. Which given the episodic nature of storms in the Pacific Northwest, and the unaccounted time for receding waters, this could be interpreted as multiple more months of inundation. Durations of flows flooding the pasture for the proposed design (Scenario 8), shows a significant decrease assuming flows are not out of bank until flows exceed 50 cfs. For this threshold, flooding is expected to occur, on average, 3 percent of the time per year or roughly 2 weeks (Table 9). Again, it is important to note that this does not account for time draining the pastures after an event has occurred, thus the effective reduction in pasture flooding will be less than presented here.

Reductions in duration of flooding are likely to occur when storm magnitudes begin to diminish in spring and summer with peak flow rates more likely to be less than 50 cfs. While the other two scenarios are included in the durational analysis and in the table below, the threshold for over topping banks is less than 50 cfs, hence reductions of durations will be considerably less with Scenario 7 (removal of vegetation choke points) the least effective, marginally increasing in-channel capacity to approximately 8 cfs before flows go out of bank. While Scenario 9 (removal of vegetation choke points, some sediment removal) does significantly improve conveyance capacity over existing conditions (i.e. approximately 40 cfs in channel capacity), over bank flows are estimated to occur 33 percent more of the time (i.e. 4 % of total duration versus 3 %) relative to Scenario 8.

Table 9 Percent of time flows are equaled or exceeded at each of the flow rate thresholds for flows passing through the project area (catchment MVL outlet at 148th Avenue SE) based on HSPF simulation WY 1949 – WY 2008.

Flow (cfs)	Scenario 1	Scenario 7	Scenario 8	Scenario 9
6	24.04%	23.77%	23.41%	23.49%
10.4	13.77%	13.65%	13.44%	13.50%
28	6.03%	6.01%	5.99%	5.99%
40	4.01%	4.02%	4.04%	4.04%
50	2.94%	2.96%	2.98%	2.98%
75	1.39%	1.42%	1.46%	1.45%
100	0.70%	0.74%	0.78%	0.77%
150	0.26%	0.26%	0.29%	0.28%
175	0.18%	0.18%	0.19%	0.19%
200	0.12%	0.12%	0.13%	0.12%
233	0.08%	0.08%	0.08%	0.08%
275	0.04%	0.04%	0.04%	0.04%
300	0.03%	0.03%	0.03%	0.03%

Flow (cfs)	Scenario 1	Scenario 7	Scenario 8	Scenario 9
350	0.02%	0.02%	0.02%	0.02%
400	0.01%	0.00%	0.01%	0.01%

5.4 Flow Rates Competent to Pass Silts through the System

Based on the estimated incipient motion for silts, shear stresses equal to or greater than 0.004 psf are capable to mobilize silty-fines assuming non-cohesive and median diameter size of 0.000328 feet (0.10 mm). Using HEC-RAS computations of shear stress, it is assumed that shear stresses greater than or equal to 0.01 are capable to mobilize silts. Two flow rate profiles were evaluated for their competency: mean annual flow rates and 29 cfs. The goal was to obtain shear stresses above 0.01 for the mean annual flow rate of 8.6 cfs throughout the project area. Results show that shear stresses do equal or exceed 0.01 through the project area except for in the wetland downstream of 148th Ave SE bridge (Table 10). The threshold of 29 cfs is meant to represent that typically, there is at least one event of that magnitude each month between November and May—with a few exceptions during dry months.

However, barely meeting the minimum target is not optimal given the possible inaccuracies in assuming non-cohesive silty fines. Fines with a diameter of 1 mm or greater require shear stresses above what the mean annual flow rate can produce. Moreover, during summer months when flows are their lowest, it would be expected that silts will settle on the channel bottom based on these results. This introduces another uncertainty about characterization of the fines when the next wet season starts. If vegetation has started to grow in the channel bottom, the root system will bind the soils together thus increasing the incipient motion threshold. Therefore, it is unclear what levels of flow rates are necessary to eradicate any new growths that may occur in the channel bottom during the summer.

These results are also dependant on the relocation of where Long Marsh creek deposits gravels during storm events. This tributary has clearly been shown in the past to deposit enough gravels to effectively backwater May Creek upstream of its confluence. Additionally, shear stresses in May Creek are far below forces necessary to mobilize gravels that are being deposited by Long Marsh creek.

Table 10 HEC-RAS calculated shear stress in channel for Scenario 8, mean annual equals 8.6 cfs. Zero shear stresses are highlighted in tan color

Station No.	Mean Annual	29 cfs	Station No.	Mean Annual	29 cfs	Station No.	Mean Annual	29 cfs
4.9926	0.01	0.03	4.6120	McFarland footbridge		4.4550	148th Ave SE Bridge	
4.9882	0.02	0.07	4.6112	0.01	0.03	4.4512	0.01	0.03
4.9749	0.1	0.25	4.6067	0.02	0.07	4.4454	0	0.01
4.9687	0.53	0.26	4.5754	0.01	0.05	4.4325	0.01	0.04
4.9491	0.04	0.09	4.5628	0.02	0.05	4.4265	0.01	0.06
4.9372	0.03	0.08	4.5516	0.01	0.04	4.4155	0.01	0.04
4.9120	0.01	0.05	4.5429	0.01	0.03	4.4116	0.01	0.04
4.8874	0.01	0.06	4.5323	0.01	0.02	4.3947	0	0.02
4.8701	0.29	0.32	4.5144	0.01	0.04	4.3876	0	0.01
4.8680	Parcel #0223059005 footbridge		4.5076	0.01	0.04	4.3783	0	0
4.8658	0.18	0.18	4.5055	0.01	0.03	4.3713	0	0.01
4.8613	0.42	0.12	4.4974	0.01	0.03	4.3694	0	0.01
4.8286	0.02	0.06	4.4821	0.01	0.04	4.3538	0	0.01
4.7885	0.01	0.04	4.4814	0.01	0.03	4.3360	0.04	0.07
4.7495	0.04	0.1	4.4788	0.01	0.03	4.3234	0.06	0.08
4.7207	0.02	0.07	4.4729	0.01	0.03	4.3195	0.05	0.08
4.7087	0.02	0.06	4.4648	0.01	0.03	4.3108	0.14	0.23
4.6403	0.01	0.02	4.4613	0.09	0.14	4.2892	0.04	0.07
4.6226	0.01	0.04				4.2861	0.31	0.21
4.6177	0.02	0.06				4.2826	0.06	0.09
4.6138	0.03	0.09						

5.5 Durations of Flow Rates in the Ravine

Using the results in the May Creek Phase II Sediment Transport Study, erosive flows in the ravine were observed to occur in the range of 73 to340 cfs, and a theoretical estimate of incipient motion approximate to 275 cfs. Refinements for the flow events used in that study, presented in this study, narrows the range of observed mobilization of sediment estimates for incipient motion to a range of 75 to 275 cfs (rounding to the nearest 5 cfs).

For this study, characterization of changes in erosion in the ravine are based on evaluating durations of flows above the defined incipient motion. As previously mentioned, an HSPF model was used to estimate continuous flows at one hour increments for a period of 60 years (from October 1, 1948 through September 30, 2008) using scaled historical precipitation and evapotranspiration. This method enables a statistical characterization using observed historical climate data for May Creek in the valley and ravine that would not be available otherwise. Thus, any phases of dry years, wet years, and everything in between are contained in this analysis as opposed to any potential climate bias using local data of shorter time spans.

Durations of flows evaluated in the range of 75 cfs or greater for the ravine show that there are small increases that progressively get smaller the higher the flow rate. Comparing Scenario 1 to Scenario 8, these increases start with 0.07 percent increase at 75 cfs and become less than 0.01 percent for flow rates at or above 275 cfs. As an example, there is 0.003 percent (i.e. 0.00003) increase in duration of flow rates between 300 and 350 cfs. Over 60 years of duration, that is equivalent to 15 hours increase or on average 15 minutes more per year (Table 11). This level of detail is beyond the accuracy of the calibrated model, but any biases in the models would be consistent such that relative comparisons between scenarios like this are valid.

Table 11 Percent of time flows are equaled or exceeded at each of the flow rate thresholds for flows in the ravine (catchment CCP) based on HSPF simulation WY 1949 – WY 2008 (525,960 hours).

Flow	Scenario 1	Scenario 7	Scenario 8	Scenario 9
6	43.26%	43.15%	42.97%	43.00%
10.4	25.06%	24.89%	24.63%	24.68%
28	8.84%	8.77%	8.66%	8.69%
40	6.10%	6.10%	6.06%	6.08%
50	4.60%	4.61%	4.61%	4.61%
75	2.41%	2.44%	2.48%	2.48%
100	1.32%	1.36%	1.40%	1.39%
150	0.47%	0.50%	0.52%	0.52%
175	0.33%	0.34%	0.36%	0.35%
200	0.23%	0.24%	0.26%	0.25%

Flow	Scenario 1	Scenario 7	Scenario 8	Scenario 9
233	0.15%	0.15%	0.16%	0.16%
275	0.10%	0.10%	0.10%	0.10%
300	0.072%	0.074%	0.075%	0.075%
350	0.041%	0.041%	0.040%	0.040%
400	0.027%	0.029%	0.028%	0.028%

5.6 Other Considerations

Presented in all the scenarios except for Scenario 1 (existing conditions), the channel bottom profiles were based on survey elevations taken to the harden channel bottom and not on top of the soft sediment. This assumption is based on the fact that after the project when velocities are improved, existing silts will flush out of the system over some period of time.

Although in-channel silt mobility analysis indicates improvements will restore competence to mobilize this material, there are in-channel features that will likely retard this process including re-vegetation by aquatic plants during summer base flow conditions, thus changing soil mobility characteristics and estimates of flushing for the next wet season.

Lastly, the modeling verification is still in progress. Low flow conditions in the 10 to 20 cfs range should be validated to a reasonable accuracy, but given how well the model simulated for slightly larger flow rates (i.e. > 40 cfs), it's reasonable to assume that the model represents conditions at lower flows acknowledging that there will be numerous specific locations in channel geometry and vegetation blockages not included at every location. However, it is intended to include all major features.

At present, the results of the modeling characterize out of bank flows in the 6 cfs range along with the durational analyses estimating that on average over time, pasture areas are flooded on average 3 months out of the year and likely longer. However given the preliminary hydraulic verification, existing channel capacity is uncertain at this time.

6.0. REFERENCES

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