U.S. Army Corps
of Engineers
Seattle District

H&H Appendix

Qwuloolt Restoration Project

Marysville, WA

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CENWS-EN-HH-HE

FINAL

1.0 – OVERVIEW

This appendix summarizes all work completed to date by the USACE Seattle District H&H PDT members and by the Project Sponsor AE team prior to USACE taking over the design of the levee portion of the project. Seattle District performed independent technical review of all AE products that informed the 10% and 35% design. This appendix provides background information, and then summarizes relevant technical studies, memorandums and reports that inform the 95% design package. Excerpts of important summary charts and figures are included in this appendix. The reader is referred to the provided Adobe PDF Portfolio "H&H References.pdf" for documentation compilation of all referenced H&H work products.

2.0 - BACKGROUND

2.1 Authorization:

The proposed project, located on former agricultural lands between RM 2.8 and 3.6 on Ebey Slough in Marysville WA, is authorized by Section 544 of the Water Resources Development Act of 2000 which authorizes implementation of critical restoration projects in the watersheds that drain directly into Puget Sound. The intent of the Section 544 Qwuloolt Restoration Project is to restore important hydrologic processes (both tidal and stream) and by doing so, associated structure and functions such as the establishment of intertidal habitat suitable for rearing and foraging by juvenile salmonids. This would be accomplished by lowering and breaching the existing dike at Ebey slough to reestablish tidal inundation to the site and reconnecting the site to the slough. There would be additional restoration work by providing an unobstructed opening to Allen Creek to allow for fish access

2.2 Setting:

The Qwulloolt restoration site is located in the lower Snohomish River basin, the second largest river tributary to the Puget Sound after the Skagit River (**Figure 1A**). The Snohomish begins at the confluence of the Snoqualmie and Skykomish rivers, which drain the foothills and headwaters of the Central Cascades. Due to the low gradient of the Snohomish, tidal effects extend more than 16 miles upstream of Possession Sound, nearly to the Snoqualmie River confluence.

Discharge in the low gradient tidally dominated estuary is bifurcated into four principal distributary channels including the mainstem Snohomish River, Steamboat Slough, Ebey Slough, and Union Slough. Ebey Slough connects Possession Sound with the mainstem Snohomish river 13 miles upstream of its confluence with Quilceda Creek. As the high tide propagates upvalley, it can cause flow in the Sloughs to reverse during periods of low runoff

(tidal flows occur in opposing directions). During higher flows, the freshwater rides atop a salt wedge that pushes up into the sloughs and adjoining marshes. Because Ebey Slough is longer than the mainstem, tides reach the confluence with the Snohomish before they can propagate all the way up Ebey Slough itself. This frequently causes stages and flows into the upstream end of Ebey Slough to increase, until the stage of the high tide is reached and tides begin to fall.

Ebey slough is also connected to Steamboat Slough at RM 5.5 and RM 6.75 (**Figure 1B, Figure 4**). Ebey Slough narrows by about 50% downstream of its confluence with Steamboat Slough, and Steamboat Slough thusly enlarges downstream of its confluence with Ebey. The project is situated about 3.5 miles up Ebey Slough, which is the longest and northernmost distributary channel of the Snohomish River delta.

The City of Everett lies at the South end of the delta, and the Tulalip reservation flanks the north end. The City of Marysville owns or has an interest in properties on all sides of the project, including the existing levee on the right bank of Ebey Slough (**Figure 1C**). City commercial properties are located in the FEMA 100-year floodplain behind an existing levee just east of the Marysville wastewater treatment plant (**Figure 1C**), however no residential properties are in the FEMA floodplain in this part of the valley. The Tulalip Tribe (project sponsor) has acquired all undeveloped property behind the levees within the FEMA 100-year floodplain (13 foot contour elevation).

2.3 Historic changes in delta impacting project area

Documentation of the magnitude and sources of change to the wetland resources of the rivers of Puget Sound, including the Snohomish are well documented by others (Collins 2000, Haas and Collins 2001, Collins and Sheik, 2005).

From Haas and Collins (2001):

> *Beginning in the mid-19th century, settlers cleared, drained, ditched and diked the Snohomish River valley. Snag boats removed wood obstructions to facilitate steamboat navigation. The cumulative impact of these actions has been dramatic losses and widespread degradation of salmonid habitat. Habitat loss and degradation, in turn, has reduced salmonid production capacity in the Snohomish River basin.*

Due to the lack of flood control dams on the upper forks of the Skykomish and Snoqualmie, residential and industrial uses of the floodplain are not widespread which has preserved the rural character of the valleys. Haas and Collins (2001) estimated that only 1/6th of the historic tidal marsh areas of the Snohomish remain. Extensive modification of floodplain wetlands and side channel habitat were also documented. In the last two decades

recognition of the links between these losses and the declining health of Puget Sound salmon have resulted in actions to restore areas of former tidal marsh and wetlands. Projects completed to date include Spencer Island and Union Slough near the Everett waste water treatment plant, about 3 miles south of this project. Dike removal is actively being pursued at locations where willing landowners have agreed to sell their properties for conservation purposes, such as at the Qwuloolt site.

At the project site (**Figure 1C**) a dike was constructed on the north bank of Ebey Slough and tidegates were installed at the mouth of Allen and Jones Creeks, and a series of ditches were dug to convert the marshland to pasture to keep the pasture dry. As a consequence, the dike and tidegates prevented tidal access to the historic floodplain which destroyed the estuary marsh habitat and restricted salmon and other estuarine-dependent species from utilizing this highly productive environment.

Due to decades of continuous over-draining, the land behind the existing levee on Ebey Slough has subsided by 2 feet below its estimated previous average marsh plain elevation (7 feet, based on adjacent undeveloped marsh plain elevations). This project specifically addresses improving the marine health of Puget Sound and provides critical habitat for several listed species under the Endangered Species Act.

3.0 - PURPOSE

The overall purpose of the Qwuloolt project is to restore the natural resources of the former estuarine marsh to as close to pre-settlement conditions as possible. To achieve this purpose requires restoration of historic hydrologic processes and functions such as tidal exchange, tidal channel formation and migration, deposition and marsh plain development and other associated functions. By restoring the historic tidal regime to the site the following project objectives will also be accomplished.

3.1 Goals:

The principal goal of the project and the overall restoration efforts are to restore tidal processes to 400 acres of currently fallow pasturelands. This will improve local streams and wetlands for fish such as threatened Chinook salmon, bull trout and steelhead. Fish will also be able to access the project area for refuge, feeding and spawning.

4.0 PROJECT CONSIDERATIONS AND CONSTRAINTS:

- The project must be acceptable to the Corps, the sponsor, and other project stakeholders.

- The project should be feasible for construction and will not negatively impact adjacent landowners (must be permitable, retains existing level of flood protection, is politically acceptable, physically possible to construct)
- The project is acceptable for environmental compliance

5.0 - Hydrologic investigations summary

5.1 Snohomish River

In the Puget Sound basin, the Snohomish River is the second largest watershed (after the Skagit River) by area, (~1,800 square miles at the mouth), draining the west-central Cascades from an elevation of 7,950 feet to sea level at Possession Sound, in Western Washington State (**Figure 1** and **Figure 2**). Basin average annual precipitation exceeds 90 inches, and more than 60% of the basin is covered by forest (**Table 1**). Mean annual discharge for 45 years of record at the Monroe stream gage at river mile 20 is 9,500 cfs.

Due to natural attenuation of the flood wave and division of the flows into the distributary channels the Snohomish River at Everett can reach 170,000 cfs during the 100 year event, according to the FEMA Flood Insurance Study for Snohomish County (FEMA 2005). However, the peak flow rate in Ebey Slough near the restoration site during this event is 19,000 cfs (**Table 2**)(FEMA UNET MODEL 2005). Similar trends are observed for all floods modeled in the FIS.

5.2 Tides

Tidal data for the site was collected by NOAA between 2000 and 2002 (NOAA Station 9447729). The tidal datums for Ebey Slough and Seattle are shown on **Figure 6**. The data was sufficient to estimate tidal datums and constituents for Ebey Slough. The data showed the high tides match Seattle well, lagged only by an hour. Low tides depend on flow conditions in Ebey Slough and run higher than Seattle by about 1.2 feet. Coastal storm events are not able to push wind driven waves up the sloughs, however tidal surge from low barometric pressure and wind setup can force water levels in the Lower Snohomish to rise to levels high enough to overtop levees.

5.3 Tidal Flux

Several efforts have been undertaken to evaluate potential changes in tidal flows following restoration. The modeling conducted to date shows that tidal discharge on Ebey Slough can exceed the daily freshwater flow by several fold. This is due to the flat slope of the channel and long distance (large tidal prism) subject to daily tidal inundation. Generally the studies indicated that as the former marsh areas are inundated they will cause more flow to enter the lower end of the distributary channel network, and may slightly reduce flow above the

restoration site or delay progression of the high tide slightly as the site fills (Battelle 2007, 2008, 2010, USACE 2002, USACE 2012D). On the outgoing tide the restoration site will add to the ebbing flows, increasing tidal flux below the site (PWA 2005) (Battelle 2007, 2008, 2010, USACE 2012D). Please see Geomorphologic Investigations Summary section for discussion of the implications of increased tidal flows for the restoration site and Ebey Slough. Figure 11 shows changes in tidal flux expected during average wintertime conditions, and during a major flood event. Restored marsh stage duration data from the USACE 2002 study are provided in Table 6.

5.4 Local Runoff

The Qwuloolt site includes the historic confluence of Allen and Jones Creek with Ebey Slough. Allen Creek drains 11 square miles of urbanized areas of Marysville. Previous work by others for Snohomish County was used without adjustment to estimate flows on Allen Creek that may enter the northwest corner of the project site (USACE 2002). The runoff of Jones Creek and from the residential areas along the site perimeter into the restoration are insignificant compared with the available area for ponding (400 acres) and were neglected in the evaluations. Runoff behind the proposed setback levee was estimated using the WWHM 3.0 continuous simulation hydrologic model. See Section 6.5 for detailed discussion of the interior flood H&H analysis and requisite design features of the proposed setback levee.

6.0 HYDRAULIC INVESTIGATIONS SUMMARY

6.1 Flood Sources

The Snohomish River channel capacity, floodplain area, and proximity to Puget Sound creates a condition at the project site where riverine flooding does not increase flood elevations greatly above the highest tide levels at this location (FEMA 2001, FEMA 2005), although any increase in flood levels can cause levees to fail or overtop more frequently. Without the existing dikes along Ebey Slough, the Marysville Industrial Park would be flooded daily during high tides.

Flood levels determined for this project originate from two modeling efforts. Flood levels exterior to the levees originate from the FEMA UNET hydraulic model prepared by WEST consultants in 2000 (**Figure 4A**), under contract to the Seattle District. The UNET model was updated to version 4.0 by Seattle District in 2011 (USACE 2011B) so that it could be run under Windows XP to perform a zero-rise analysis. Comparisons with the published FEMA FIS flood stage values were conducted to ensure the output was not affected by the model update.

In 2012, USACE obtained a georeferenced RAS-Unsteady version of the UNET model which includes the levees, but lacks bridges and storage areas. This model was used to evaluate daily tidal changes (USACE 2012D) while the UNET model was used to evaluate the potential flood impacts of the project on the floodplain, industrial park, and Ebey Slough.

In 2010 Battelle conducted modeling of the estuary under restored conditions using the 3D model FVCOM. Limitations in the modeling unfortunately prevented its use in this design effort.

USACE evaluated interior flood potential under existing and with project conditions behind the proposed setback levee within the Marysville Industrial Park (also referred to as Brashler's Industrial Park). Interior flood levels, under pre-project conditions, are driven primarily by the seasonal runoff on Allen or Jones Creek that ponds behind the tide gates (that drain the site) when Ebey Slough water levels are high enough to shut the tide gates and restrict drainage from the 400 acre site. During the winter the site becomes fully saturated and ponding is widespread. The ponded areas rise and fall with the tides (USACE 2007a). The street elevations in the Industrial park are only a few feet above the site average water level and can be flooded if high intensity rainfall occurs while the tide is high and the creeks are full. At present this occurs several times a year, and has been documented in all seasons, although is more frequent in the fall and winter (USACE 2007A).

Interior to the levee, within the Marysville Industrial Park, flood levels are controlled by the amount of local runoff during rainstorms, the available area and volume for ponding, and the magnitude and duration of outboard stages that can cause water to pond behind the levee.

Refer to the summary of USACE 2011b in Section 6.4 for more information on potential project impacts on FEMA floodplain flood elevations, and USACE 2011a, and USACE 2012 for more information on the interior flood hydrologic analysis. For discussions of the levee breach and tidal channel sizing see the Geomorphic Investigations summary in Section 7.0. See Sections 6.5 and 8.2 for summaries of interior drainage analysis.

6.2 Expected post-project average daily hydraulic conditions

Project (restoration site) water levels will be controlled by daily tidal fluctuations, the Snohomish River, and by local runoff. Breaching of the levee will immediately convert the site back to a tidal, brackish marsh and mudflat. Constructed and remnant tidal channels will flood and ebb twice daily. Due to the large marsh area it is expected that water will fill and drain through a large 1st order primary tidal channel constructed through the levee near RM 3.5 that will join with remnant and constructed tidal channels in the interior of the site. The breach channel dimensions are sized using empirical hydraulic geometry

relationships (PWA 1995) to ensure full inundation of the site up to the high tide level. The setback levee will prevent flooding of low lying portions of the Marysville Industrial park during daily high tides. Channel dimensions for interior channels were sized by the Sponsor. The desired end state is normative inundation frequency and access for fish into the site interior, Allen, and Jones Creek, and a restored native vegetation communities and marsh plain elevation.

Physically, the hydrology of the tidal channel network is governed by the principal tidal channel area, tidal channel network density, tidal prism (volume stored between marsh plain and mean higher high water elevation), and tidal constituents (tidal phase, amplitude), sediment and vegetation characteristics, and storm effects (PWA 2008a, PWA 1995, USACE 1991). Tidal prism data were computed from available topographic survey data (PWA 2008a). Tidal channel discharge can be estimated numerically with hydrodynamic models. Please refer to Section 7.1 below for detailed summaries of analyses performed to estimate tidal channel flows and constructed tidal channel dimensions.

Tidal datums are shown in Figure 6. Expected elevation duration relationships for seasonal conditions following restoration are shown in Table 6. Contours of the restored tidal range (MHHW, MLLW, 50% exceedance) within the site are shown in Figure 13. Generally due to the subsidence, water will be out of the tidal channel banks more frequently than it is the banks (Figure 13). This regime will remain in place until the marsh plain elevation is reestablished. Due to the large, deep breach channel, and Tribal efforts to construct 3rd and 2nd order channels within the marsh site, full reestablishment of tidal hydrology is expected.

The impacts of tidal reintroduction will extend at most 2000 feet into Jenning's park, measured along the centerline of Allen Creek (PWA 2008C). The average ground elevation is greater than the MHHW contour (9.2 ft) in the majority of Jennings park (Figure 13). Thus water heights will range from those seasonally experienced at low tides, to the high tide elevation, the average increasing only slightly. Increased flows due to tidal flux reintroduction are only occurring a short distance into the park and are not problematic. Adjacent private properties will not be affected because the impacts of the project are restricted to the wetlands within Jennings park.

6.3 Pre and post-project flood risk

It is a design requirement for this project that the setback levee match or exceed performance of the existing levee along Ebey Slough to ensure that landowners are not worse off with respect to flooding after completion of the restoration project. At present the existing levee just contains the FEMA estimated 10 year flood event. Because the levee changes the hydraulic regime in the site, the levee is being designed to account for factors that cause water levels to increase above static levels.

To design the setback levee to modern standards, the team accounted for expected future sea level rise, settlement, and wave runup in determination of the final levee crest height (Section 8.1, USACE 2012C, PWA2008). This level of protection exceeds that shown in the Snohomish County comprehensive flood control plan (5-year + 1 foot, Snohomish County 1991). Because an allowance is being made for future conditions that are expected to worsen, this will initially reduce the risk of overtopping for the constructed portion when compared with existing conditions. A small section of remnant levee is lower than the proposed setback levee and will continue to overtop at the same frequency, baring sea level rise. However due to the reduced total length of the levee that can overtop following completion of the setback levee and exclusion of a lower more frequently overtopped section of the breached levee upstream, flood levels are predicted to be lower for all flood events analyzed with the exception of the 500-year event. (USACE 2012H, 2012A, 2011A).

Existing conditions flood flows on Allen Creek are several times the expected increase due to the restoration of tidal hydrology. Hydrology in Allen Creek upstream of the tidal influence is not affected by this project, thus, flood flows will not be problematically increased (Reference PWA 2008C, USACE 2002) on Allen Creek due to levee removal. If the culvert at 3rd street is replaced with a bridge, the extent of tidal influence during a tidal surge event on Ebey Slough may extend 2000 feet along Allen Creek upstream of 3rd St into Jennings's park, however the flood levels will be lower than those currently experienced (PWA 2008C) because the 3rd street culvert is undersized with respect to potential flood flows on Allen Creek. Existing flood heights exceed daily high tides by several feet. Flooding is not worsened. Adjacent properties will not be affected because the impacts of the project are restricted to the wetlands within Jennings park. Velocities due to the increased tidal flux will not be erosive.

6.4 FEMA Zero Rise Analysis

Washington State Department of Ecology, on behalf of the City of Marysville, requested that USACE perform a FEMA zero-rise analysis to establish the effects of breaching and setting back a levee on the FEMA floodway and floodplain base flood elevations (USACE 2011B). Just under half of the setback levee will be located in the mapped floodway. Snohomish County Code requires that a zero-rise analysis be performed if a floodplain project will place fill in areas mapped as floodway along the Snohomish River. It was determined that the existing conditions FEMA model should be slightly modified to reflect the hydraulic effects of an existing floodplain dike protecting the City of Marysville Wastewater Treatment Plant (WWTP) that was not included in the original model. Inclusion of the existing dike in a modified existing conditions model resulted in small changes in the

100-year (1% annual exceedance probability AEP) flood event in the vicinity of Marysville. Within Marysville flood levels are decreased but just upstream of the WWTP within the Qwuloolt project site, water levels were slightly increased. Inclusion of the proposed with-restoration project conditions into the modified existing conditions model by increasing the height of an ineffective area for one cross section to reflect the new levee did not result in further base flood elevation changes, with the exception of a 0.1 foot increase immediately upstream of the WWTP, within the restoration site, at one model cross section (2.87). The increase is 0.5 feet lower than the computed allowable flood level increase due to the future filling of the FEMA floodway fringe. Since the project does not change the width of the floodway fringe at the locations of the hydraulic model cross sections, there is no increase in the floodway flood levels that can be computed for this project without including more cross section data, however if floodway fringe areas within the restoration site are converted to floodway after restoration, this could result in decreases in computed floodway elevations.

FEMA and Washington State Dept of Ecology concluded that this computed rise, resulting from an ecosystem restoration project, was allowable, and the City of Marysville concurred. FEMA Region X accepts a 0.1 t rise in the floodway for ecosystem restoration projects.

6.5 Interior Drainage and Flood Risk

The performance of the drainage system in the City of Marysville Industrial Park (also called Brashler's Industrial Park) serving properties behind the existing Ebey Slough dike was documented by USACE in 2007 (USACE 2007a). The analysis indicated that site water levels are high due to inadequate discharge capacity of the existing tide gates, which causes water levels to gradually increase during wet periods. This causes a high ground water table within the industrial park and nuisance ponding and street flooding near catch basins is a routine occurrence. The source of frequent flooding is primarily offsite runoff from Allen and Jones Creek, combined with local rainfall runoff. Maximum flood levels will be reached if and when the existing or new setback levees are overtopped. The entire site is in the 100-year floodplain, and this is not expected to change, as the new levee ties into a levee lower than the100-year event at the south end.

Interior flood risk is defined as the risk of flooding behind a levee due to local sources of runoff, combined with the effects of major floods on the riverward side of the levee. Fortunately due to the small basin size behind the proposed levee (**Figure** 3, Figure 4, Figure 5) project flood levels can be controlled through construction of appropriately sized conveyance ditches, swales, and ponds to store local runoff during high tide or river flood events, and an outfall system to discharge water back to Ebey Slough on the outgoing tide. The frequency and magnitude of flooding behind the levee is expected to lessen following

completion of the setback levee and upgraded storm drainage system. Highly reliable backflow prevention gates will be necessary at the stormwater outfall. Clogging of the outfall will force water into the existing basin draining Allen Creek. The existing 60" culvert has more than ample capacity to drain the maximum expected runoff. At this time HE Section recommends against reliance on the existing outfall due to concerns over future maintenance of this feature. A new detention pond and outfall structure will be constructed through the new levee. Please see detailed summary information on the interior flood analysis in Section 8.4.

6.6 Residual Risks

Residual Risks are the risks of flooding a protected area that remain following completion of a flood risk management project. Residual risks associated with construction of the setback levee are not worsened due to the fact that the existing levee (located adjacent to the setback levee) is not negatively impacted. This section of levee has the lowest functional crest elevation of the levee system providing protection for the industrial park. Some levee overtopping is expected to continue at the low spot on the Marysville/Diking District 3 levee adjacent to the setback levee every 10 years on average post project, matching existing conditions.

Because a low section of levee upstream of the levee breach will be part of the restored site, USACE has determined through use of the FEMA FIS 1-D UNET model that the duration and magnitude of levee overtopping during floods that exceed the 10-year event will actually be reduced, which reduces flood heights within the industrial park during major floods (USACE 2012H, 2012A, 2011A).

The proposed levee tie in with the existing levee will include strengthening of the backside and top of the existing levee with properly sized rock to allow overtopping to occur without breaching. This will maintain existing flood frequency and in severe floods will reduce hydrostatic pressure acting on the setback levee. This reduces the risk of a sudden breach and increases time for evacuations in the event of a major flood. No inhabited properties are located behind the levee. If sea level rise accelerates, and the overtopping frequency increases, the City may elect to independently raise its segment of the levee to maintain overtopping frequency.

Because this analysis neglects flood fighting conducted by USACE, Marysville, and Diking District 3 as part of the PL84-99 Program, flood risks will likely be reduced further due to the larger length of higher, modern levee available to protect the industrial park.

Residual risks can be increased if the levee owner fails to perform required O&M activities. Failure to perform O&M could compromise drainage structures or levee integrity.

Development of floodprone areas can increase residual risks but putting structures in harms way and reducing areas for floodwater storage. USACE recommends reserving undeveloped low lying areas behind the levee for ponding of stormwater to allow for a buffer against future changes that could increase flooding, such as higher intensity rainstorms, and sea level rise.

Residual risks associated with potential failure of the adjacent City of Marysville waste water treatment plant dike are unknown but are not expected to be changed by this project. The treatment plant dike is lower than the FEMA base flood level and the plant will be flooded at the same time and to approximately the same height as the protected area during the 1% AEP event. If the dikes were to suddenly rupture, 18 acres reserved for stormwater storage would be filled first. Four culverts would be available to drain the outflow to Ebey Slough and the restoration site (See Section 8.5).

7.0 -GEOMORPHIC INVESTIGATIONS SUMMARY

The Qwuloolt restoration project is located at the former Portinga dairy farm property. Much of the area around and within the Qwuloolt project footprint (**Figure 1C**), including the Allen Creek floodplain upstream of 3rd Street, and the City of Marysville Wastewater Treatment plant were historically a transition point between a large freshwater forested tidal wetland complex and tidal marsh (Collins 2000, Haas and Collins 2001, Collins and Sheik 2005). The minimal changes in planform and active width of all major distributary and tidal channels documented since the first aerial photos of the valley were taken (1938, **Figure 10 A**) indicate that either the impacts from diking were not significant on channel morphology, or that the river adjusted quickly to the changes by the time the first photos were taken and has remained stable due to the low valley gradient, and the presence of vegetation, streambank armoring and dikes, among other factors.

7.1 Restoration Tidal Channel Network Evolution

Reintroduction of tides into the restoration site will first and foremost restore daily tidal fluctuations that will closely match those of Ebey Slough in the 1st order primary breach channel and 2nd order channels. Two daily high tides and low tides will be experienced within the marsh. Water levels shown in Figure 6 and Table 6 reflect conditions expected within the restored site. The expected tidal range was converted to contours which are overlaid with lidar data for the site in Figure 13.

The most dramatic adjustments in the constructed tidal channel network are expected near the breach channel, where tidal flux is greatest. For this reason, the confluence was relocated to a pool section where increased energy can be more readily dissipated . Erosion of Ebey Slough at this location due to confluence scour is expected but will likely persist a short distance downstream (to the next crossing). Due to the relocation of the confluence of the 1st order channel with Ebey Slough about 1 mile upstream, flows near the existing tide gates will not be restored to their historic conditions. A short section of the lower reach of Allen Creek will convey less tidal flow (and energy) than it did historically because the junction with the Jones creek tidal channel has been moved upstream to the new breach location. The new confluence location is perpendicular to a pool on the outside of a meander bend, which is similar to its historic location near the Allen Creek tide gates. The new location is expected to be more energetic due to its location further up Ebey Slough, and presence of modest super-elevation along the bend. The Tulalip Tribe and NOAA have a detailed monitoring plan to evaluate geomorphic and biological responses to restoration over time.

Due to the increased tidal forcing resulting from a subsided marsh plain elevation, there will likely be more water present (greater depths and durations) in the channel network and in the marsh than before diking (PWA 1995, 2008A). Because the tidal marsh emergent vegetation line averages around 5.2 feet in elevation (**Figure 13**, **Table 6),** large areas of the site below this elevation may initially revert to mudflat, which may allow for an increase in the density of the tidal channel network (relative to an unsubsided site) due to the relative ease of 1st order channel formation in muds. Modeling conducted by USACE in 2002 indicates that 50% of the time water will be near this elevation, which is slightly higher than the mean tide level, due to ponding in subsided areas (Table 6). Die off of non-native vegetation within the site near this elevation is possible due to the reintroduction of brackish water during high tides. Loss of vegetation may increase the rate of natural tidal channel formation and migration through the unconsolidated fine grained sediment deposits that form in quiescent areas during floods. Assuming that loss of vegetation may increase the potential for channel migration initially, USACE has included armoring of the levee in areas where it is potentially exposed to erosive conditions.

The orientation of the setback levee with respect to the location of the principal tidal channel will likely draw turbid flood water from Ebey Slough, create a large quiescent areas at the southwest corner of the site, which may locally enhance sedimentation rates and storage of woody debris. At this time it is not clear how dynamic the restored 1st and 2nd order (smallest and second smallest) tidal channels will be (PWA 1995). It is assumed that if the initial dimensions of the primary breach channel are sufficient to convey flows into and out of the marsh as the tides rise and fall on Ebey Slough with little if any lag, that the

channel dimensions will be relatively static. USACE has considered the possibility of initially increased rates of migration due to vegetation die off and has included scour protection for the levee adjacent to Allen Creek where needed (Section 8.4).

While levee lowering along Ebey Slough is planned, significant portions of Marysville owned levee will not be altered. This will ensure that a City wetland mitigation project and recreational opportunities along Ebey Slough are not impacted by the project. The setback levee, industrial park, and remnant levees preclude full restoration for the site.

The Tulalip Tribe and their consultants (ESA/PWA) have also designed the 2nd, 3rd, and 4th order tidal channels that will connect to the USACE constructed breach channel. These will be constructed prior to dike breaching to maximize tidal exchange throughout the marsh.

Modeling conducted to evaluate scour potential along the levee (Section 8.4) and to size the breach channel (Section 8.3) indicates that average daily (non flood) peak flows in Allen Creek (downstream end of Jennings Park to 3rd street) will be modestly higher than under existing conditions, due to the reintroduction of tides to the site which will subsequently ebb on the outgoing tide, adding to the flow in the channel (See Figure 13). The peak flow in the tidal cycle occurs when the high tide reverses and the creek is mostly backwatered. In this point of the tidal cycle velocities are quite low. Despite increased daily flow, velocities are not expected to increase problematically. Small 1st order tidal channels will likely establish through the reed canary grass wetland complex on the margins of Allen Creek above and below 3rd street as the grass gradually dies off and converts to brackish tolerant marsh.

See the next section for discussions of potential flood elevation changes within the site due to restoration. The modeling performed to size scour protection and design the breach accounts for this condition.

7.2 Impacts of increased tidal flux on Ebey Slough

A major assumption that has been carried forward since the project was conceptualized in the 1990s is that since the dimensions of Ebey Slough were formed under conditions where no marsh areas were diked, total daily tidal prism in the distributary channels and marsh network was larger than at present. From this assumption it follows that restoring the diked marsh area is not likely to have adverse impacts on geomorphologic regime of Ebey Slough.

Studies subsequently performed by the Tulalip Tribe during development of the 10% design elicited concerns that the increased daily tidal flux may increase scour and erosion potential in Ebey Slough. This concern originated from a preliminary analysis of the primary tidal channel dimensions that relied upon empirical regression relationships for tidal marshes

(PWA 2008a). It is our understanding that these equations were developed for sizing breach dimensions for purely tidally forced conditions; however the concern over channel enlargement due to increased tidal prism overlooks the fact that the greatest discharge in Ebey Slough occurs during fluvial floods, under conditions that are significantly more channelized (due to levees) than under predevelopment conditions.

To address the channel stability concern, analyses were subsequently conducted using a 3D hydrodynamic model (Battelle 2008) of average and moderately high flow conditions. Modeling indicated that peak flux below the project is likely to increase (Battelle 2007, 2008). This was confirmed in subsequent modeling performed as part of this effort (USACE 2011B, 2012D). It is logical that smaller flows on the Snohomish that don't suppress tides will allow for increased tidal flux into and out of the lower reach of Ebey Slough due to the subsided condition of the large area to be restored. In 2010 3-D hydrodynamic modeling was performed for a large flood event, approximating 100-year conditions; however no baseline without project modeling was performed which prevents comparisons of bed stress changes with that model (Battelle 2010).

To better understand the potential difference in bed stress between the conditions pre and post restoration, USACE modified the levee elevations and added a storage area to the FEMA UNET model and ran all the FEMA flood events with a several weeks of typical tidal boundary conditions (USACE 2011B). Inspection of output of the 1-D hydrodynamic model under a variety of tidal and fluvial flood conditions confirms that the greatest depths and velocities (and by inference, channel shear stresses and erosion potential) are experienced as flood discharge increases (**Figure 11 and Figure 12**). This indicates that even though daily tidal flux may increase bed stresses, these new peak stresses are routinely exceeded under current conditions during floods. An analysis of conditions after restoration during floods suggests that there is a potential for very small increases in discharge after restoration, as the water draining from the restoration site adds slightly to the peak on the recessional limb of the fluvial hydrograph (**Figure 11**, USACE 2011B).

Because levees line the banks of nearly all the distributary channels in the lower estuary, water surfaces are artificially raised between the levees during floods, which increases the bed stress and scour potential over un-diked conditions (FEMA 2005, USACE 2011B). This effect would likely persist for any flood event where water is artificially impounded. When the levees and dikes were first constructed, beginning in the 1880s, there was likely a period of generalized and localized channel deepening and/or widening which was then followed by additional armoring as required to stabilize the banks. Due to the widespread presence of armoring and levees it is assumed that current conditions during major flood events are more erosive then those prevailed under pre-settlement conditions. Comparison

of the earliest (1938) aerial photograph series with the 2009 series indicates that the lower estuary has experienced little observable planform change (widening, narrowing, migration, avulsion) (**Figure 10**). This suggests that the channel network had already adjusted to the presence of the levees prior to the 1938 photo, or that the channel network morphology was not sensitive to diking, having been shaped by a larger pre-levee tidal prism and floods much greater than those in the period of record.

As stated previously, hydrodynamic modeling indicates that daily tidal flux will increase. This may alter the average widths and depths in the channels where armor or vegetation is lacking, however the available modeling preformed for very large floods indicates that flood heights and velocities at the peak of the floods will not increase. Since scour is directly related to discharge, scour potential is not expected to increase for the largest floods. When daily tidal flux increases (**Figure 11**), peak ebb and flood velocities will increase, but still remain significantly below those experienced during floods. The range of daily peak velocities remains generally lower than handbook erosion thresholds for the cohesive silts and marsh vegetation that line banks.

In summary, the stresses on the channel bed and banks experienced routinely during floods significantly exceed the expected daily with-project bed stresses resulting from increased tidal flux. Because peak stresses are not expected to change during floods, adverse geomorphic change resulting from restoration is not expected. Please see the summary of investigations performed for the design of the restoration project below for further information about the geomorphic analyses and investigations performed for this project.

Even though the weight of available evidence strongly suggests that restoration will not pose significant off-site risks, a detailed assessment of the potential for offsite impacts and a characterization of the erodibility of Ebey Slough is being conducted during the 100% design phase. This is a City of Marysville permit requirement for the Sponsor. See Section 10 for details of this investigation.

8.0 - SUMMARY H&H ANALYSES PERFORMED for 95% DESIGN INCLUDING RECOMMENDATIONS for PDT

Between the 65% and 95% design phases, several investigations were conducted to fine-tune the design for purposes of economizing or clarifying features of the work. During 65% the expectation of the PDT was that the project was going to be constructed with equipment rental contracts. The switch to the invitation for bid method required effort to refine the analyses and design details of the stormwater conveyance and detention system, erosion and scour protection, and levee breach dimensions. The following summarize the memorandums completed in this phase of the design.

8.1 Final Levee Height

In an effort to reduce costs the PDT closely scrutinized the factors governing the levee height. After closer investigation and team discussion the design levee height was lowered to 14.5 feet from 15.0 feet, accounting for a lower expected long term settlement and reduced wave runup estimate. During the evaluation the levee height was finalized (USACE 2012C) as follows:

Initial Crest height (after pre-loading) = 14.5 feet

This combines the following factors:

11.7 feet: surveyed low levee crest elevation of adjacent City of Marysville dike allowing floodwater into the protected area. This is the basis for the new setback levee height. The team could not rationalize providing a higher standard than this as it was viewed as a betterment. This crest elevation is 0.1 ft higher than the published FEMA 10-year flood level (FEMA 2005).

1.5 feet: Expected wave runup along levee (USACE 2012B) – this is included because levee removal and breaching cause a changed hydraulic condition along the new levee. Levee overtopping due to wave action was unacceptable to the team and sponsor.

0.9 feet: Expected median long term sea level rise at Seattle (EC-1165-2-212). This is included to adhere to current USACE policy.

0.25 feet Expected long term settlement (pers. communication Ghada Elithy, USACE Geotechnical Engineer). This is accounted for based on a team discussion regarding constructability and construction tolerances.

0.15 feet: Allowance for roundup to nearest half foot (pers. communication Ghada Elithy, USACE Geotechnical Engineer). This is accounted for based on a team discussion regarding constructability and construction tolerances.

Final Crest height (after long term settlement) = 14.25 feet.

Please refer to section 9.2 for background information on the previous iterations of the levee crest elevation.

8.2 Storm Water Pond and Outfall Sizing Updates

REFERENCES: USACE 2012E, USACE 2012H

In the 65% designs the stormwater pond was designed to maintain a water elevation below 5 ft NAVD88 for the 25-year event with a safety factor of 1.1. Recognizing that this is a significant improvement over existing conditions (**Figure 7**, USACE 2007A), the system was reanalyzed and modified to keep flood elevations below levels where buildings begin to

flood (6.5 ft NAVD88), which is less than the existing peak flood stage (6.93 ft) measured by USACE in the industrial park at the corner of 47th Ave NW and 56th Pl NE in Jan 2008 (USACE 2007A). This change in design maximum ponding elevation from elevation 5.0 ft to 6.5 ft allows a reduction in excavated pond volume while increasing the safety factor. Increasing the design water level will also increase the driving head which will aid in draining the pond.

The same methods were used in sizing the 95% design pond as were used for previous iterations (See sections 9.4, 9.5). Output from the Western Washington Hydrologic Model (WWHM3) was combined with an unsteady HEC-RAS model of the site that took into account variable tides, seepage through the levee, and coincident timing of high tide and runoff inflow to the detention pond.

Per previous investigations, flood levels for with project conditions in the industrial park are significantly lower than under existing conditions, despite the significant pond volume reduction. Please refer to **Figure 8** and USACE 2011H for comparisons of existing and with project flood levels for the 1 through 500 year flood events.

Because the low lying areas around the industrial park are frequently flooded, and located in the floodplain, they are undeveloped at present. The analysis included existing conditions and conditions where all areas outside the constructed pond are filled with and without fill around the constructed pond were conducted to account for potential future conditions. The final pond was designed assuming all areas around the pond become filled. It was found that reducing the pond volume by 65% to an excavated volume of 10,800 cubic yards below the current ground elevation resulted in a safety factor of 1.3 for flooding above elevation 6.5 during the design 25-year storm.

During the 100-year event, the safety factor against water levels exceeding 6.5 feet is 1.1.The water quality cells are required to be 25% to 35% of the total pond volume. Though the pond was designed with all overflow areas filled it is not recommended due to uncertainties in future hydrology, potential for existing levee overtopping, and uncertainties of the WWTP dike stability.

The city of Marysville conducted a backwater analysis showing that flood level would be higher in the industrial park during peak runoff events because of head losses through the drainage system (See **Figure 4** and **Figure 5**). There backwater analysis elevations were added to pond stages during the time of peak inflow in the HEC-RAS model simulation for the same flood events. Water surface elevations exceeded previously calculated peak pond stages for the 50-year and 100-year events by 0.18 ft and 0.49 ft respectively, though results still showed a decrease in flooding over existing conditions.

The outfall culvert crossing was moved to station 24+60 so that it exits into the remnant portion of Allen Creek. This was done to minimize uncertainties about erosion and channel migration issues from having the outfall directly connected to Allen Creek as in the 65% designs.

8.3 Levee Breach Update

REFERENCES: USACE 2012D, USACE 2012E

The levee breach design was evaluated to see if other breach geometries could provide adequate performance (fully restored tidal hydrology) and potentially decrease excavation volumes and costs. Two HEC-RAS 1-D hydrodynamic models were created of the Qwulloolt restoration site to evaluate the levee breach size. One used the modified (with restoration project) FEMA UNET model output at the new proposed breach location as a downstream boundary to evaluate the effects of various breach sizes during tidally affected 100-year flood conditions. The other model combined with an HEC-RAS model of the Snohomish River Delta generated by WEST Consultants to evaluate effects within the breach and Ebey Slough for tides and within-levee flow conditions.

Five different breach cross section designs were evaluated in the models for velocities, erodibility potential, and stage lag times to fill and drain the restoration site. These breach configurations included the design from the PWA memo (bottom width = 50ft, invert = -12ft), the 65% design (bottom width = 50ft, invert = -9 ft), a U-shaped breach (bottom width = 10ft, invert = -9 ft), a raised U-breach (Bottom width = 10ft, Invert = -7 ft), and a wide and shallow design (bottom width = 100 ft, invert = -5 ft). Please see Figure 14 for comparisons of the expected velocities and velocity duration in the primary breach channel for the different breach channels considered.

The differences in effects between the designs are relatively small in Ebey slough, though conclusions were able to be drawn to allow for selection of a recommended cross section. The u-shaped breaches slightly reduced velocities in Ebey slough, but increased velocities in the breach channel due to smaller breach area. This implies that the potential for local scour is much higher for the u-shaped breaches, than for the PWA, 65%, and wide breach alternatives. The PWA design had the slowest velocities through the breach, but the effect of mean velocity in Ebey Slough is slightly higher, and this design requires the most excavation. The wide and shallow breach also slightly increases the mean velocities in Ebey slough, and is expected to incise which could cause location, migration, and orientation complications. The 65% design was determined to be the recommended alternative for final design. The potential for erosion at the breach location and local scour in Ebey Slough is considerably less than with the smaller volume breach alternatives. The 65% design also has

a slightly smaller effect on cross section averaged velocities in Ebey Slough than either the PWA design or the wide and shallow breach.

The final levee breach location was qualitatively evaluated to determine the most stable location. The location has been moved about 300 ft upstream from that shown for 65% design. A deep linear ditch is located along the same alignment and runs north-south between new confluence of Jones and Allen creek and a ditch that runs behind the Ebey Slough levee. At the proposed location of the breach Ebey Slough forms a deeper pool. This configuration of a tidal channel approximately normal to the slough, at a pool section, is supported by several similar configurations at other nearby tidal channels in Ebey Slough. The deeper pool also will allow for redistribution of any high momentum flows from the levee breach to reduce downstream impacts. The new alignment follows an existing drainage ditch that may somewhat reduce excavation costs. Lastly, the channel intersects Ebey Slough at a slight downstream-oriented angle because of the indications from modeling that flows are downstream from the reconnected marsh into Ebey Slough for a majority of the tidal cycle.

Typical post-project tidal datum contours are shown in Figure 13.

8.4 Levee Erosion and Scour Protection Update

REFERENCES: USACE 2012B, USACE 2012E

The need for erosion control through armoring and vegetation arises from the potential for waves to break and runup on the levee face or site perimeter slopes, eroding the underlying material. USACE developed levee rock armoring designs assuming that the wind driven waves propagate unobstructed across the inundated site, and designs for rock armoring that factor in the presence of randomly spaced vegetated hummocks and berms that will obstruct the waves, limiting their heights when they reach the shoreline. The 2012 wind wave analysis (USACE 2012B) confirmed previous design wave heights of 1.5 ft with a period of 2.5 seconds (USACE 2007B). Based on the frequency of exposure to wind waves it was determined that the riverside face of the levee below elevation 9.0 NAVD88 (approximately MHHW) should be armored with Class A or Class B rock (equivalent of large quarry spalls or small riprap). Subsequent refinements reduced the rock blanket thickness to 15”, which includes a 3” filter layer. Because the riverside face of the levee above elevation 9.0 is exposed to wind waves less frequently and can be sufficiently protected with either a turf reinforcement mat or a 1 ft thick layer of quarry spalls. The landward side of the levee should be protected with a mat of native grass.

Erosion control also pertains to limiting local erosion from excess turbulence, velocity, or shear stress that would otherwise result in scouring of the channel or banks. Erosion within and along tidal channels in the interior of the site is expected and a desired outcome of restoration, and will be encouraged in the design by constructing over-steepened pilot channels that will then scour and enlarge over time as the marsh channel network is re-established.

The need for scour protection along the new levee has been significantly reduced from the 65% design because of a realignment of Allen Creek in the 95% design. No scour protection is needed where there is adequate distance between the levee and creek, though inspection and maintenance will be required if future channel migration threatens levee. Inclusion of large wood erosion protection along the outside of tidal channel meanders would increase fish habitat and reduce risk of tidal channel migration and is recommended. Between stations 5+40 to 39+00 no buried riprap toe is required due to sufficient offset (roughly 100 feet of separation between the bank and toe of levee).

Where Allen Creek is within the offset of the levee, scour depths were determined using hydraulic data output from a 1-D HEC-RAS model of Allen Creek, and multiple channel scour equations including: Modified Lacy, Modified Blench, and USBR. Bend scour was also looked at using equations from USACE, Maynard, and Thorne. It was concluded that between stations 0+20 to 5+40 scour protection was needed to an elevation of -2 ft NAVD88. Riprap sizes were calculated using output from the HEC-RAS model with SAMwin version 1.0 hydraulic design package. The gradation was determined to be a Dmax = 2.5 ft, D90 = 1.77 ft, D30 = 1.22 ft, D50 = 1.46 ft. This was then converted to an equivalent WSDOT gradation. An allowance was made in the scour analysis for channel incision of the upper reach of Allen creek down to an elevation that matches the rest of the marsh, and enlargement, to account for the increased channel stresses resulting from a lowered average water level and increased daily tidal flux.

8.5 WWTP Dike Failure Investigation

REFERENCES: USACE 2012C, USACE 2012F

It was recognized that despite provision of a robust setback levee, the industrial park will have other flood risks that remain. These are termed residual risks. The southwest part of the area protected by the new levee is also adjacent to the Marysville Waste Water Treatment Plant (WWTP) dike on the west side of the industrial park. This dike is used to impound and treat several acres of sewage. Typical water surfaces in the WWTP are 4 ft higher than first floor elevations in the industrial park. A “what-if” analysis was conducted

to determine how water elevations within the industrial park could be affected if the WWTP dike were to breach with and without the Qwuloolt setback levee in place to ensure the design won't increase water elevations to an unsafe level.

The assumed source of the breach is an earthquake or other foundation failure that causes the lagoon dike to rupture. No attempt was made to quantity the probability of the event or the relative stability along the dike as this lies outside the scope of the restoration project. The levee breach routines in HEC-RAS were used to evaluate potential flood levels following a sudden "sunny day" breach.

In the event of sudden breach of the north lagoon, flooding would fill the low lying areas of the site reserved for stormwater ponding initially, then inundate the lowest lying business by about 2 feet (with project), which is about 2 feet more than without project since the ponding area is much larger. About half of businesses may not experience flooding. No life safety risk would be posed by this failure because the lagoon is located adjacent to land that is owned by the Tribe and will not be developed. Additional hydraulic capacity (modeled as 4 additional 3 ft culverts) does not significantly reduce peak flood levels. High velocities at breach may erode and breach the setback levee from the backside which would lower peak water levels. Until the stability of the dikes is evaluated the probability of flooding associated with dike failure is unknown.

8.6 Drainage Ditch Design

REFERENCES: USACE 2012I

Switching to the invitation for bid contracting method eliminated the responsibility of developing a Temporary Erosion and Sediment Control (TESC) plan, which was included in the 65% plans. The TESC plan included a ditch between the stormwater pond and the Ebey Slough tide gate. It was recommended that this feature remain as part of the final project to function as a temporary drainage pathway to the remaining Ebey Slough tide gate during construction. The ditch is to remain a permanent facility, and to function as an emergency overflow during flood events, or as a backup drainage system if the primary system needs to be taken offline. To minimize earthwork the ditch will be cut relatively shallow and have a riprap outlet at the end connecting the ditch to the existing wetland in front of the Ebey Slough tide gate. The ditch was sized in HEC-RAS to contain most expected flows within the banks during non-flood conditions and should remain stable if vegetated.

8.7 South Tie in Levee Overtopping and Armoring Needs Analysis

REFERENCES: USACE 2012G

The need to evaluate the south tie-in levee overtopping stems from the concerns over scour risk adjacent to the City of Marysville Wastewater Treatment Plant (WWTP). Due to high expected turbulence, the PDT purposely moved the new confluence with Ebey Slough upstream from its historic location, where the existing tide gates are now located. Because the levees will remain between the tie in with the new setback levee and the breach, water will enter the site at a higher elevation than will occur at the historic confluence, and will allow water to overflow from the backside of the levee during very large flood events. Concerns were raised that erosion from overtopping could lead to breaching of the levee at this location allowing uncontrolled tidal exchange adjacent the Marysville WWTP. Three unsteady HEC-RAS models were created to envelope model assumptions and calculate flow, velocities, and height of water overtopping the levee. Four separate riprap sizing equations used the HEC-RAS model output to calculate the needed rock size on top of the existing levee to prevent erosion. It was determined that crushed rock with a d50 of 1" (2" maximum) would prevent erosion if overtopped. The first 400 feet of existing levee east of the tie in should be armored to prevent erosion at this location.

8.8 Potential Site Water Levels During Construction

REFERENCES: USACE 2012J

This memorandum provides data and information to apprise construction staff of potential water levels and flooding during construction. Information developed during the design phase was summarized and combined with links to websites that provide precipitation, water level, and flood forecasts and observations near the work site. If provided in the solicitation package the information will help ensure bidders better understand likely site conditions. Contractors will be able to use the data and forecasts to assist with planning their operations.

9.0 – SUMMARY OF SUPPORTING PREVIOUS H&H STUDIES

The 95% project design is the end result of several iterations of hydrologic, hydraulic, geomorphic, and geotechnical investigations and analyses by USACE and the Sponsor, as well as others. The analyses performed demonstrate that the project as designed is feasible, that is, it can restore full tidal hydrology without increasing flooding for adjacent properties. This

Appendix is intended to roll up the various studies into one stand alone document. It is not intended to be exhaustive, and the reader is required to review the referenced documents for more detailed information.

The project design is a combination of five principal factors – each requiring investigation and analysis. The principal design factors are dike breach configuration, site flood levels, setback levee configuration, erosion control, and interior drainage.

The principal investigations that have driven the hydraulic and hydrologic design are described below. Generally the investigations have been additive - that is, the site features have been refined over time as new information becomes available. Discussions of superseded efforts are provided below as background for understanding the design iterations.

9.1 Dike breach configuration

REFERENCES: USACE 2002, PWA 2008A

(10% analysis - superseded by 95% design)

Two methods were used to estimate breach size to fully restore tidal hydrology to the entire site. A 1-D hydrodynamic model was used to evaluate the ability of breaches of various widths to restore tidal inundation, and to estimate the resulting impacts on flood levels. This analysis is discussed in detail in the PAS study (USACE 2002). The PAS study determined that a breach on the order of 400 feet would allow for near full restoration of tidal hydrology. The Project Sponsor conducted additional analyses, based on geomorphic principals of hydraulic geometry for tidal channels with respect to tidal prism (PWA 2008A), and estimated that a channel on the order of 260 feet wide at MHHW would be needed to fully restore the tidal hydrology (**Table 3**). Due to concerns over the potential for channel erosion at the confluence with Ebey Slough, the PDT decided to move the location for the levee breach and tie-in with Allen Creek upstream to the center of the site. This minimizes the chance that scouring flows would undermine adjacent infrastructure (City of Marysville WWTP). The current design enters Ebey slough opposite open marsh on Ebey Island, about 0.4 miles upstream from the former confluence with Allen Creek and Marysville WWTP.

9.2 Design Flood level and Setback Levee Configuration

REFERENCES: USACE 2012A, USACE 2011A, USACE 2009

(10% - 65% analysis, superseded by 95% design)

The site cannot be restored without first constructing a setback levee along the western perimeter of the site, which parallels the City of Marysville WWTP levee. To design the levee, a design height is needed, which is tied to the current level of flood protection

provided by the levee. For this project, the design flood is defined as the flood event which first overtops the existing dike into the site. The design flood elevation has been associated with an average annual Snohomish River flood exceedance frequency of 10-years (10% annual exceedance probability). This flood event could exceed the dike crest elevation at a low spot on the southwest corner of the site (USACE 2011A). This portion of the Snohomish County Diking District is designed to contain the 10-year flood event with freeboard. The restoration project must demonstrate that the overtopping frequency is not increased with the set back levee, to ensure that flood risk is not increased for adjacent properties.

The PDT has recognized that combinations of tides, riverine flows, and wind events will contribute to the actual total water level once the dike is breached (PWA 2008B). A coincident stage frequency analysis was then performed to establish the with-project 10-year return period flood level that includes the combined tidal and riverine still water flood elevation with a wind generated wave runup elevation, and an allowance for sea level rise (reference). The with-project flood elevation appears to have increased, when compared with the flood level that first overtops the low spot on the dike (reference). This increase is due to inclusion of expected sea level rise and an allowance for coincident wind generated wave action, however, these processes would be just as likely to affect the site without the project if the existing dike breaches in the future during the 10-year event. The levee only needs to provide protection for the low lying properties in the City of Marysville industrial park. The City of Marysville treatment plant perimeter dike is higher than the existing dike that will be removed as part of this project. The perimeter private properties are elevated above the FEMA Base Flood Level, which is not impacted by this project. Isolated private parcels that would require a ring levee to protect have been purchased by the Project Sponsor.

Estimation of the levee crest elevation was based on two sets of hydraulic data (coastal and fluvial). The first method starts with the crest elevation equal to the low point on the existing levee (11.7 ft NAVD) plus accommodation for wave runup (2 ft) and the predicted sea level rise by 2050 (0.5 ft), yielding a levee crest elevation of 14.2 ft NAVD without freeboard. The USACE suggested the hydraulic criteria is estimated by adding the 10-yr water level (estimated to be 11.6 ft NAVD), 2 ft freeboard, and 0.5 ft of sea level rise, yielding 14.1 ft NAVD levee crest elevation. A larger freeboard is often used if wave runup is ignored, with typical freeboard values between 2 and 3 feet.

The design levee crest elevation may be a higher elevation due to round-up, geotechnical factors or other factors. A lower levee crest elevation could be justified using a vegetated bench to decrease the wave runup component directly and earth berms to limit wave runup by limiting the wave climate. The final levee top elevation (15 feet), designed to contain the

10-year flood event with freeboard, exceeds the site 100-year flood still water level by at least 1 foot.

As the existing levee and new setback levee have a pre-defined design flood event, and will not provide certifiable protection against the 1% AEP event, a risk based analysis is not required to design the levee. It is recommended that the project be designed to allow for future levee elevation should sea level rise rates exceed forecasts.

9.3 Erosion Control

REFERENCES: USACE 2007B, USACE 2011B

(65% analysis, superseded by 95% design)

The need for erosion control through armoring arises from the potential for waves to runup on the levee face or site perimeter slopes, eroding the underlying material. USACE developed levee rock armoring designs assuming that the wind driven waves propagate unobstructed across the inundated site, and designs for rock armoring that factor in the presence of randomly spaced vegetated hummocks and berms that will obstruct the waves, limiting their heights when they reach the shoreline. Based on the results of the final wind wave analysis the following are recommended for wind wave protection:

Erosion control also pertains to limiting local erosion from excess turbulence, velocity, or shear stress that would otherwise result in scouring of the channel or banks. Erosion within and along tidal channels in the interior of the site is expected and a desired outcome of restoration, and will be encouraged in the design by constructing over-steepened pilot channels that will then scour and enlarge over time as the marsh channel network is re-established.

Scour that undermines the prism of the levee should be prevented. Given the stability of the tidal channels in the estuary, migration is not expected. Because no formal analysis of channel migration within the restored marsh will be conducted as part of this project, engineering judgment has been used to include a riprap buried toe where the levee is adjacent to the restored tidal channels. An interim recommendation for scour protection is as follows:

> Place and bury a 3 ft thick lift of Class 3 riprap (18 inch minus) from the existing ground surface at the levee toe (or stability berm) down to

elevation – 4 at a slope of 1.5 to 1 between levee stations 2+00 and 5+50, 10+10 to 12+60, 21+50 to 24+30, and 27+50 to 30+30. Between levee station 34+00 to 39+40 place toe down to elevation -2.0. This riprap should extend up to elevation 9.0 at the south end of the site, and 10.0 at the north end, unless some other methods for controlling wave erosion are included (anchored woody materials). Above this elevation vegetation is adequate to armor the levee face. A turf reinforced mat should be used to protect the levee face while the vegetation establishes.

Local scour and erosion is expected in Ebey Slough at the confluence of the primary tidal channel due to confluence scour. The exact configuration of the main tidal channel will evolve over time. The location of the levee breach was selected to move the location of greatest expected turbulence, bed stress, and resulting channel change nearly a mile upstream from the Marysville treatment plant so that the flow field downstream will not be significantly affected.

9.4 Interior drainage

REFERENCES: USACE 2012A, USACE 2011A
(65% analysis, superseded by 95% design)

Interior flood levels must not increase as a result of the project, as this would flood several businesses that routinely experience flood problems. The local sensitivity to flooding is high, and any change in flood conditions is likely to be attributed to the project. The setback levee design includes a large open space between the City WWTP dike and the new setback levee. This area is the current location for Allen Creek. Allen Creek is being restored into a historic alignment by the Sponsor, which allows for use of the former channel and floodplain as a detention area. Based on the interior runoff analysis, the gravity drainage system will be able to detain and discharge the 25-year rainstorm event on the outgoing tide (Stormwater Management Manual for Western Washington SMMWW design standard for stormwater facilities) without flooding the Industrial Park. This is a significant improvement over current conditions, and allows for improved water quality treatment, which will reduce the amount of pollutants entering the site.

The Western Washington Hydrology Model (WWHM3) was chosen as a continuous runoff model to estimate runoff from the industrial park and local drainage areas into the

stormwater detention pond. The storm water detention pond was designed to capture the 25 year 24 hour storm below elevation 5 ft NAVD88 and release it on the outgoing tide. The 25 year storm was selected from WWHM3 data to match the 25-year return flow rate. WWHM3 gave a 6-month 24-hour water quality design volume of 1.96 acre-ft. To create a hydrograph for the 6-month storm the 25-year flood event was scaled to match the 6-month 24-hour volume and run through an unsteady HEC-RAS model of the site that took into account variable tides, seepage through the levee, and coincident timing of high tide and runoff inflow to the detention pond. To size the pond the detention pond was modeled as a storage area and the stage volume curve was edited until the water quality storm could be contained below elevation 4 ft NAVD88. It was found that the volume necessary to maintain a pond elevation under 5 ft NAVD88 during the 25 year event was 14.2 acre-ft, and the volume necessary to maintain a water quality elevation of 4 ft NAVD88 during the 6-month event was 4.7 acre-ft (**Table 4**). The designed water quality pond size and the WWHM3 calculated size are different because of the effects of seepage through the levee, dead storage within the detention pond, and some partial filling in the model before the 6-month 24-hour volume arrived. The outlet works orifice and weir sizing were calculated in accordance to the SMMWW, and it was found that 4 orifices 5 inches in diameter were needed for water quality treatment, while a total weir length of 3.25 feet was needed to pass the 25-year 24-hour storm event at elevation 4 ft NAVD. Table 3 summarizes storm water pond hydraulic data and Table 4 summarizes the design requirements for storm water detention. The design safety factor for the flood detention volume is 1.1. The outfall capacity is also greater than necessary, to allow for redundancy (shut off of one of the outfalls for maintenance or if there is failure). Thus some further optimization of the design for cost considerations can be achieved if necessary.

To compare before and after flood levels in the Industrial park an analysis of potential flood levels under existing and with project conditions due to levee breaching or overtopping without breaching was conducted. Overflows into the site during existing conditions occurred only at low spot on the levee about 1 mile upstream, peaked at 1,250 cfs for the 100-year flood and increases the stage in the existing storage area by about 1.5 ft (USACE 2012C). On the contrary, with-project conditions removes this upstream low spot from its hydraulic connection to the industrial park, reducing the amount of levee overtopping that occurs into the industrial park and potentially reducing flood stages during the 100-year event.

9.5 Marysville Industrial Park Stormwater Improvements for the Qwuloolt Tidal Restoration Project

(USACE 2011A)

(35% analysis, superseded by 65- 95% design)

The purpose of this study was to ensure flood elevations presently experienced by property owners behind the proposed levee are not increase, and support the design of the industrial park storm water facility. A continuous runoff model (WWHM3) was used to determine runoff behind the proposed levee, while an unsteady HEC-RAS model was developed to model water elevations within the industrial park storm water facility. Tides and Snohomish River flood hydrographs were taken from the Snohomish River UNET model and used as a downstream boundary in the storm water HEC-RAS model. A Hypothetical Event Analysis (HEA) approach was used to combine river floods with storm water pond inflows, assuming they are independent, to compute a combined probability of storm water pond stages.

It was concluded from this study that the 35% designed storm water pond will reduce flood levels for all conditions analyzed. The pond could also be reduced in size while still reducing flood levels. Flooding due to overtopping and breaching of the existing levee to remain along Ebey slough is likely when water elevations reach 12 feet NAVD88.

9.6 Snohomish Estuary CFD Modeling Study

(Battelle 2007, 2008, 2010)

The report documents development and modeling of the Snohomish estuary with the FVCOM 3D finite volume, unstructured grid CFD (computational fluid dynamics) model. Tides, salinity, river flows, and wind are forcing mechanisms. The model was calibrated to one week of observed tides, winds, and low flow conditions at multiple locations to multiple parameters (current, stage, salinity). The model has not been verified by comparing the results to observed conditions during other time periods, but the calibration appears robust. The analysis indicates that dike breaches will increase tidal flows downstream of restoration sites, but reduce them upstream. Salinity levels will increase due to increased tidal prism. The report presents plots of depth averaged 2D velocity fields, and shear at bottom of each grid cell. The effort did not model flood conditions.

In 2008, this analysis updated to include the 2-year recurrence interval riverine flood (62,000 cfs) during a typical tidal cycle. The memo compares model output at low flow and high flow, under fully restored and existing conditions. Comparisons of high flow under existing and with project conditions show no change in the water elevations. This confirms the 2002 PAS analysis that indicated insignificant water level changes in Ebey Slough resulting from dike breaching. This analysis contradicts a preliminary Sponsor analysis, based on empirical methods, which suggested a 60% increase in tidal exchange and an

increased potential for erosion (PWA 2005). The CFD modeling indicates that flow is clearly increased to the distributaries under high flow/restored conditions (the modeling includes net effects of not only the Qwulloolt project but several other restoration projects planned in the lower basin). Modeled salinity changes in Ebey Slough were minimal.

At the request of the PDT and Sponsor, further model refinements and high flow simulations were conducted in 2010, which includes the 100-year recurrence interval discharge. The purpose of the additional analysis was to be able to evaluate channel stability for a higher flow event along the streambanks, bed, and especially at bridge crossings. The output data includes peak velocity and direction, water surface elevation, and bed shear stress. The with-project conditions are modeled to show the effect of the project. Tidal energy is clearly concentrated in the deepest channels on site. Velocities in Ebey Slough do not appear to be significantly increased by the project, but should be great enough to readily transport course sand and erode over-steepened banks.

9.7 Assessment of Flood Risk

(Philip Williams and Associates (PWA) 2008)

The Tulalip Tribe contracted PWA to summarize results of a water level analysis to determine the existing flood potential for the Qwuloolt restoration site near Marysville, Washington. The objective of the exercise was to determine the total water level, defined here as the sum of the still water level (SWL) and the total wave run-up (R), for flood events of varying magnitude and source. The project objective is to not increase the existing flood potential for land owners that are adjacent to the site. Existing reports were reviewed and analyses were completed for the region in regard to flood hazard mapping and effects of restoration at the Qwuloolt site. Based on the existing information on extreme water levels and wind-waves, the flood frequencies for combined coastal and fluvial flood sources and estimated the height of wave run-up were computed. This analysis also included a prediction of estimated sea-level rise.

9.8 Allen Creek Flood Modeling

(Philip Williams and Associates (PWA) 2008)

The Tulalip Tribe contracted PWA to provide a detailed flood analysis for Allen Creek, which was performed as part of the greater Qwuloolt Tidal Wetland Restoration Project. The purpose of this analysis was to assess the flood impact the tidal restoration will have on the community of Marysville, WA along Allen Creek upstream of the 3rd Street crossing. The previous investigation had provided inconclusive results using a steady-state model. The existing steady-state HEC-RAS model was updated to include unsteady boundary conditions and geometry reflective of current conditions at the site. This model was run for a variety of

hydrologic scenarios for existing and proposed conditions. The Contractor updated the existing HEC-RAS model of Allen Creek to include unsteady-state boundary conditions and recent changes in the channel geometry in the lower reach of the creek. The proposed restoration of the Qwuloolt restoration site was incorporated into the model's geometry and compared to baseline conditions. The hydraulic model predicted negligible changes in peak water surface elevations for the lower reach of Allen Creek due to the tidal restoration project. Additionally, a conceptual-level culvert design for the 3rd Street Bridge was incorporated into the post-restoration geometry and the model predicted significantly reduced water levels through Jennings Park up to the main Jennings Park Bridge. Based on this analysis the contractor concluded that the Qwuloolt Tidal Wetland Restoration Project will not increase flood levels or flood risk in Allen Creek. The existing culvert at 3rd street is a hydraulic constriction that controls water levels upstream to the main Jennings Park Bridge.

9.9 Outboard Levee Breach and Tidal Channel Sizing

(Philip Williams and Associates 2008)

(10% - 65% analysis, superseded by 95% design)

The purpose of this study was to provide a basis of design for sizing the outboard dike breach. For this analysis, the concept of a breach entails the removal of the outboard dike and allowing tidal inundation on the inboard side of the dike. The hydraulics of the restored tidal marsh system requires adequate breach dimensions to convey flow between a tidal slough and the restored marsh site.

9.10 Industrial Park Stormwater Improvements

(Philip Williams and Associates 2008)
(10% analysis superseded by 65-95% design)

The purpose of this study was to evaluate and size a storage basin for stormwater runoff from a neighboring industrial park. This project doesn't neatly fit into the categories outlined in the guiding regulatory framework – City of Marysville Drainage and Erosion Control Design Standards and the Stormwater Management Manual for Western Washington (2005) (Western Washington Manual). The City of Marysville drainage standards indicate that 25-yr conveyance from the site is required. The site does not presently conform to this criterion and is in the floodplain resulting from coastal, fluvial and local flood sources. One goal of this project is to maintain the existing level of flood risk or improve conveyance of flows up to the 25-yr storm. Hence, a system including a levee and a drainage detention basin with tide gates has been developed to a preliminary level of detail.

Interior drainage design criteria

- Provide water quality treatment for all runoff that enters the system.
- Provide sufficient detention so that pumps are not required for the 25-yr, 24-hr storm.
- Runoff will be detained during high tides and released at low tide.
- Adjust the conceptual levee alignment to accommodate the detention system and future pump station for large storm events.

The SCS Curve number method was used to calculate the runoff for both the water quality storm and the 25-yr storm. The calculations are enclosed in an attachment. The water quality storm depth, also referred to as the 0.5-yr, 24-hour storm, at Everett, WA was retrieved from the Western Washington Manual, while the 25-yr, 24-hour storm depth was looked up in the NOAA Atlas 2 for Washington State. The rainfall depth for the water quality storm is 1.10 inches. The 25-yr storm depth is 2.5 inches.

The minimum requirements for the detention basin and treatment wetland include:
Total runoff volume to be detained, 6.3 ac-ft; the 25-yr runoff,3.9 ac-ft; water quality volume, 2.4 ac-ft. The outlet structure will consist of corrugated riser connected to barrels of the same size through the levee. Two 3-ft diameter discharge pipes fitted with tide gates will serve as a suitable outlet structure to drain the 25-yr storm in 48 hours.

9.11 Site Water Level Investigation

(USACE 2007A)

This study evaluated the baseline condition to the adjacent Marysville Industrial Park, Allen Creek, and Ebey Slough for a new set back levee as part of the restoration efforts for the Qwuloolt restoration project. The adjacent properties already experience some flooding and this study was conducted to determine if the construction of a new levee near the industrial park would exacerbate the situation. Data consist of 6-minute pressure readings for a 1 year period (low and high flow) at six locations. Minor flooding (street ponding) of the industrial park was documented on several occasions at the intersection of 47th Ave NE and 56th Pl NE and on the south end of 47th. Water levels rose above elevation 6.0 on several occasions at the two sensors placed on the west side of the Industrial park. A major storm caused water levels to rise above elevation 6.9 ft on Jan 18 2008 at the most floodprone sensor (corner of 47th and 56th). This flooding was confirmed by discussion with property owners. Flood problems appear to be a result of under-sized storm water outfalls to Ebey Slough on Allen Creek, resulting in ponding and backwater that then restricts outflow from the industrial park drainage system. At the time of the monitoring, an inflatable bladder was installed to isolate the section of the stormwater line draining the

intersection of 47th and 56th Pl NE. Pumps were being used to pump ponded water over a levee. The City reports that general settlement in the area has caused the damage to the pipes which is why the bladder was installed.

9.12 Qwuloolt Restoration project wind-wave analysis

(USACE 2007B)

(updated during 95% design)

This study investigated the potential fetch that may result from opening up an area behind a levee at the Qwuloolt restoration (near Marysville, WA) area to tidal influence. It is speculated that some form of shoreline protection and/or fill material mounding may be required in the area to dissipate the newly present wave energy. In this MFR, a wind-wave analysis is performed using historic wind data near the site. The size of armor material necessary to accommodate waves from the fetch is calculated. Finally, a recommended location for mound placement is discussed.

9.13 Computation of sea level change for Qwuloolt Restoration Project following EC-1165-2-211

(USACE 2010)

(10% - 65% analysis, superseded by 95% design)

In July 2009 the Corps issued new guidance for incorporating projected future sea level change in Civil Works Programs. Specifically, planning studies and engineering designs should consider alternatives that are developed and assessed for the entire range of possible future rates of sea-level change. Three projected values of sea level change were computed corresponding to the low, intermediate, and high rates outlined in EC-1165-2-211. Sea level change estimates must be based on tidal stations with more than 40 years of data. The closest tidal station to the Qwuloolt project with adequate record length is the Seattle, WA station. At this station the mean sea level rise from 1898 to 2006 is 2.06 mm/year. The projected construction date for the project begins in 2010. The intermediate sea level rise estimate for the 50 year design life in 2060 will be 0.72 ft. (USACE 2010).

Currently the set back levee is designed to a crest elevation of +14.5 ft NAVD88 with crest width of 12 feet (PWA 2008B; USACE 2009). This crest elevation currently includes 2 ft for wave run-up, and 0.5 ft for sea level rise, plus an additional 0.3 ft for settlement. Currently the set back levee design properly accommodates the projected intermediate sea level rise estimate. However, it is recommended that adaptability of the set back levee to

accommodate larger sea level rise be included in the operations and maintenance plan. For instance, if monitoring data indicate mean sea level rise approaching the estimate closer to the "High" estimate of 1.95 ft by 2060, the levee design should be able to incorporate incremental lifts in crest height while still functioning as intended.

9.14 Qwuloolt/Portinga Technical Report prepared for the Tulalip Tribes of WA

(USACE 2002)

This H&H study evaluated the feasibility and potential effects of breaching the existing dikes to restore Tidal hydrology and processes and associated habitats. The FEMA FIS 1-D hydrodynamic model UNET was modified and calibrated to match data collected by a tide station installed by NOAA on Ebey Slough to support this project. The model reproduced observed conditions well. Several Dike breach alternatives were evaluated under typical winter and summer conditions. As expected, full removal resulted in site water levels matching Ebey Slough water levels. Average site water levels post-restoration will be about 5.5 ft NAVD88. As the breaches were shortened the site water level response was muted. An optimal breach width (400 feet) that would be equivalent of full dike removal was determined through trial and error.

9.15 Snohomish River Revised Flood Insurance Study (RFIS) (Seattle District Corps/WEST consultants for the Federal Emergency Management Authority

(FEMA 2001)

An unsteady HEC-UNET model was constructed of the Snohomish River, including all major distributary channels. This model was calibrated to reproduce flooding that occurred during the 1990 flood. This model was used to determine the 10, 50, 100, and 500-year flood profiles, the 100 and 500-year floodplains, and floodway. The draft RFIS model was under review when accepted by USACE for application to the Qwuloolt restoration project PAS study. The RFIS model assumes that the levee protecting the restoration site will fail under 100-year flood conditions. Thus, behind the levees, the FEMA 100-year flood elevation should be the same regardless of the size of any restoration project. No flood analysis of Allen or Jones Creek was performed. The finalization date for the RFIS occurred in 2005. USACE is unfamiliar with any changes to the model that may have occurred since 2001.

9.16 Snohomish River Comprehensive Flood Control Plan

(Snohomish County 1991)

This comprehensive flood plan covered the Snohomish River valley from the confluence of the Skykomish and Snoqualmie Rivers to the mouth of the Snohomish River delta. The existing flood protection system involves a series of levees built and maintained by independent diking and drainage districts. The purpose of the plan is to establish County policies which will guide future construction and maintenance of levees and the overall flood protection needs of the valley. The Qwuloolt levee along Ebey Slough is part of Diking District #3, and provides "5-year + 1 ft" protection against high tide events.

10.0 - ADDITIONAL H&H ANALYSES UNDERWAY TO FULFILL SPONSOR OBLIGATIONS TO TRUSTEES

While the above analyses and investigations provide support to the design, additional effort is needed to verify that the project effects are locally acceptable. A condition of the City of Marysville shoreline permit for the sponsor is to conduct an analysis of the potential for offsite erosion to properties along Ebey Slough near the project. While some changes are expected, senior team members familiar with the available modeling and site conditions believe that the risk of adverse offsite geomorphic changes for infrastructure is relatively low.

At the request of the sponsor, USACE is actively engaged in a study of this potential risk. The study is scheduled for completion in October 2012. The study scope includes a PDT field reconnaissance of the banks of Ebey Slough adjacent to developed areas to obtain a "baseline" condition, focused sampling of the Ebey slough streambed to establish material properties and thresholds for erodibility in a "sed-flume". The work will include a comparison of measured bed sediment erosion thresholds with the Battelle high flow 3-D hydrodynamic modeling of with-project conditions and with the 1-D modeling performed to support the design. If erosional "hot spots" are identified, the geotechnical engineer will provide estimates of bank stability at critical locations (near infrastructure). If the erosion study determines that channel enlargement is a potential risk, the study will recommend monitoring protocols and potential mitigation measures that can be deployed if problems develop post-restoration. The Tribe will leverage the technical information provided by USACE to lead development and implementation of any required mitigation measures independent from the restoration project.

11.0 - REFERENCES

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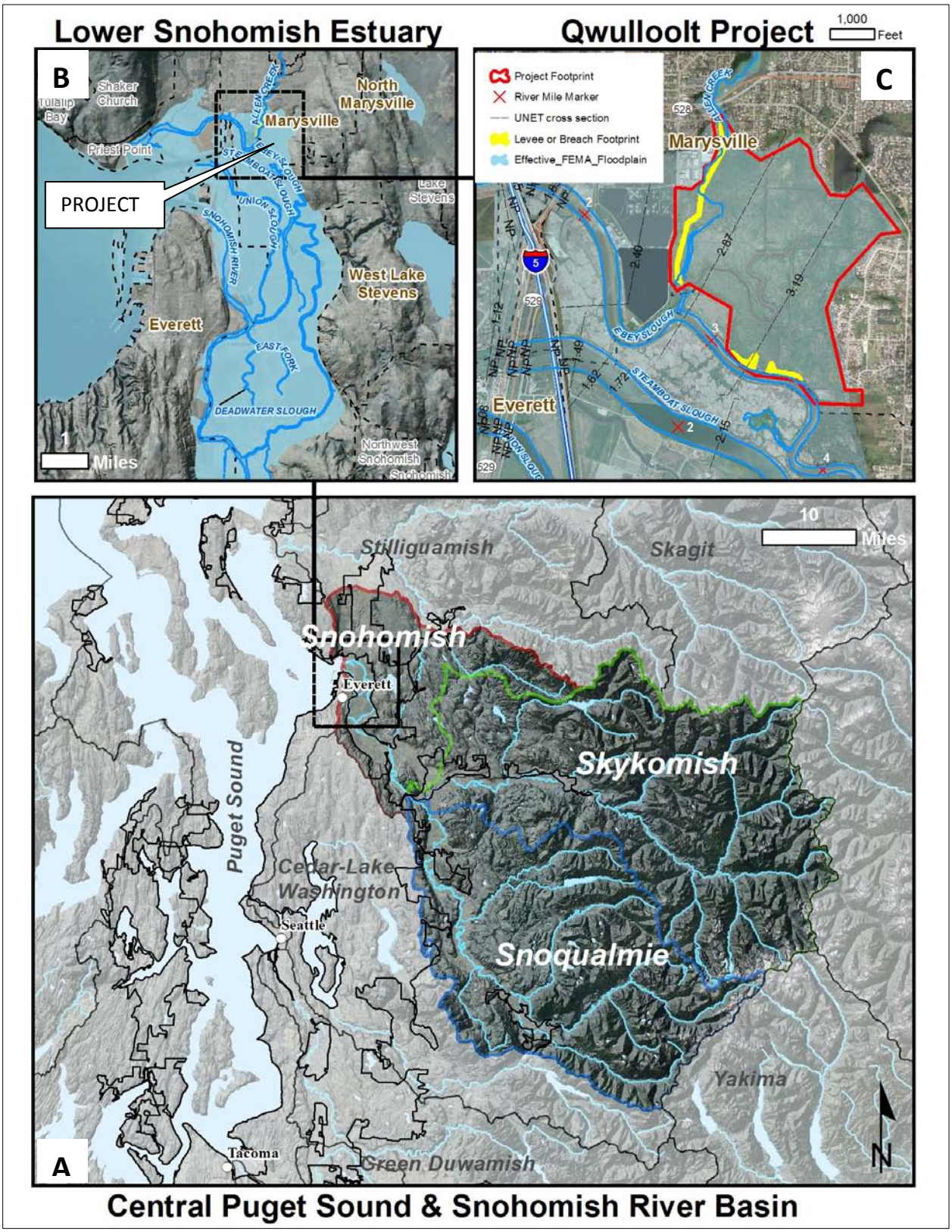


Figure 1 **Snohomish river basin, estuary, and project location map**

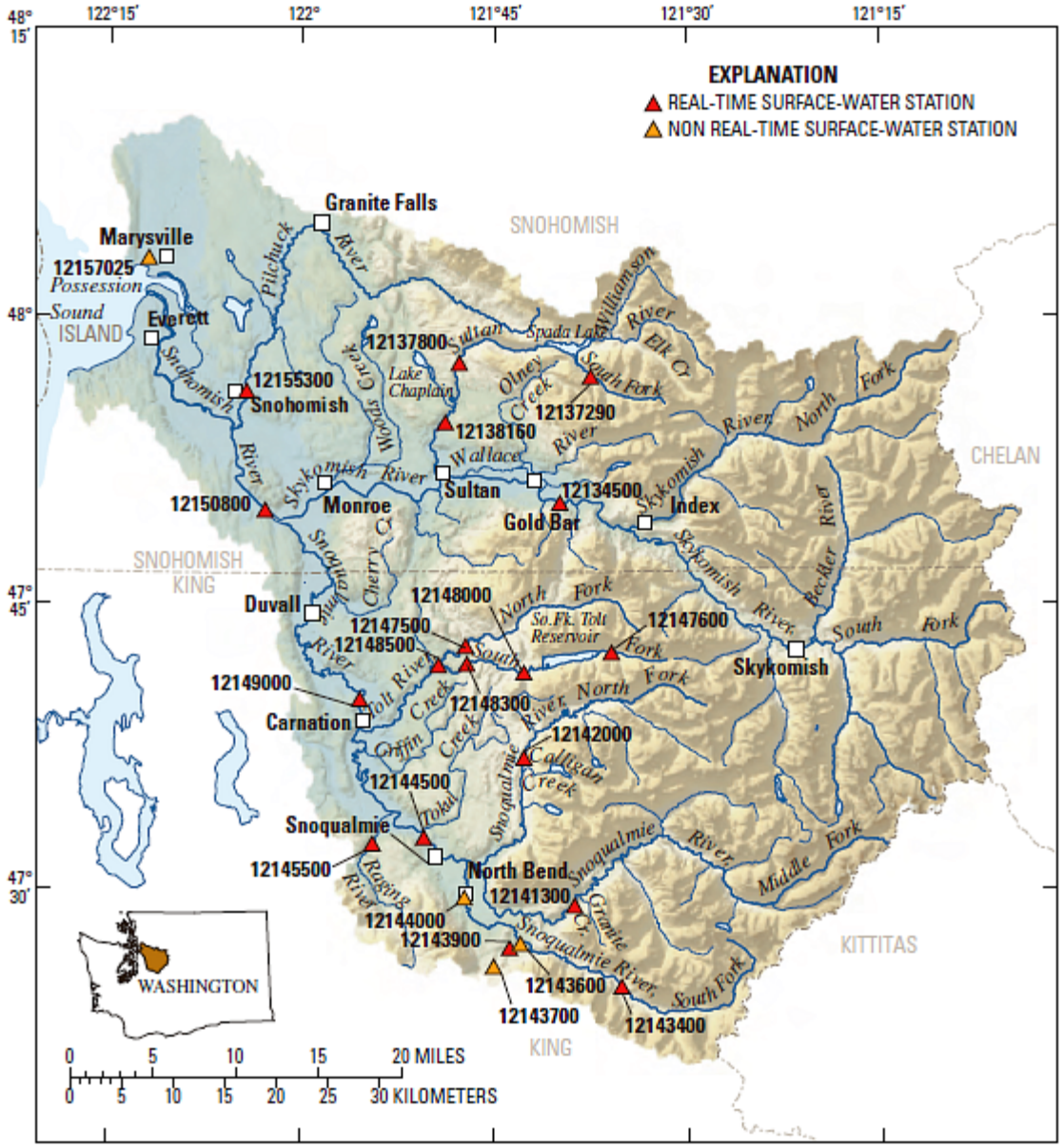


Figure 2 **Snohomish river basin USGS discharge monitoring stations**

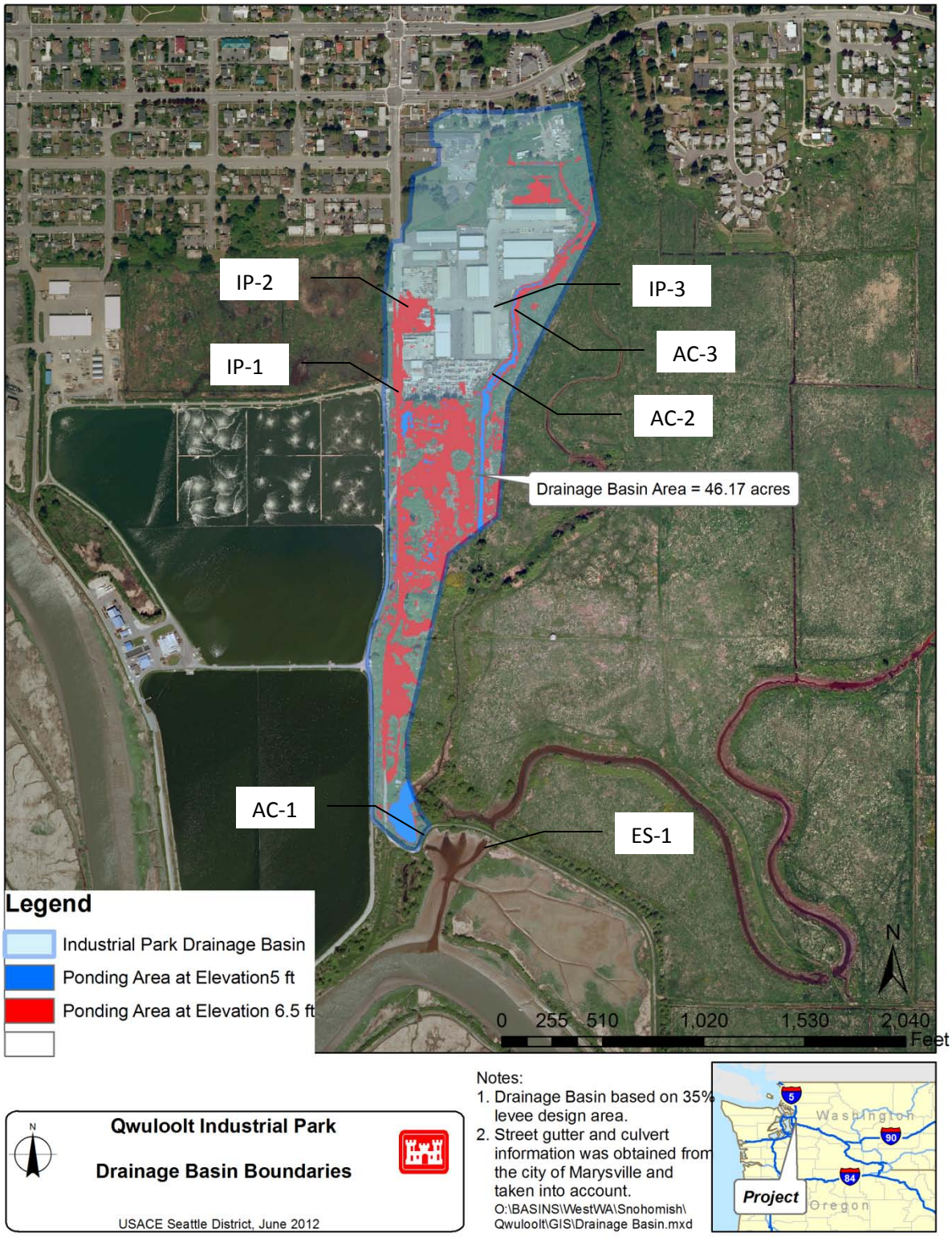
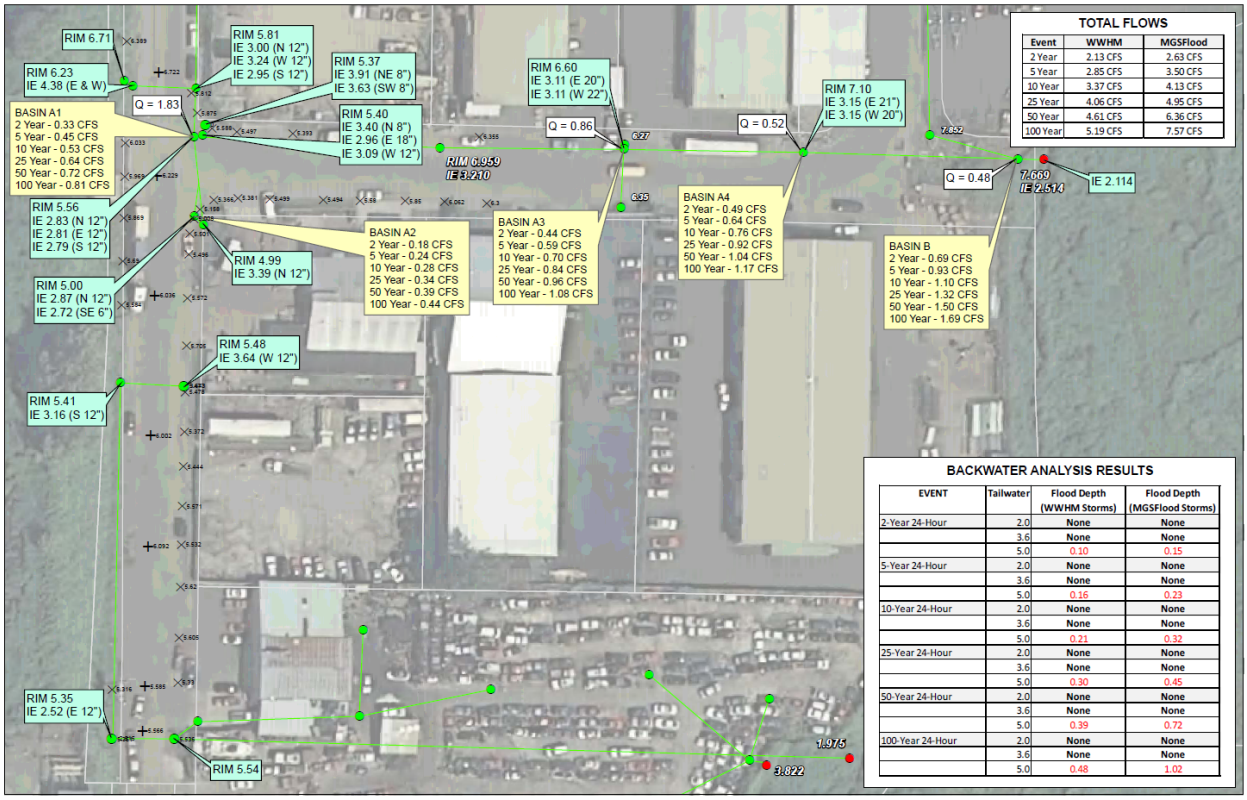


Figure 3 **Marysville industrial park basin map showing gage locations**

Figure 4 **Industrial park land use analysis by Marysville**



TOTAL FLOWS

Event	WWHM	MGSFlood
2 Year	2.13 CFS	2.63 CFS
5 Year	2.85 CFS	3.50 CFS
10 Year	3.37 CFS	4.13 CFS
25 Year	4.06 CFS	4.95 CFS
50 Year	4.61 CFS	6.36 CFS
100 Year	5.19 CFS	7.57 CFS

BACKWATER ANALYSIS RESULTS

EVENT	Tailwater	Flood Depth (WWHM Storms)	Flood Depth (MGSFlood Storms)
2-Year 24-Hour	2.0	None	None
	3.6	None	None
	5.0	0.10	0.15
5-Year 24-Hour	2.0	None	None
	3.6	None	None
	5.0	0.16	0.23
10-Year 24-Hour	2.0	None	None
	3.6	None	None
	5.0	0.21	0.32
25-Year 24-Hour	2.0	None	None
	3.6	None	None
	5.0	0.30	0.45
50-Year 24-Hour	2.0	None	None
	3.6	None	None
	5.0	0.39	0.72
100-Year 24-Hour	2.0	None	None
	3.6	None	None
	5.0	0.48	1.02

Figure 5 **Industrial park backwater analysis by Marysville**

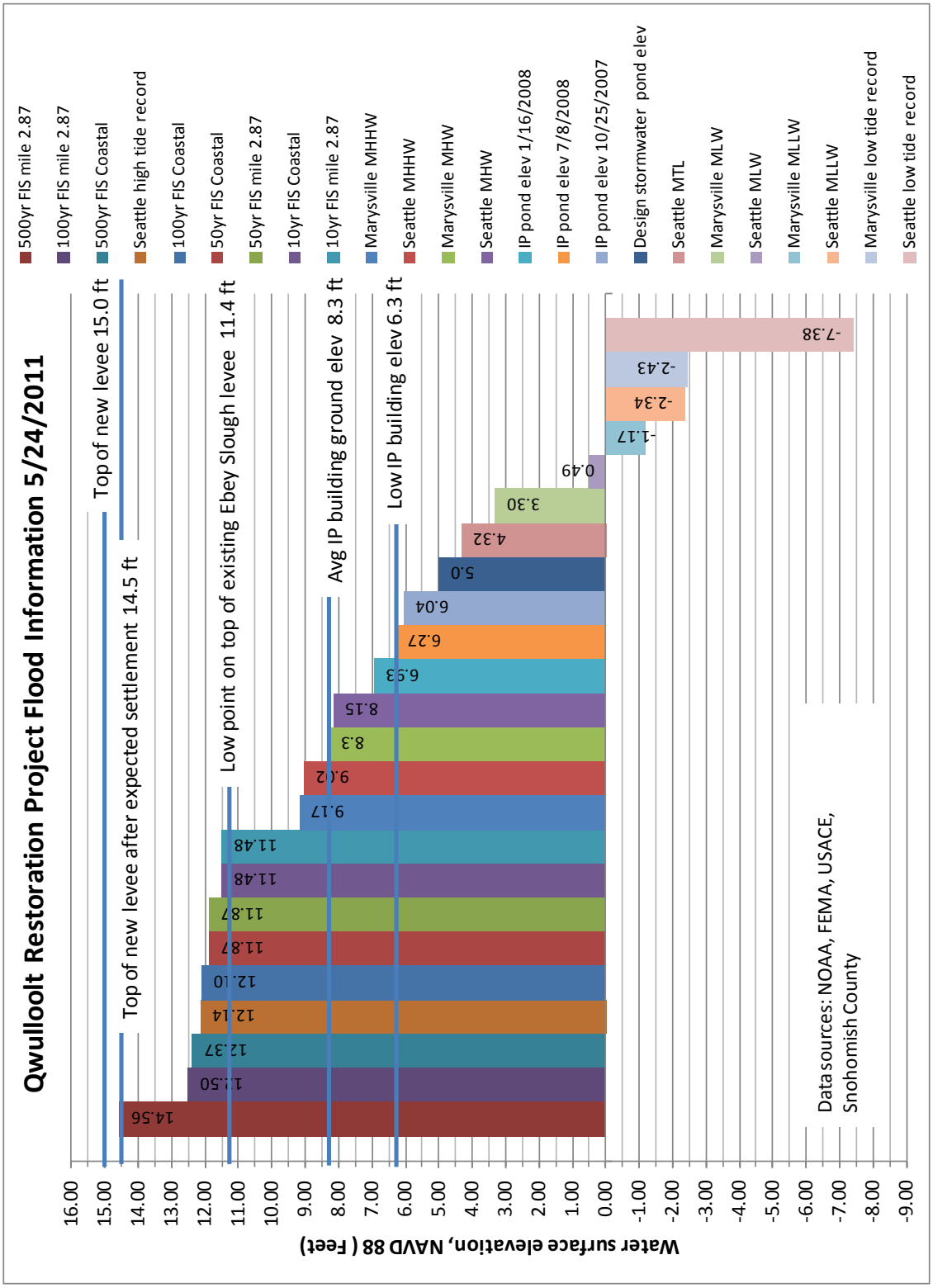


Figure 6 **Qwulloolt site water level summary**

Table 1 Snohomish River Basin Characteristics Report

Date: Mon Jun 21 2010 17:12:38
Latitude (NAD83): 48.0201 (48 01 12)
Longitude (NAD83): -122.2072 (-122 12 25)

Parameter	Value
Area that drains to a point on a stream in square miles	1780
Relief, maximum - minimum basin elevation in feet	7950
Mean basin elevation in feet	2220
Maximum basin elevation in feet	7950
Minimum basin elevation in feet	0
Mean basin slope in percent	37.8
Percent of area with slope greater than 30%	54.1
Percent of area with slope greater than 30% and facing North	14.1
Percent of area covered by forest	63.4
Mean annual precipitation in inches	94.2
Source: USGS Streamstats, 2010	

Table 2 Snohomish River Basin Peak Flood Discharge Data (FEMA UNET Model 2005)

Location of Measured Peak Flow in UNET Model	Peak Flow (cfs)
Ebey Slough Cross Section 3.19 (At Restoration Site)	19,000
Steamboat Slough Cross Section 2.6	56,000
Union Slough Cross Section 1.47	11,000
Snohomish Mainstem Cross Section 2.02	81,000
Sum of Peak Flows in Sloughs and Mainstem near Restoration Site	167,000
Snohomish Mainstem Cross Section 20.438 (Upstream End of Model)	204,000

Table 3 Levee Breach Dimensions (PWA 2008a)

Breach Design Parameter	Constructed Dimension
Depth Below MHHW (ft)	21.2
Invert Elevation (ft NAVD)	-12
Bottom Width (ft)	50
Side-slope (H:V)	5
Width at MHHW (ft)	262
Cross-sectional Area below MHHW (sf)	3,307

(USACE 2012D)

Breach Design Parameter	Constructed Dimension
Depth Below MHHW (ft)	18.0
Invert Elevation (ft NAVD)	-9.0
Bottom Width (ft)	50
Side-slope (H:V)	4
Width at MHHW (ft)	194
Cross-sectional Area below MHHW (sf)	2,200

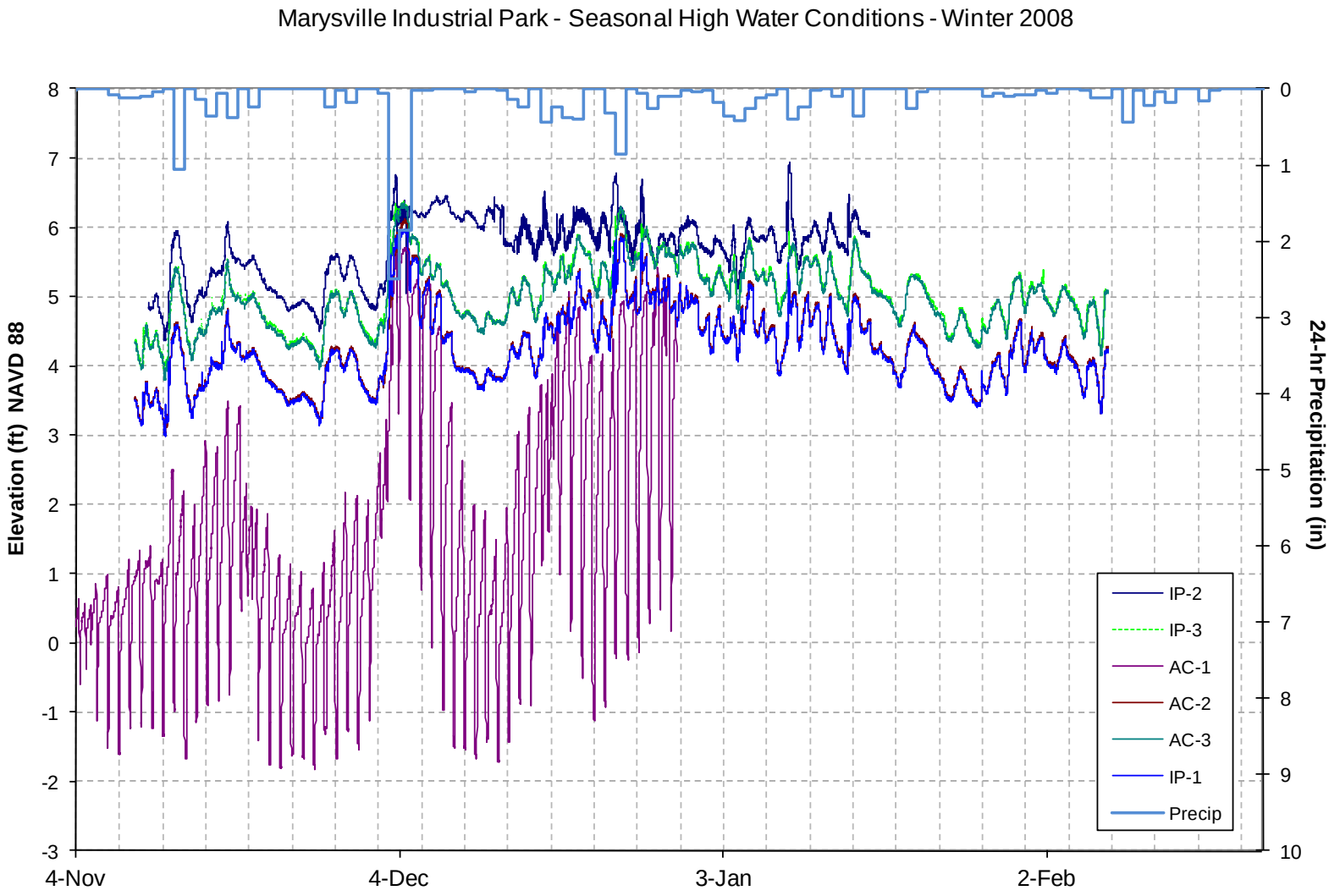


Figure 7 **Marysville (Brashlers Industrial Park) Water Level Monitoring (USACE 2007A)**

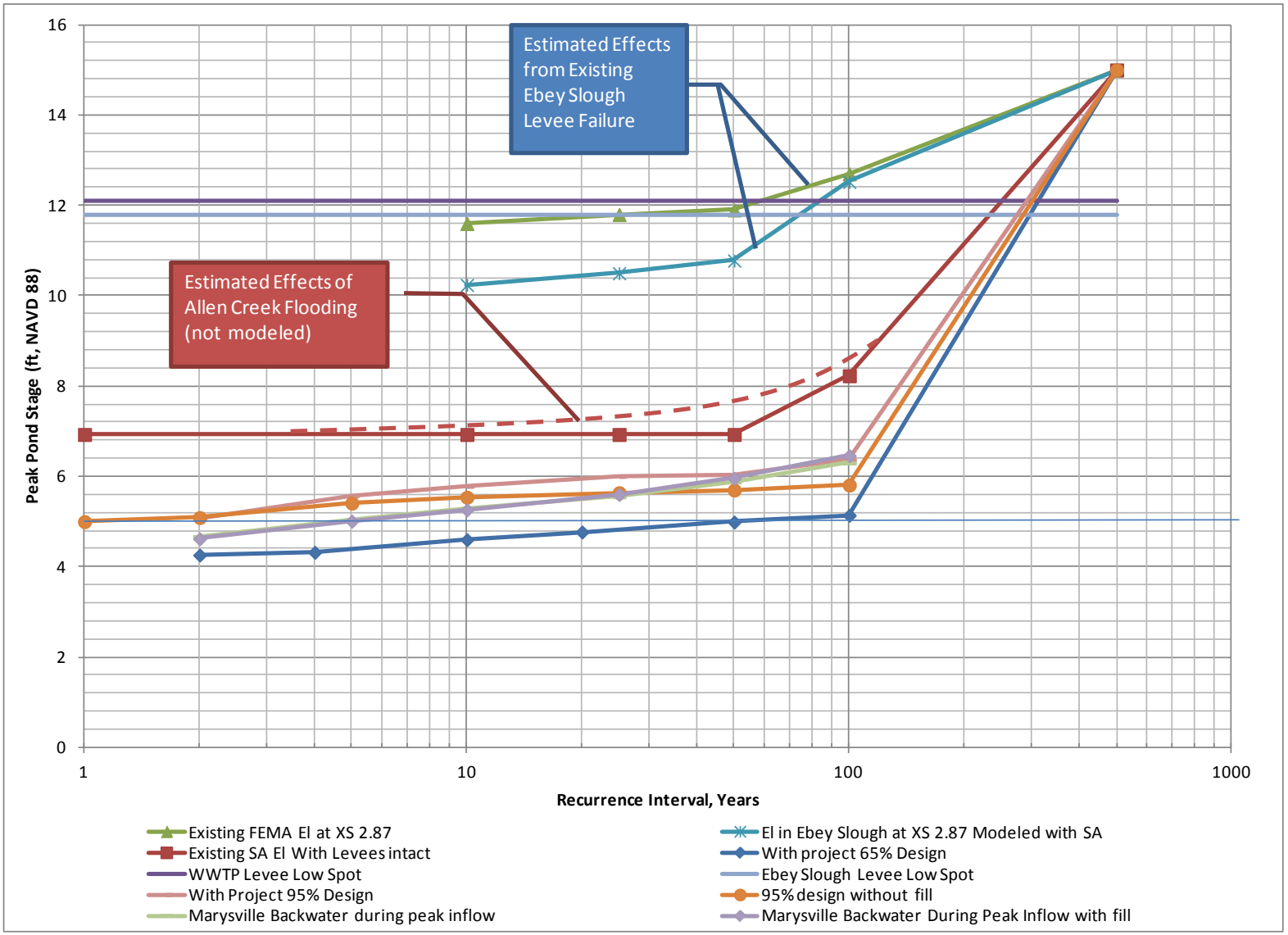


Figure 8 **Marysville (Brashlers Industrial Park) Water Levels Pre-Post Restoration (USACE 2012H)**

Table 4 Stormwater pond hydrologic data (USACE 2012A, 2012H)

100-year peak inflow rate (cfs)	13.3
100-year peak inflow volume (24 hour) (acre-ft)	10.85
100-year peak pond stage (coincident with 2-year high tide) (ft)	6.5
100-year peak pond stage (due to 100-year event levee overtopping) (ft)	4.6
10-year peak inflow rate (cfs)	8.6
10-year peak inflow volume (24 hour) (acre-ft)	6.9
10-year peak pond stage (ft)	5.8
2-year peak inflow rate (cfs)	5.3
2-year peak inflow volume (24 hour) (acre-ft)	4.1
2-year peak pond stage (ft)	5.1
0.5-year 24-hour peak inflow rate (cfs)	2.7
0.5-year 24-hour inflow volume (acre-ft) – Water Quality Storm	2.0
Water Quality Storm peak pond stage (ft)	4.0

Table 5 Stormwater pond design requirements for flood detention storage (USACE 2012A, 2012E, 2012H)

Pond area at elev. 5.0 ft NAVD88) (acres)	2.5
Pond area at elev. 3.0 ft NAVD 88 (acres)	0.0
Pond volume for flood detention storage (below elev. 5 ft NAVD88) (acre-ft)	5.0
24-hr 0.5 year WQ storm runoff volume, acre-ft	2.0
Pond volume for WQ storm detention (below elev. 4 ft NAVD88) (acre-ft)	1.7
Pond volume for flood detention storage (below elev. 5.5 ft NAVD88) (yd3)	10,800
Pond side-slope (H:V)	3:1
# outfall pipe(s) , risers, flapgates (ea.)	2
Outfall pipe diameter (in)	36
Number and diameter of WQ treatment orifices in riser pipes (quantity, ft)	4, 5 inch
Outfall riser pipe invert elevation (downstream) (ft NAVD 88)	-2
Outfall riser pipe invert elevation (upstream) (ft NAVD 88)	1.0
Outfall pipe slope	2%
Recommended pipe material	Concrete or HDPE or CMP

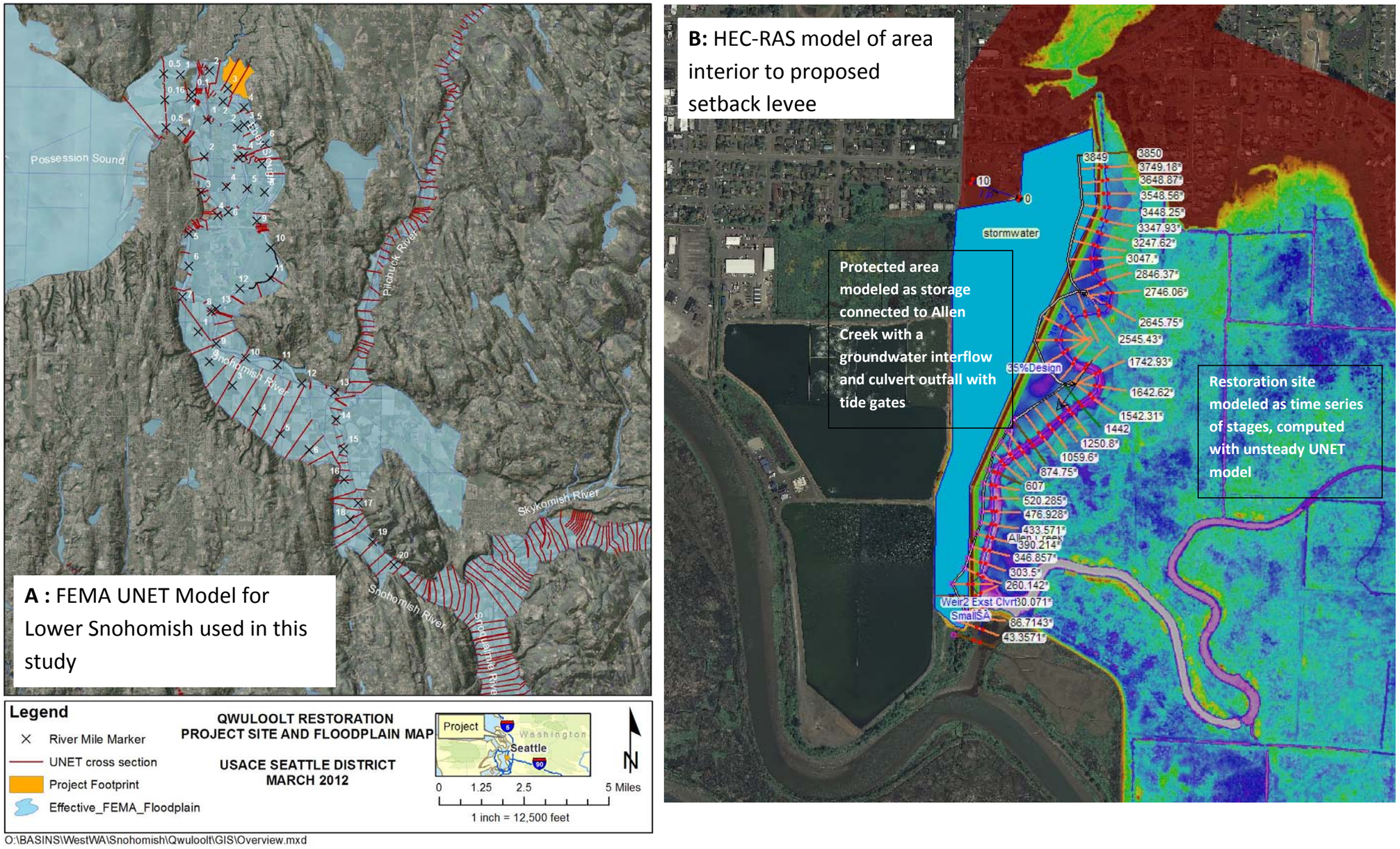


Figure 9 **Hydraulic Model Schematics for Qwulloolt Restoration Project**

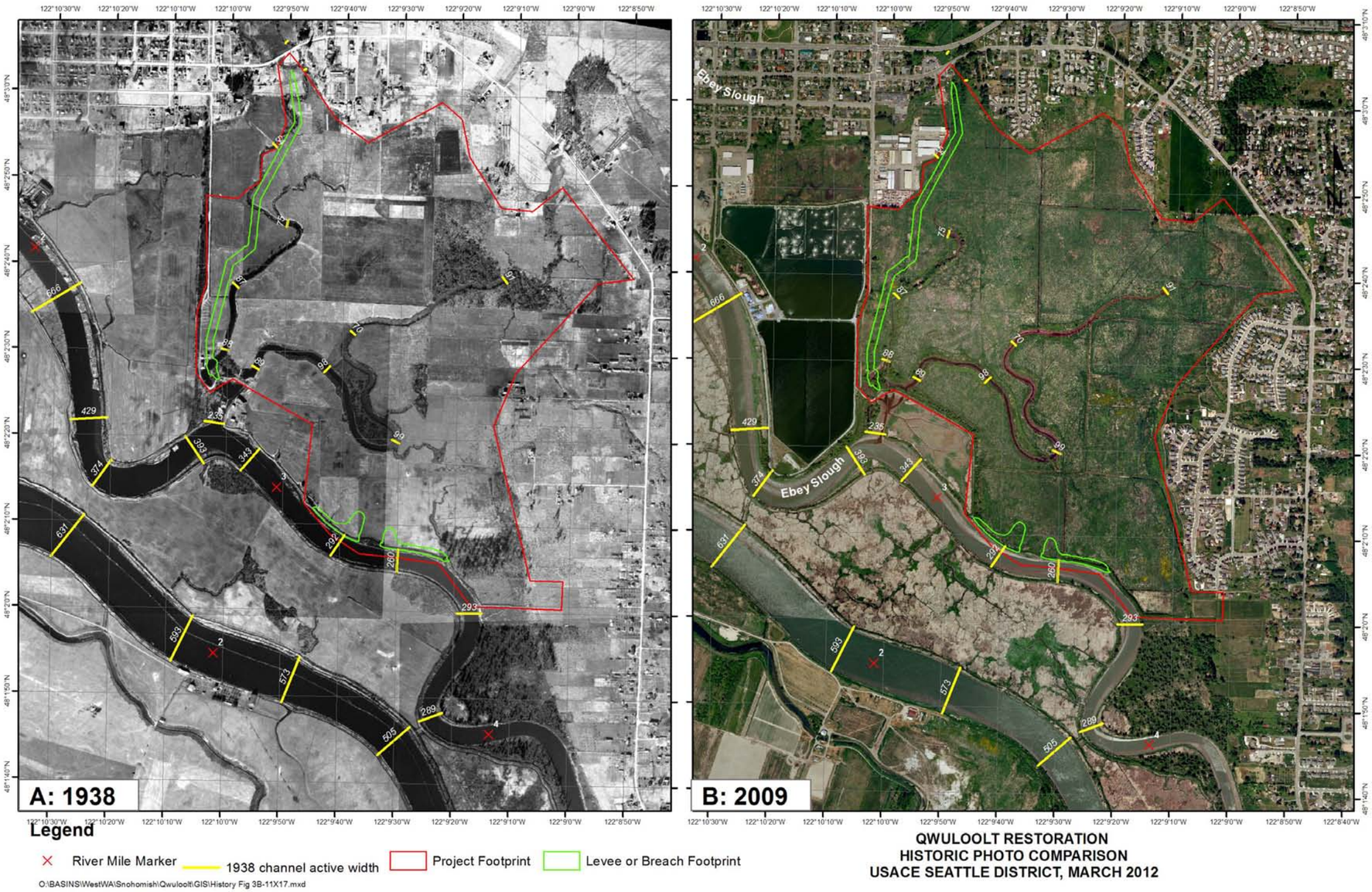


Figure 10 **Comparison of Channel Width and Planform in the Lower Snohomish Estuary, 1938 vs. 2009**

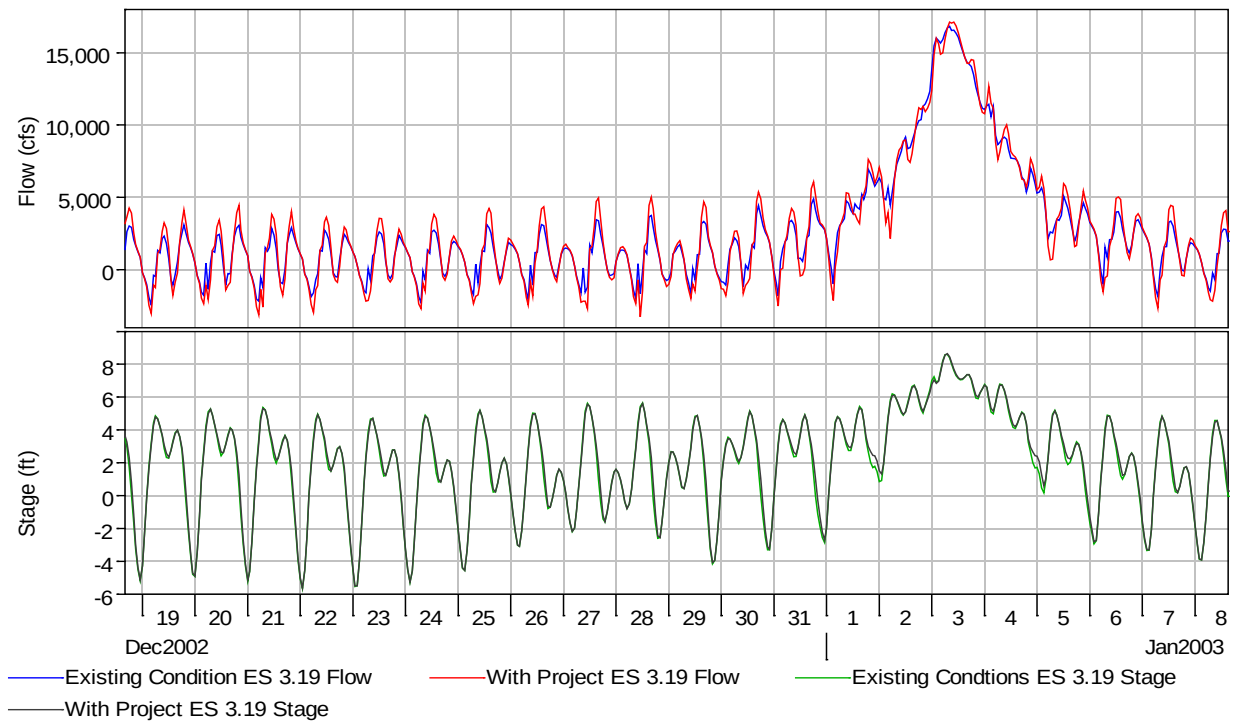


Figure 11 Comparison of 100-year flows and stages at project site, pre and post restoration. (ES = Ebey Slough)

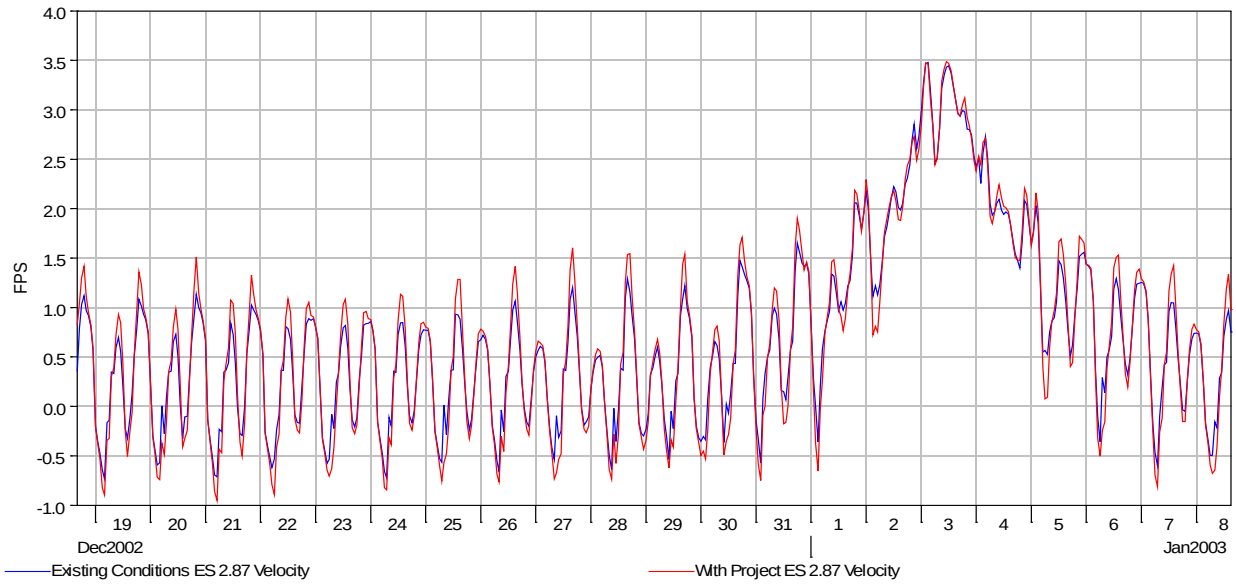


Figure 12 Comparison of 100-year velocities downstream of project site, pre and post restoration. (ES = Ebey Slough)

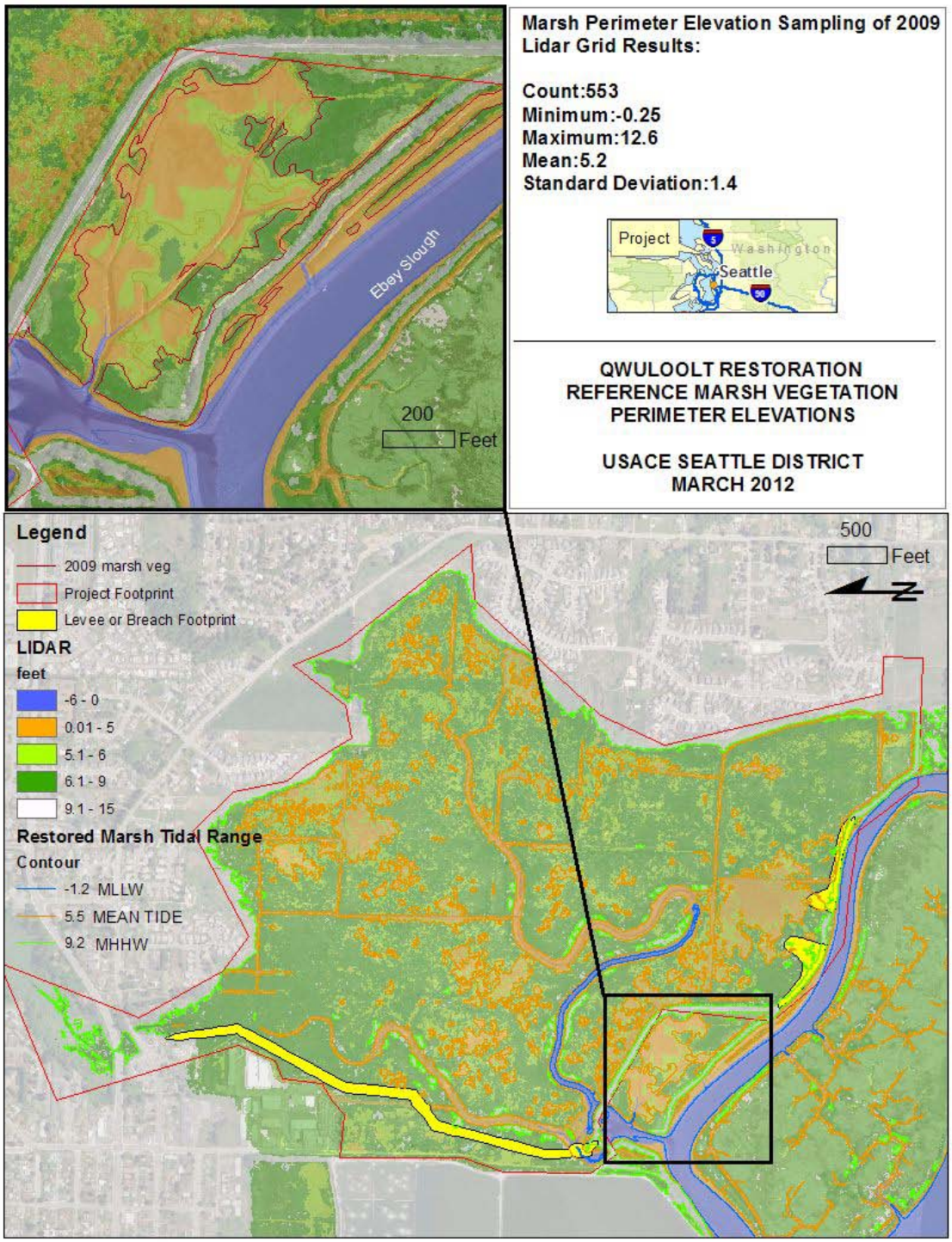


Figure 13 Reference marsh site vegetation perimeter elevation analysis and Restored Marsh Tidal Range

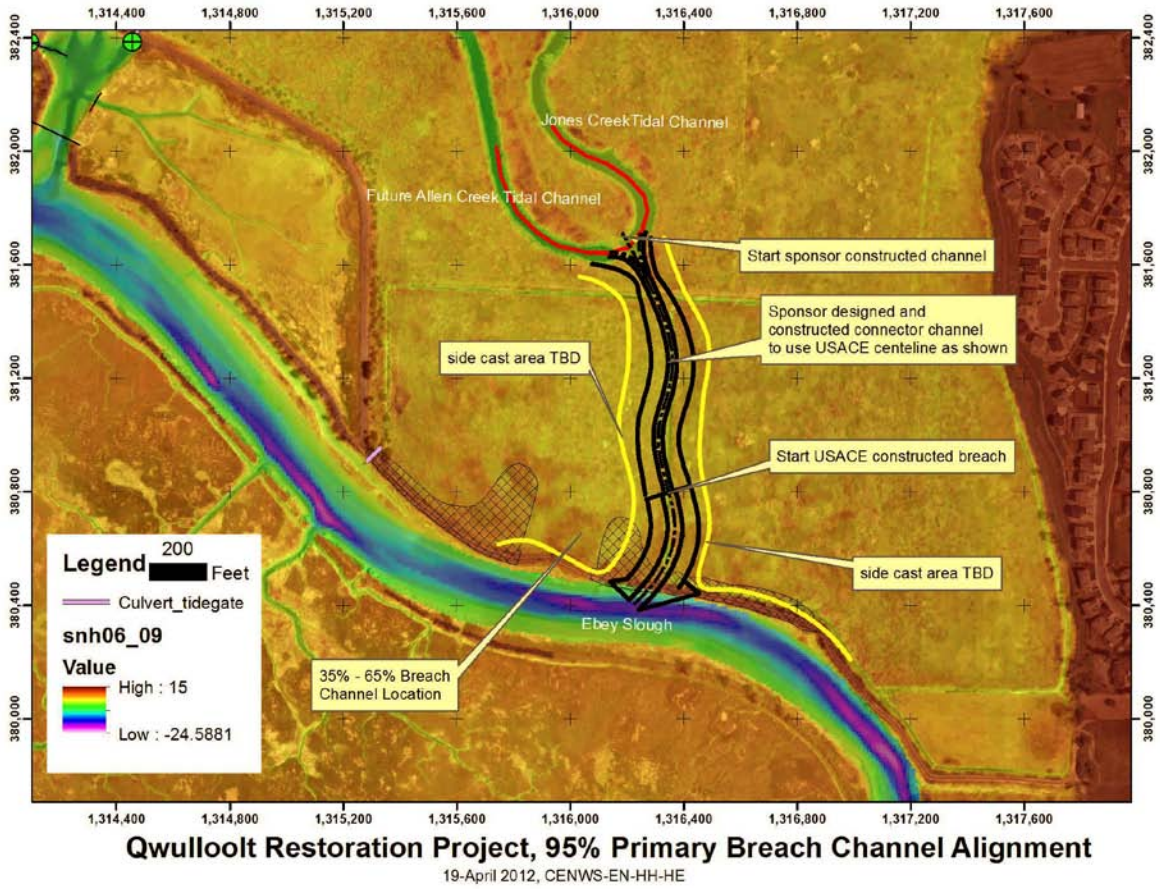


Figure 14 **Recommended breach channel location and configuration**

TYPICAL WINTER (as represented by December 12,1980)

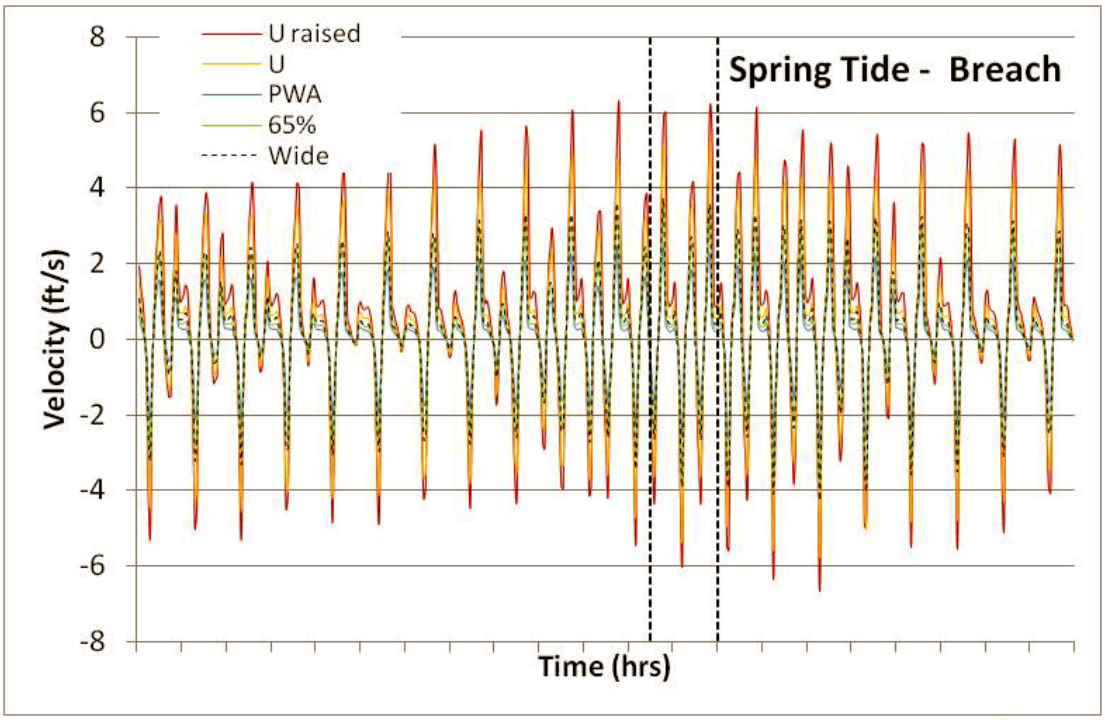
STAGE INTERVAL NAVD 88 DATUM	% OF TIME INTERVAL INUNDATED		% OF TIME STAGE IS WITHIN INTERVAL	
	OPTION A	OPTION B	OPTION A	OPTION B
-1 TO 0	88	87	12	13
0 TO 1	83	83	5	4
1 TO 2	80	80	3	3
2 TO 3	78	78	2	2
3 TO 4	74	74	4	4
4 TO 5	66	68	8	6
5 TO 6	43	40	23	28
6 TO 7	24	24	19	16
7 TO 8	14	14	10	10
8 TO 9	7	3	7	11
9 +	0.5	0.5	6.5	2.5

TYPICAL SUMMER (as represented by June 12, 2000)

STAGE INTERVAL NAVD 88 DATUM	% OF TIME INTERVAL INUNDATED		% OF TIME STAGE IS WITHIN INTERVAL	
	OPTION A	OPTION B	OPTION A	OPTION B
-1 TO 0	94	94	6	7
0 TO 1	89	90	5	4
1 TO 2	87	88	2	2
2 TO 3	83	85	4	3
3 TO 4	80	81	3	4
4 TO 5	60	60	20	21
5 TO 6	40	40	20	20
6 TO 7	18	18	22	22
7 TO 8	10	7	8	11
8+	0.5	0.5	9.5	6.5

To convert to NGVD, subtract 3.687 ft.

Table 6 Seasonal inundation duration information for restored marsh for typical summer and winter, from USACE 2002. 95% Design is condition between Option A and B. Typical (50% exceedance) stage post restoration is approximately 5.5 ft, or slightly above the average site elevation.



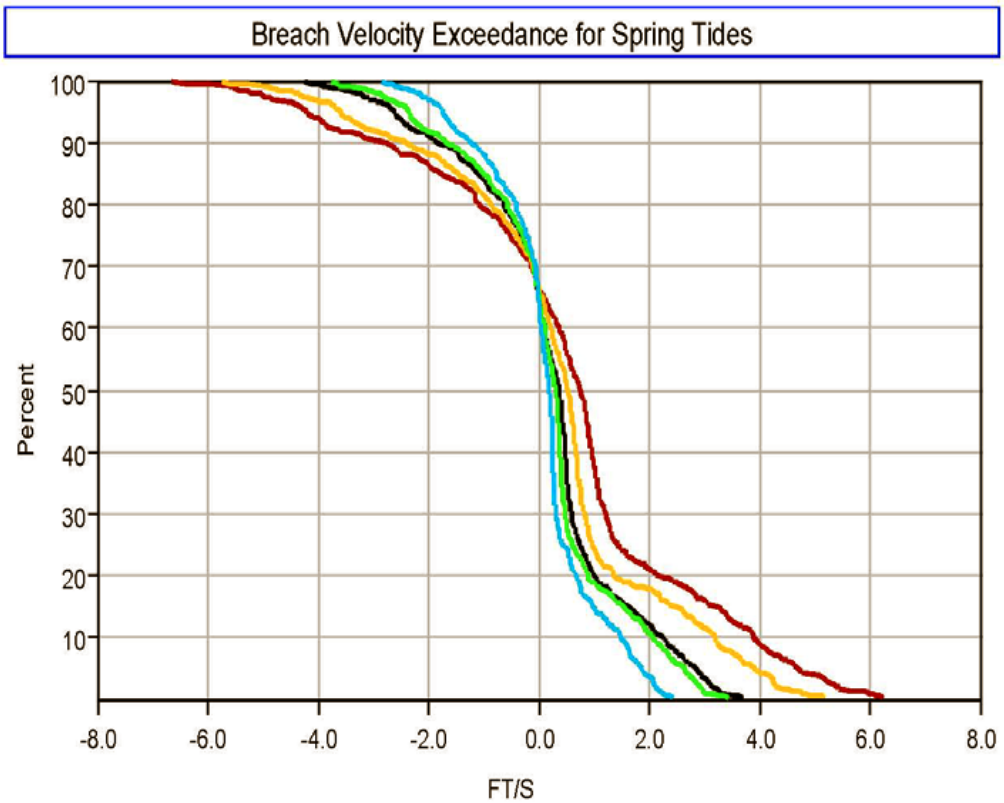


Figure 15 Breach channel velocities during typical spring tide for options evaluated for 95% design (USACE 2012D). 65% design (green line) was selected as final design. Positive velocity values reflect outflow from the site.