Fisher Slough Final Design and Permitting
Final Basis of Design Report

Skagit County, WA

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Prepared by:

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1.0 INTRODUCTION

The Fisher Slough Restoration Project is a 60 acre tidal marsh restoration project located on the South Fork Skagit River Delta. The project has several components, including: restoration of historical tidal marsh vegetation communities and rearing areas for juvenile Chinook; removal of fish passage barriers; provide watershed connectivity for coho, chum and other native fish species; and improvement of flood and sediment storage conditions for the tributary levee system.

The project is a collaborative effort between The Nature Conservancy (TNC), Dike District #3 (DD#3), Drainage District #17 (DD#17), Skagit County, and a number of other government agencies, technical reviewers and local landowners. TNC has actively engaged the project partners using a proactive collaborative approach for making informed technical decisions regarding the project development, design, implementation, long term operations, maintenance and adaptive management strategies.

The Fisher Slough project site is located just south of the town of Conway, WA, at the downstream end of the Carpenter Creek watershed, at the confluence with Tom Moore Slough on the South Fork of the Skagit River (Figure 1).

Restoration planning has been underway for the project since 2004 and the planning process was recently completed in March of 2009. This Final Basis of Design Report is the third design submittal to TNC, and incorporates the recommendations from the planning process into final design, permitting, and development of construction contracting documents.

1.2 PROJECT RESTORATION OBJECTIVES

The Nature Conservancy, along with their project partners and technical advisory committee, has developed a mission statement and objectives for the Fisher Slough Restoration Project. The mission statement and guiding principles are:

Maximize the area influenced by natural stream and tidal processes, allow for a broad range of ecosystem variability, restore estuarine rearing habitat for juvenile salmon to the maximum extent possible, and improve flood protection and storage capacity for Carpenter Creek and Fisher Slough.

A series of primary and secondary objectives were developed in order to provide guidance and strategies for prioritizing different aspects of the project. Each of these objectives are further refined into specific design and evaluation criteria in order to support evaluation and selection of final design configurations and recommendations.

**Primary objectives (should drive alternatives selection and design):**

- Create a diverse array of native vegetative communities.
- Create freshwater tidal marsh Chinook (*Oncorhynchus tshawytscha*) salmon rearing habitat.
- Provide fish passage for coho (*Oncorhynchus kisutch*) and chum (*Oncorhynchus keta*) spawning access.
- Improve flood storage to protect agricultural uses of adjacent properties.

**Secondary objectives (should not compromise primary objectives):**

- To improve habitat opportunity for cold water fish other than Chinook.
• Improve habitat opportunity for migratory birds.
• Identify opportunities to address ongoing sedimentation and flooding issues in Carpenter Creek tributaries (e.g. Johnson and Sandy Creeks).
• Maximize the opportunity to export lessons learned.

1.3 PREVIOUS STUDIES AND INVESTIGATIONS

There are a number of previous restoration planning and technical analysis documents and studies that have been performed on the project to date. These documents form the basis of understanding of the project baseline site conditions including: environmental resources, hydrology, hydraulics, sediment, geomorphology, geotechnical, structures and facilities. They also document the process and decisions for the final restoration plan and design recommendations. A brief summary of each of these supporting documents and the information available in each report is provided for reference.

Fisher Slough Preferred Restoration Plan, TNC, March 2007

The Fisher Slough Preferred Restoration Plan is a feasibility level document that established the restoration goals and objectives of the project, and documented the baseline site conditions including geomorphology and landscape, habitat and vegetation, and hydrologic, hydraulic and sediment assessments and modeling. Concept design features, plans and cost estimates were identified that addressed the restoration objectives. Major concept design features included the following:

• Floodgate replacement
• Levee setback alignments
• Big Ditch crossing realignment
• Tributary and tidal channel realignments and excavations
• Marsh, wetland and riparian plantings

These concepts were evaluated in three distinct alternatives and compared with the existing baseline conditions to document restored condition improvements in tidal marsh habitat, flood storage and sediment storage. Three major outcomes came from the analysis. The first was that the levee setback alignment was linked to the Big Ditch crossing location, and realigning Big Ditch allowed for full levee setback, thereby maximizing tidal marsh habitat areas, and demonstrated improvements in long term flood and sediment storage. The second was floodgate replacement is an integral part in providing fish passage as well as flood control operations for the Slough, that was not originally included in preliminary restoration planning. Finally, two restoration approaches, active and passive, were available for the tidal marsh area. The overall outcome of the study was to include floodgate replacement, Big Ditch realignment, full levee setback and further evaluate the active and passive restoration approaches for implementation on the project.

Technical investigation information relative to the final design that were developed in the site feasibility assessment include the following:

• Determination of project tidal, NGVD29 and NAVD88 datum relationships were made. The project datum is NAVD88. The following conversion factors are applied at the site:
  o NGVD29 (Elevation) + 3.78ft = NAVD88 (Elevation) – Mt. Vernon USGS Gaging Station (I.D. 12200500 - Datum is NGVD29) is the reference station for the project site.
  o MLLW (Elevation) – 1.50ft = NAVD88 (Elevation) – Ala Spit (I.D. 9447993 – Datum is MLLW) is the reference tide station for the project.
  o Fisher Slough is a freshwater tidal marsh (currently no brackish conditions) and has tidal flow signature for low flow/stage conditions similar to Ala Spit, with an approximate 1-hour lag/delay in tidal stages. During higher flow spring runoff and flood conditions,
Fisher Slough stage is represented by Mt. Vernon flow stages, as adjusted for changes in elevation between the sites.

- Hydrologic modeling using HEC-HMS and HEC-RAS routing model to predict flood inflows to Fisher Slough from Hill Ditch, Big Fisher Creek and Little Fisher Creek were performed. Hydraulic modeling shows significant upstream road and levee overtopping as a result of Fisher Slough MHHW (9.5ft NAVD88) backwater conditions occurring between the 10 and 25-year tributary watershed runoff events.

- FEMA maps were reviewed and mapped flood elevations inside the Dike District #3 levees were shown as 9.0ft elevation (NGVD29) and adjusted to NAVD88 are 12.7ft. Outside the levees on the Skagit River, FEMA maps show the flood inundation levels as 13.0ft (NGVD29) and adjusted to 16.7ft (NAVD88)

- Review of Skagit River hydraulic UNET/FLO-2D modeling results (NGVD29) from the Skagit River Flood Study (USACE, 2005) to identify downstream boundary conditions and flood depths as a result of levee Probable Failure Point (PFP) scenarios for the Dike District #3 levee south of Mt. Vernon, WA. Flood depths for the UNET modeling node downstream from Fisher Slough were used for initial flood modeling. Also, the FLO-2D modeling output was reviewed and 9.0ft depths were observed on the floodplain grid just upstream from the site. These both correlate well with the FEMA published flood elevation of 16.7ft both inside and outside the levees. The levee failure scenario stage of 16.7ft is used to model peak flow conditions and water surface elevations for inverted siphon and levee designs (seepage inundation elevations and emergency spillway conditions).

- Initial MIKE-11 study modeling results documented that Fisher Slough levee setback, floodgate retrofit, and tidal marsh restoration alternatives would have little effect on Skagit River flooding conditions, but demonstrated significant improvements in lowering flood stages for more periodic and nuisance flooding associated with the 5 to 10-year flood events. The modeling documented nearly 3-ft reduction in flood elevations for proposed floodgate and levee setback conditions for 5-year flood event.

Carpenter Creek, Hill Ditch and Fisher Slough Watersheds Initial Flood and Sediment Study, Skagit County, March 2007
A parallel study was performed addressing flooding and sedimentation on the contributing tributary watersheds to Fisher Slough. The intent of the study was to assess flood hydrology and sediment transport conditions of the six tributary watersheds, providing watershed context to the Fisher Slough Restoration Project and identify preliminary design features, and flood management plans for reducing flood and sedimentation risks in the watershed. A number of concept design alternatives and management strategies are presented and prioritized in a summary list for planning and implementation of future projects. In addition, the study provided baseline hydrologic, hydraulic and sediment inputs to the Fisher Slough modeling systems.

Technical investigation information relative to the final design from the Carpenter Creek flood and sedimentation investigation includes the following:

- HEC-RAS modeling showed how lowering of stages in Fisher Slough could reduce the amount of flooding and road overtopping on the Hill Ditch system.

- Sediment transport investigations were used in conjunction with Fisher Slough dredging and as-built records to document long-term sedimentation rates. These rates were then assessed to compare long term sedimentation rates for different levee setback and channel restoration and grading alternatives.

- A number of flood improvement alternatives were identified for future implementation that would ultimately reduce the peak flows from the tributaries into Fisher Slough.

The Floodgate Design Recommendations report contained engineering analyses to evaluate and select the preferred gate replacement plan. Within this report, detailed hydrologic and hydraulic modeling was performed to assess hydrologic conditions during key fish passage and flood periods.

The engineering analysis of the floodgate alternatives involved comparing 1-gate replacement and 3-gate replacement scenarios. It was demonstrated that for fish passage conditions, the 3-gate replacement was ideal in meeting fish passage velocity barrier criteria. For flooding conditions, it was also demonstrated that the replacement of the floodgates in combination with the levee setback aspect of the project significantly improved flood conditions by providing increased flood storage for more frequent tributary flooding and associated problems upstream. The project has no effect on the Skagit River 100-year flood conditions in the project area.

The outcome of the report was a recommendation for replacing 3-gates, and provided the initial basis for developing the operations and maintenance plans of the floodgate aspect of the project.

Technical investigation information developed for the final design from the Carpenter Creek flood and sedimentation investigation includes the following:

- Evaluation of flood risks by period (Fall/Winter Flood Period, Spring Juvenile Chinook Migration Period, Summer Irrigation Period) to assist in defining future floodgate operations, and evaluate proposed retrofit floodgate conditions during these periods.
- Evaluation of acceptable operating ranges for the floodgate determined to be 7.5ft NAVD88 (Fall/Winter Period), 9.5ft (NAVD88) (Spring Juvenile Chinook Migration Period) and fully open (Summer Irrigation Period).
- Documented increases and significant improvements in fish passage criteria for the percent time that the gates would be open and the ability to meet minimum depth and maximum velocity criteria for juvenile chinook.
- Evaluated tidal muting characteristics between 1-pair (2.0ft of muting) and 3-pair (0.8ft of muting) floodgate replacements which resulted in different amounts of tidal marsh inundation.
- Documented proposed 311 acre-ft (3-pair gates) of available flood storage with a high stage occurring, which is approximately 50% of the total runoff for the 5-year event and 15% storage of the 100-year event.


Immediately following the floodgate design recommendations report, an expedited floodgate performance specification design report was developed to support installation in 2008. The report outlines the potential types of gates that were available for installation at the project, and provided contract and bid documents for manufacturer bidding and installation. Bids were received by contractors in the summer of 2008, but did not meet project objectives, likely due to the expedited schedule.

The floodgate installation Phase I project was delayed at this point, plans, specifications and bid documents revised and reissued in the fall of 2008. At that time a qualified manufacturer was selected and the final design process initiated. The Phase I Floodgate Replacement was completed in October 2009.

Additional hydraulic analyses have been performed for the floodgate replacement. These are specific to evaluating floodgate performance of the manufacturer proposed gate, as well as additional scour design analyses.
- Performed modeling to document that flood conditions for the short term period between Phase I and Phase III completion, that operating floodgates at the 7.5ft and 9.5ft (NAVD88) positions would not have significant effects on flooding. For future conditions with the levee setback, significant improvements in flood storage would occur.
- Performed scour analyses to assess if scour protection materials were necessary at the sheetpile cutoff wall to which the floodgates are attached. It was determined that future scour conditions would not significantly exceed existing scour conditions as maximum velocities and scour conditions are a function of head differential that occurs from tidal exchange, and currently occurs at the site while the floodgates are lashed open in the summer time. Flood related velocities and scour are much less for the new floodgates due to lower velocities and deeper flow conditions during flood conditions.


The Fisher Slough Final Design Recommendations Report was developed for two reasons. The first was to collect supplemental data and perform additional studies (such as comprehensive geotechnical soils investigations) and the second was to address the outstanding design issues identified in the Preferred Restoration Plan. The outstanding issues were identified as follows:

- Identify the Big Ditch realignment and conduct a preliminary hydraulic analysis of proposed inverted siphon crossing structure to document changes in local drainage and gain approval from Drainage District #17 and local affected property owners.
- Conduct additional site characterization for soils testing and geotechnical investigations to determine soil suitability for reuse of materials for levee setback construction and floodplain grading schemes. No geotechnical or soil testing had been performed to date at the site.
- Evaluate different restoration strategies using either a passive or active approach for tidal marsh restoration. The passive approach includes minimal restoration activities such as levee setbacks, disposal of soils at off-site locations, and minimal Reed Canary Grass (RCG) management. Whereas the active approach would include additional construction work including pilot channels, floodplain grading, marsh and riparian plantings, installation of large woody debris and additional RCG control measures.
- Perform additional hydraulic modeling of floodgate and tidal marsh restoration alternatives to evaluate hydrologic, geomorphologic, vegetative response and expected tidal marsh habitat conditions for the passive and active restoration approaches.
- Evaluate and develop a RCG invasive species management plan for the passive and active restoration approaches, including marsh and riparian planting plans.
- Revise and update costs to support an alternatives analysis, as well as, ongoing grant application and fund acquisition processes.
- Identify and plan permit application process for project.

The outcome of the study report was development and agreement on all final design elements including the location of the Big Ditch Realignment and use of an inverted siphon crossing, a Reed Canary Grass
management strategy, and selection of specific restoration measures from the passive and active alternatives that best met the project objectives and budget.

This document also contains significant amount of environmental and engineering analyses and design to support final decision making for the project. Technical investigation information relative to the final design from the Fisher Slough Final Design Recommendations Report includes the following:

- Evaluation of passive and active alternatives for different vegetation patterns, potential for tidal channel network development periods and ponding conditions.
- Revised scour analysis and evaluation of inverted siphon crossing design depths and protective scour protection materials.
- Confirmation of flood storage volumes.
- Final evaluation of flow depths and emergency spillway conditions.
- Evaluation of levee hot spots need for erosion protection.

Study Document Availability
All of the documents developed for the project are available for download at the following FTP location:

ftp://swgguest:welcome2swg@ftp.tetratech.com/Outbound/Fisher_Slough/DOCUMENTS/

1.4 BASIS OF DESIGN REPORT CONTENTS
The contents of this report include detailed description of all project design elements, construction methods, sequencing and permit related information. The report also identifies critical design issues that were addressed in development of the final design. The objective of this basis of design report was twofold. The first was to document design assumptions and second to communicate with the project owners, partners and regulatory community regarding engineering and design issues. The contents of the report include the following topics:

- Construction Phasing and Sequencing
- Project Elements and Supporting Design Information
- Engineer’s Cost Estimate (Submitted under separate cover for privacy)
- Project Schedule
- Permits Overview
- Construction Contracting
- Real Estate, Easements, Agreements and Right of Way
- Supporting Appendices
  - Appendix A – Technical Design Memoranda
    - A.1 – Geotechnical Assessment Reports, Addenda and Design Memoranda
    - A.2 - Preliminary Temporary Erosion and Sediment Control Plan (TESC)
    - A.3 – Inverted Siphon Design Memoranda
    - A.4 – Temporary Crossing Design Memo
    - A.5 – Levee Design Memoranda
    - A.6 – LWD Design Memo
    - A.7 – Planting Plan Information
    - A.8 – Survey and Real Estate Information
    - A.9 – Well Report
  - Appendix B – Fisher Slough CAD Plans
  - Appendix C – Contract Specifications
Many of the appendices and technical memoranda are not included as paper copies in this report, rather they are electronic files submitted to The Nature Conservancy with this report as part of their records for the project.
Figure 1. Fisher Slough Project Location Map
2.0 PROJECT DESIGN

The project design encompasses the features identified in the Fisher Slough Final Design Recommendations Report (TNC, 2009a). One of the critical aspects of this project is the construction sequencing due to the limited work windows and significant number of water diversions and connections. In light of that, the project design elements are organized sequentially, within three distinct design phases corresponding to the overall construction sequence. This section of the report outlines and describes all associated project design elements that are to be constructed for the project (Figure 2).

2.1 CONSTRUCTION PHASES AND SEQUENCING

Project development and implementation is occurring in three distinct phases, namely:

- Phase I – Floodgate Replacement
- Phase II – Big Ditch Realignment & Levee Setback Pre-loading
- Phase III – Levee Setback Final Loading, Levee Removal & Tidal Marsh Restoration

Each of these phases of design and construction are further described herein.

2.1.1 Phase I – Floodgate Replacement

The Phase I – Floodgate Replacement design is complete, permits have been received and the floodgates were installed in August through October 2009.

2.1.2 Phase II – Big Ditch Realignment & Levee Setback Pre-loading

Phase II of the project is planned for construction in 2010. The project involves realigning Big Ditch to the west to consolidate with other crossing infrastructure, construct an inverted siphon crossing, perform levee setback pre-loading, regrade local irrigation drainage ditches, pre-excavate pilot channel and establish tributary realignments (Figure 3).

2.1.3 Phase III – Levee Setback Final Loading, Levee Removal & Tidal Marsh Restoration

Phase III of the project is planned for construction in 2011. The project involves final loading of the levee setback structure, tidal marsh restoration, pilot channel and tributary and main tidal channel realignment connections, landscaping and planting, and removal of the existing south levee and existing Big Ditch culvert crossing (Figure 4).

2.2 PHASE II & PHASE III PROJECT ELEMENTS

All project elements are shown on the construction general site plan (Figure 3 and Figure 4). The following section of the report describes each of the elements involved with project construction on the site.
Figure 2. Project Features Map
Figure 3. Phase II Project Elements

1. Phase II construction elements include:
   A. Perform all site preparation work as necessary.
   B. Realign Big Ditch to west to consolidate with other crossing infrastructure.
   C. Construct an Inverted Siphon.
   D. Perform levee setback pre-loading.
   E. Realign local irrigation drainage ditches.
   F. Pre-excavate pilot channel, main teals and tributary realignments.
   G. Grade 20,000 cy of topsoils on Junquedst farm area.
   H. Perform all site preparation work as necessary. This includes all teals, swimp, care and diversion of water, and general site set up work.
   I. Realign Big Ditch to western alignment salvage all topsoils for disposal and grading on Junquedst farm area. Secure all levee construction suitable materials for use in levee setback. Salvage all other materials for use in other project fill grading work. Connections of ditch to be performed within in-water work window Aug. 1 to Oct. 15, 2010.
   J. Construct Inverted Siphon realignment and profile on sheet C2.
   K. Contractor to provide construction plan for Inverted Siphon alignment and profile.
   L. Preliminary plan in this set of documents is not complete for construction all in-water work to be performed Aug. 1 to Oct. 15, 2010. Reconstruct levee sections of Inverted Siphon using levee materials, compaction and moisture content specifications per levee setback plans on C05.
   M. Clean and crush S. levee setback along entire levee realignment per specifications as shown on sheet C27. Upon completion of clearing alignment, perform inspection of cleared areas with TSC field engineers to document soil conditions. Depose of all large trees, mts tied and tracks along abandoned RR alignment.
   N. Pre-load S. levee setback realignment per specifications and details on sheet C27. Contractor required to install settlement plates and maintain to receive payments for soil settlement during construction per sheet C32.
   O. Do not fill or realign Big Ditch downstream from existing Big Ditch culvert until Phase 3 levee setback realignment and construction area complete. The existing Big Ditch culvert and downstream Big Ditch are the emergency overflow spillway for Fisher Slough levees.
   P. Perform realignment of drainage channel on southern side of Junquedst property running east-west along farm road.
   Q. Perform pre-excavation of all pilot channels, main teals, channel and tributary channel realignments in dry areas behind existing South Levee prior to its removal in Phase 3.
   R. Place dispose of 20,000 cy of topsoils to designated areas on Junquedst farm parcels P17468 & P17523. Salvage all other topsoils and excavated materials for use in other project fill and grading work.
Figure 4. Phase III Project Elements
2.2.1 Mobilization and Demobilization

For each phase of construction, the contractor will be mobilizing and demobilizing a variety of equipment. Some of the equipment may be very large and require temporary road control while delivering to the site. An example would be a crane for the inverted siphon pipe installation, or directional drilling and pipe pulling equipment. The contractor will be required to submit a list and schedule of all equipment to be mobilized and demobilized to the project site. This schedule and equipment list will be submitted by the contractor to The Nature Conservancy.

2.2.2 Site Preparation

2.2.2.1 Temporary Construction Facilities

The project will require temporary construction facilities including an office trailer and portable toilets. The office trailer will be the primary location for all communications, on-site meetings, and check-in point. The temporary project office will be located on the Smith A staging area.

Portable toilets will at a minimum be located at the project office and in each of the three staging areas. Additional facilities are at the discretion of the contractor.

There are no formal power or telephone connections at the site. The contractor will be responsible for their own power and telephone. Connections to local power and phone, or use of generators and cell phones are at the discretion of the contractor. The contractor will need to inform the owner of power and telephone plan prior to beginning construction.

2.2.2.2 Access, Haul Routes and Staging

There are five access routes to the project site:
- Jungquist North Access
- Jungquist South Access
- Smith A Access
- Smith B Access
- Sersland Access from Franklin Road (OPTION)

Each of the access areas will be gained from local roadways. A rock construction access pad will be built at each access point as an erosion control and water quality pollution protection measure. Rock access pads will be built using 6in thick, 1 1/2in minus base course subgrade and 3in thick of 5/8in minus (9in total thickness) road surfacing materials.

There are 3 major staging, stockpile areas for the project:
- Jungquist Staging/Stockpile Area
- Smith A Staging/Stockpile Area
- Smith B Staging/Stockpile Area

The staging and stockpile areas will be used for employee parking, equipment and materials storage, temporary restroom and office facilities, potential dewatering outlet areas and sedimentation ponds, and stockpiling and sorting of the various materials during the project.

Initially, a number of haul routes were proposed for the project using rock roadways. During 50% design review, it was suggested that the project specify the use of off-road, large wheel equipment for the project, so that rock haul routes would not be necessary. The recommendation was made to ensure that...
placed rock does not damage or limit future farm activities at the project site. Therefore, future haul routes will be cleared areas of bare soil, for which off-road large wheel equipment can travel during a range of soil wetness conditions. Other vehicles may have times/periods when travel on the haul routes is difficult.

A number of permanent and one temporary crossings are shown in the haul plan. These structures are installed towards the completion of the Big Ditch Realignment, and are discussed in the following related Big Ditch crossings sections of the report. A temporary crossing is currently proposed across Fisher Slough at the existing Big Ditch Culvert crossing to reduce the amount of vehicle traffic accessing Pioneer Highway.

In addition to the permanent crossings shown in the plan, the contractor will likely need to install a number of temporary crossings to allow irrigation drainage and stream crossings during the work. These crossings will likely be to allow for agricultural drainage to continue during construction. A typical temporary crossing installation would include a 24” pipe spanning through the 12’ wide haul route and fill material laid at a 2:1 slope in the crossing channel, which are typically 3’ deep.

Also, a temporary crossing of Fisher Slough has been included in the project and is discussed further in the Bridge Crossing Section 2.2.2.4. (App. A.4). This structure is a temporary measure to allow for the contractor to travel north and south of Fisher Slough without entering the Pioneer Highway.

The road surface of the temporary crossings (excluding the temporary Fisher Slough Bridge Crossing) is the same as other haul routes. All temporary crossings will be removed at the end of the project. It is estimated that up to 5 temporary crossings may need to be constructed during the project.

Three staging areas will be used to support the project, including a 1.21 acre area on the Jungquist property, 2.08 acres on the Smith A property, and 1.03 acres on the Smith B property. These staging areas will be used for the temporary construction facilities locations, construction employee parking, stockpile and store soils, vegetation clearing, grubbing materials, vehicles, equipment and other project related materials for project construction. These areas will have a stabilized rock entrance, and be primarily exposed soil surfaces without placement of rock. Standard Temporary Erosion and Sediment Control (TESC) facilities will be used to manage drainage and stormwater from the staging areas, including silt fences around the entire staging/stockpile areas. All rock, TESC structures and staging materials will be completely removed from these areas upon completion. It is important to fully remove all rock from farm areas after construction.

For the Smith A area, an electrical utility connection to a contractor trailer and owner trailer will be necessary. This area should be enclosed by a temporary protective (security) fence 100ft square that can provide security and protection from vandalism including 2 trailers and a number of vehicles and equipment.

2.2.2.3 Soils Management & Stockpiling

Soils management during construction is a key concern for the project owner and their partners. There are multiple design objectives involved with soils management.

- Clearing and grubbing materials will be stockpiled on site for recycling with top soil materials. It is important to keep the clearing and grubbing materials separate from those materials being reused in the levee setback project to minimize invasive species such as RCG and Himalayan Blackberry distribution.
• Stockpiling and recycling top soil and making materials available for haul and grading on adjacent farm areas on property north of the Slough.
• Stockpiling and recycling suitable soils for reuse in Big Ditch fill and Levee Setback construction.
• Excavation, removal and disposal of unsuitable soils in ditch and channel fill areas, and at an off-site location for excess cut.
• Provide direct hauling transfers and minimize stockpiling and double handling of material.

The contractor will be required to manage these separate types of soil, document the soil conditions, locations and amounts as part of the measurement and payment system for the work.

Clearing and grubbing will be required along certain sections of the project where vegetation is growing in the construction areas. The contractor will clear, grub, and stockpile these materials for use as recycled materials to be made available to local farmers approved by TNC. It is highly important to create a physical barrier between clearing and grubbing materials, and recycled suitable levee materials to limit the potential for invasive species establishment on the new levee setback structure. Stockpiling at separate locations for topsoil, clearing and grubbing materials, versus suitable levee setback materials, would be the most effective method for keeping these soil materials separate. Topsoil and clearing and grubbing materials will be stored at the Jungquist staging area, and suitable levee setback soils will be stored in the Smith A and Smith B staging areas.

All suitable topsoil materials will be located and marked on site for pickup by local farmers identified by The Nature Conservancy in the contract specifications. The contractor will be paid by TNC for stripping and removal, transfer and stockpiling to the staging areas. The contractor will then also be paid as a separate item for loading recycled material into the approved farmers haul trucks. The contractor will keep a record of the individual/entity, pickup time and volume of material, as measured by bulk volume (cubic yards) as loaded into haul dump trucks.

2.2.2.4 Temporary Erosion and Sediment Control Plans
Temporary Erosion and Sediment Controls (TESC) will be necessary for the project. The design and implementation of TESC measures are ultimately the responsibility of the contractor. The specifications include a contractor submittal of a Washington Department of Ecology, Construction General Stormwater Permit, Stormwater Erosion, Pollution Prevention Plan (SWPPP). The plan will outline the construction work items, potential erosion and pollution that could be associated with the work, and the BMPs that will be used to perform the work, for approval by TNC and project regulators as necessary. The submittal of this document will likely be needed to satisfy the regulatory requirements for release of final permits (namely the 401 permit).

A preliminary Temporary Erosion and Sediment Control Plan was submitted as part of the Skagit County grading permit (App. A.2), and will be expanded by the Contractor as necessary for the General Construction Stormwater Permit application and associated SWPPP.

The following discussion outlines the most likely TESC measures and water diversions that will be encountered during project construction, and engineering recommendations to TNC and the contractor on how to approach this work.

The intent of using TESC protective measures is to protect the water bodies of the State of Washington. There are a number of standard TESC measures and best management practices (BMPs), including silt fencing, straw bales, plastic covers, channel rock grade control, and hydrosed mulching, among others, that when used appropriately can limit the erosion and transport of soil materials to receiving water bodies.
of the state. In addition there are BMPs for preventing pollution from heavy construction equipment. These BMPs may include oil spill clean up kits, use of fish friendly hydraulic fluids and cleaning and washing of construction equipment, etc.

The most probable sources of erosion and water quality pollution are as follows:

- Erosion of new excavated channel bed and banks and sedimentation in downstream aquatic water bodies
- Tracking of soils from construction equipment off the site onto the roadway
- Wind erosion and transport of soils from large stockpile areas
- Breaking of heavy equipment hydraulic lines during construction
- Brief pulses of turbidity during re-watering and reconnecting of channels
- Erosion of existing south levee during removal
- Turbidity during installation and removal of temporary streams side and in water construction structures
- Natural erosion of pilot channels and tributary channels post construction

The plan for construction of new channel bed and banks is to work in dry areas between channel connections. For example, the Big Ditch realignment would be fully constructed and then connections will be made at the downstream and then upstream ends of the channel. If erosion occurs from localized rilling, gullying or slumping of material along the realigned channel during construction there is no potential transport of eroded material downstream as the channel is not connected. Construction sequencing is key to controlling erosion and involves first excavating the channel and then slowly connecting and watering the new channel. Watering the channel slowly, versus fully breaching the connection is recommended for all connections. These are described further below.

Logical areas to install erosion control BMPs would be along the margins of active channel areas exposed to channel flow paths. A majority of the construction work will be performed in the dry. Silt fences along open channel cuts should not be required, if there is no direct flow pathway to a receiving water body. Also, exposed topsoil surfaces are common in the Skagit Valley farm areas.

Tracking of soils from construction equipment off the site onto the roadway is very probable at the project site. The contractor will need to continually maintain the access rock pads to make sure that equipment traveling off of the site has soils removed from the tires using the rock pad entrance/exit.

Wind erosion and transport of soils from exposed cuts and stockpile areas is likely one of the larger potential soil erosion mechanisms in the project. Typically in urban areas, stockpiles of soil must be managed with plastic sheeting and weighted lines to keep the plastic in place. Other soil stabilization measures may include hydroseeding, and tackifiers. In agricultural areas, wind erosion of soils can be a more frequent occurrence due to higher percentage of exposed topsoils. It remains to be determined what measures will be required for soil stockpiling. This will likely be defined through the permit process and will likely be a part of the Ecology 401 certification which may specify the need for a construction stormwater permit.

Using heavy construction equipment in and around aquatic environments, there will be the risk for breaking of a hydraulic line leaking of hydraulic fluids. One prescribed method is to specify fish friendly hydraulic fluids to be used on all equipment with hydraulic lines. Another source of equipment related pollution are the grease and oils that can be washed off of the equipment into the project area. The BMP for managing this pollution is typically requiring the contractor to steam clean all vehicles prior to mobilizing to the site, and requiring proof of cleaning.
Another equipment related item is to require the use of large wheel, off-road equipment, rather than installing several thousand feet of crushed rock haul route surfaces. The use of haul routes along the margins of the agricultural areas is not allowed by the farm owners as the rock will degrade farm soil conditions.

It should be expected that for all channel connections and rewatering work that short intermittent turbidity plumes will be experienced. If managed properly rewatering can be limited to short durations. However, it is likely that in reality the rewatering activities will exceed regulatory turbidity standards, for short durations (on the order of hours). Exceedances for long durations (on the order of days) are not acceptable.

Another construction activity that will likely cause turbid conditions is the installation of temporary structures during construction. Two prime examples are the inverted siphon crossing and the existing Big Ditch crossing removal. Both of these will likely require in-water structures to be installed in order to build and demolish these structures. For the inverted siphon structure, an option under consideration is the use of directional drilling and pipe pulling underneath the slough. This would eliminate the potential for water quality pollution, but the cost is not yet known.

Removal of the existing levee will likely be one of the most persistent causes of turbidity. Construction crews will work during low tides and flow levels to remove the existing levee and haul routes. However, during each tidal cycle, these freshly excavated areas will be exposed to tidal flows and will likely cause intermittent turbidity plumes. One strategy for minimizing this is to remove the upper portions of the levee first, down to an elevation above the known high tide for the construction period. Then, the contractor can remove lower portions of the levee during low tide conditions. The strategy does not completely eliminate erosion and turbidity potential, but does reduce the potential erosion.

A likely source of erosion is the natural development of the pilot tidal channels. These features were evaluated in the Fisher Slough Final Recommendations report and it was shown that the construction of the pilot channels has the potential to greatly reduce the amount of time for tidal channel development in the marsh. These features will be pre-excavated and allowed to erode and develop over time. It is expected that 5,000CY of material will be eroded into the downstream channels, over a 10 year period, and deposited as fine sands and bar features, with a portion of this being transported beyond the project site. Beyond the initial channel establishment period, the marsh will become a sediment sink. The gradual erosion of these features is not likely to create exceedances of water quality standards.

Regulatory standards require turbidity levels downstream from the construction work at the point of compliance to stay below 5NTUs over background turbidity if background is less than 50NTU, or a maximum of 10% increase if background is over 50NTU. Monitoring will likely be required during construction and the contractor should have a certified Construction Stormwater Certified Professional as part of the construction and engineering team.

Overall, the TESC has several design items that need final design, management and implementation by the construction contractor. These contractor TESC and water diversion design, management and implementation requirements are defined in the specifications, for which the contractor will need to provide supplemental design and installation information to finalize project permits.

### 2.2.2.5 Water Diversions and Reconnections

A care and diversion of water plan will be required for the project. This plan will identify all water diversions and connections that will occur during the project. Ultimately, the contractor will need to define these activities, as the number and type may vary depending upon the selected construction method. A good example would be the option to use directional drilling and pipe pulling instead of in-
The Nature Conservancy  
Fisher Slough – Final Design and Permitting  

water sheet pile installation and trench excavation for the inverted siphon installation. The following list is the most likely water diversions and connections that will occur during the project (Figure 5):

- Full tidal exclusion, with tributary inflow pumping to downstream Fisher Slough, during both Phase 2 and Phase 3 construction
- Inverted siphon installation – in-water trench excavation, isolation only, no rewatering
- Big Ditch downstream connection
- Big Ditch upstream connection
- Existing Big Ditch crossing demolition – tributary diversion
- Jungquist drainage ditch regrade
- Main tidal channel downstream connection
- Main tidal channel upstream connection
- Levee Setback Removal and Pilot Channel Breaches
- Pilot Channel and Tributary Channel Connections
- Big Fisher Creek Realignment
- Little Fisher Creek Realignment

The construction contractor will need to clearly identify all water diversion locations, schedule and timing, management and monitoring activities that will occur during the project. Several water diversion methods will likely be used during the work. These include:

- Floating silt curtains for excavations along shoreline areas
- Portable hydraulic, K-dams or sandbag systems to isolate shoreline excavation areas
- Pipe diversions through construction areas
- Sheet pile and cutoff walls for major in-water excavations
- Bank and levee breaching for channel reconnections

The care and diversion of water plan, like the TESC plan, will be a required submittal where the construction contractor demonstrates an understanding of the water management issues during construction, and the corresponding plan.

One key design assumption that has been incorporated into the project is for full tidal exclusion and tributary inflow pumping during construction. The approach will be during final Phase III Levee Removal, Tributary and Pilot Channel reconnections to exclude tidal inflow to the site by fixing closed the floodgates. Tributary inflow from Hill Ditch, Big and Little Fisher Creeks will use a pump, fish screen system to convey tributary inflow downstream of the floodgates into Fisher Slough. A preliminary care and diversion of water plan is provided for initial permit and bidding purposes. The plan will need to be finalized by the construction contractor showing the pump withdrawal and return locations, and details for fish screening and exclusion during construction, as well as a schedule of operations (Figure 5). This plan will be subject to review for final permit approval.
Figure 5. Water Control, Crossings, Diversion and Return Points
2.2.2.6 Clearing & Grubbing
Clearing and grubbing of vegetation and trees will be required for the project. Haul routes, channel realignment, levee setback areas will be cleared and grubbed as necessary. One of the goals of the project is to maintain existing vegetation as much as possible in the tidal marsh restoration areas. The rationale behind this is that the vegetation will limit excessive erosion during levee removal and tidal channel development, as well as reduce the likelihood for invasive species establishment. The plans will show interior marsh areas as “vegetation preservation areas”.

Materials gained through clearing and grubbing activities can be disposed of off site, or the contractor can chip and shred the materials for use on the site as part of the landscaping and planting plans, or stockpiling and removal by others. At a minimum, blackberry and reed canary grass materials should be disposed of off-site. Trees and other clean vegetation can be used as chipped and composted materials. All materials shall be used or disposed of at the end of the project, no stockpiles shall remain.

One unique aspect of the clearing and grubbing work is that there is an interest in salvaging the large cottonwood and poplar trees growing on the abandoned railroad grade and other clearing areas. These trees may, or may not be used for habitat features in the Fisher Slough marsh restoration areas. If the project sponsor decides not to use the trees, the large wood debris will be available for other restoration projects. If there are no interested parties, the contractor will be asked to remove and dispose of the trees.

2.2.2.7 Rough Grading
The contractor will likely need to rework grades in several areas to gain access for equipment and vehicles, such as local ingress ramps onto the existing levee. The plans do not specify how or where these rough grading activities will be performed. The plans will however identify those construction work zone areas where vegetation will be preserved.

2.2.3 Big Ditch Realignment
The Big Ditch Realignment is a key element of the Phase II construction effort. The alignment of this structure is critical to the project for many reasons. These include the need to remove the levee and channel constriction through the middle of the project site that causes adverse sedimentation, levee erosion and backwater flooding. Also, the existing Big Ditch crossing structure is a fish passage barrier during low flow and low tide conditions. Upstream of this structure during these conditions, it has also been documented that the structure causes localized increases in water temperatures due to ponding conditions. Finally, realigning Big Ditch to the west along the Pioneer Highway consolidates major infrastructure crossings. These linear features create habitat fragmentation, and combining them limit watershed and ecosystem scalar effects and impacts from major transportation, utility and water infrastructure that bisects naturally occurring ecosystems.

2.2.3.1 Big Ditch Drainage Hydrology and Hydraulics
The Big Ditch drainage channel drains minor portions of Mt. Vernon, and the agricultural areas between Hill Ditch and the Skagit River. The extents of the drainage system are from Mt. Vernon to Stanwood. In general, the Big Ditch drainage system drains the areas within the protective agricultural levees, especially to the north of Fisher Slough. To the south towards Stanwood, the Big Ditch drainage system begins to collect small tributary flow from the hill areas between the South Fork Skagit and Interstate 5. The drainage ditch is 10.5 miles long in total as measured from the south edge of Mt. Vernon to the Skagit Bay outlet north of Stanwood. The drainage area to Big Ditch is estimated at 15.7 square miles and just more than 10,000 acres in total. The watershed drainage area for Big Ditch at the Fisher Slough crossing is 10.8 square miles.
The hydrology of the Big Ditch was evaluated for a range of flow conditions. A discharge measurement of 8.7cfs was made in the Fall of 2008. This represents typical low flow conditions in the ditch that occur in the late summer and early fall periods. The second flow rate evaluated was the fish passage design flow. The rate was determined using the WDFW Fish Passage Design Guidelines for culvert replacements. The estimated fish passage design flow rate for the project is 63.1cfs. This discharge is used to assess fish passage conditions for the proposed structure. In addition, the channel capacity was evaluated using a HEC-RAS model in the project study area. The estimated channel capacity (bankfull discharge) is 80.0cfs, which is the limiting factor in sizing of the new inverted siphon crossing structure. A flood capacity estimate was also evaluated using the HEC-RAS model for the existing, 6 box sag culvert. The evaluation involved establishing flood water surface elevations equivalent to the 100-year flood (el. = 16.7ft) on the upstream side of the culvert, and then determining the maximum discharge capacity through the existing culvert. This analysis estimated a flood capacity through the existing culverts of 235.0cfs with 1.3ft of freeboard on the Fisher Slough north levee (App. A.3).

Two major conclusions have been drawn from the initial hydrologic investigations of Big Ditch. First, the channel capacity is 80cfs and a significant limiting factor in the flood conveyance of the Big Ditch system. Second, the Big Ditch existing culvert crossing has a capacity of 235cfs, which is roughly the 25-year flood event.

Multiple methods were used to estimate the range of flood frequency events including the 2, 10, 25, 50 and 100-year flood events. The first method used the USGS regional regression (USGS, 2001) equations were used to estimate the range of flood flows along Big Ditch (Table 1). A second method for evaluating flood frequencies used scaling of the Fisher Slough tributary attenuated peak flow estimates from the original flood study (TNC, 2007a). These peak flow estimates account for fairly significant attenuation that occurs along the relatively flat areas of the Skagit River Delta. Overall, the flow rates match fairly well, with the exception of the Q10 event. Select discharges including the Q25 and Q100 flows using the Carpenter Creek scaled discharge methods were used in evaluating Big Ditch existing and proposed flood hydraulics due to the likelihood that overbank flows and attenuation are occurring along the Big Ditch system.
Table 1. Big Ditch Proposed Inverted Siphon Hydrology and Hydraulics

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<th>Q&lt;sub&gt;BD&lt;/sub&gt; (cfs)</th>
<th>Q&lt;sub&gt;FisherTribs&lt;/sub&gt; (cfs)</th>
<th>Q&lt;sub&gt;BD~&lt;/sub&gt; Q&lt;sub&gt;CARP&lt;/sub&gt;(A&lt;sub&gt;BD&lt;/sub&gt;/A&lt;sub&gt;FisherTribs&lt;/sub&gt;) (cfs)</th>
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</table>

A comparison analysis of existing and proposed culver/siphon hydraulics was performed. The analysis used a HEC-RAS model of the existing channel and culvert system. The analysis of the proposed inverted siphon used a HEC-RAS model along the downstream channel section and observed water surface profile data for the slope input for normal depth boundary conditions. Inverted siphon losses were estimated using engineering equations provided by the U.S. Bureau of Reclamation, 1978. Design of Small Canal Structures.

Tables 2 through 4 summarize the flow rates for the existing culvert, inverted siphon, and comparison of the two conditions. Overall, the inverted siphon significantly outperforms the existing structure. For example, the proposed inverted siphon has additional capacity beyond the existing culvert. For the 235.0cfs flow, the upstream water surface elevation is predicted to be 3.5ft lower than current conditions. The inverted siphon also has the capacity to convey the 100-year flood event of 324.0cfs with 5.8ft of freeboard on the north levee. The proposed capacity of the new culvert is ~400cfs (at upstream water surface elevation of 16.7ft), which is 1.7 times the existing capacity, and is greater than the 100-year runoff event for Big Ditch. The additional capacity will likely be exceeded at some point in the future as Skagit River flooding into the Big Ditch drainage area due to levee overtopping and flooding will likely occur in the future.

An additional element to the analysis is the fish passage flow where pipe velocities meet the fish passage design criteria of 2.0fps for juvenile salmonid and adult cutthroat. In addition, the submerged pipe system will be fully inundated at nearly all flow conditions, and thereby meet fish passage depth criteria. Additional fish passage discussions are included in the inverted siphon slide gate design discussion whereby fish passage is provided through a slot in the gate.

The proposed siphon is a significant improvement over the existing culvert from a fish passage and flood conveyance perspective.
### Table 2. Big Ditch Existing Culvert Hydrology and Hydraulics

<table>
<thead>
<tr>
<th>Design Discharge</th>
<th>Flow Rate (cfs)</th>
<th>No. Culverts</th>
<th>Culvert Flow Depth</th>
<th>WSE U/S Culverts</th>
<th>Culvert Velocity (fps)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q Low Flow</td>
<td>8.7</td>
<td>6</td>
<td>0.9</td>
<td>3.7</td>
<td>0.4</td>
</tr>
<tr>
<td>Q Fish Passage</td>
<td>63.1</td>
<td>6</td>
<td>2.4</td>
<td>5.6</td>
<td>1.0</td>
</tr>
<tr>
<td>Q Channel Capacity</td>
<td>80.0</td>
<td>6</td>
<td>2.7</td>
<td>5.9</td>
<td>1.1</td>
</tr>
<tr>
<td>Q Exist Culvert Capacity @ 16.7ft</td>
<td>235.0</td>
<td>6</td>
<td>4.5</td>
<td>13.6</td>
<td>1.9</td>
</tr>
</tbody>
</table>

### Table 3. Big Ditch Proposed Inverted Siphon Hydrology and Hydraulics

<table>
<thead>
<tr>
<th>Design Discharge</th>
<th>Channel Flow Rate (cfs)</th>
<th>Siphon Losses (ft)</th>
<th>No. of Pipes</th>
<th>Pipe Dia. (ft)</th>
<th>WSE D/S Inverted Siphon Pipes</th>
<th>WSE U/S Inverted Siphon Pipes</th>
<th>Pipe Velocity (fps)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q Low Flow</td>
<td>8.7</td>
<td>0.0028</td>
<td>2</td>
<td>4.5</td>
<td>2.9</td>
<td>2.9</td>
<td>0.3</td>
</tr>
<tr>
<td>Q Fish Passage</td>
<td>63.1</td>
<td>0.1741</td>
<td>2</td>
<td>4.5</td>
<td>4.8</td>
<td>5.0</td>
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<tr>
<td>Q Channel Capacity</td>
<td>80.0</td>
<td>0.2841</td>
<td>2</td>
<td>4.5</td>
<td>5.2</td>
<td>5.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Q Exist Culvert Capacity @ Q25</td>
<td>235.0</td>
<td>2.5267</td>
<td>2</td>
<td>4.5</td>
<td>7.6</td>
<td>10.1</td>
<td>7.4</td>
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<tr>
<td>Q 100</td>
<td>324.0</td>
<td>3.5308</td>
<td>2</td>
<td>4.5</td>
<td>8.7</td>
<td>12.2</td>
<td>14.3</td>
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</table>

### Table 4. Comparison of Existing and Proposed Big Ditch Inverted Siphon Hydrology and Hydraulics

<table>
<thead>
<tr>
<th>WSE U/S CULVERT/SIPHON (FT)</th>
<th>Flow Rate (cfs)</th>
<th>Existing Culvert</th>
<th>Proposed Inverted Siphon</th>
<th>Difference Proposed-Existing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8.7</td>
<td>3.7</td>
<td>2.9</td>
<td>-0.3</td>
</tr>
<tr>
<td></td>
<td>63.1</td>
<td>5.6</td>
<td>5.0</td>
<td>-0.6</td>
</tr>
<tr>
<td></td>
<td>80.0</td>
<td>5.9</td>
<td>5.5</td>
<td>-0.4</td>
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<tr>
<td></td>
<td>235.0</td>
<td>13.6</td>
<td>10.1</td>
<td>-3.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CULVERT/SIPHON VEL. (FPS)</th>
<th>Flow Rate (cfs)</th>
<th>Existing Culvert</th>
<th>Proposed Inverted Siphon</th>
<th>Difference Proposed-Existing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8.7</td>
<td>0.4</td>
<td>0.3</td>
<td>-0.1</td>
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<tr>
<td></td>
<td>63.1</td>
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<td>2.0</td>
<td>1.0</td>
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<tr>
<td></td>
<td>80.0</td>
<td>1.1</td>
<td>2.5</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>235.0</td>
<td>1.9</td>
<td>7.4</td>
<td>5.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CULVERT DEPTH (FT)</th>
<th>Flow Rate (cfs)</th>
<th>Existing Culvert</th>
<th>Proposed Inverted Siphon</th>
<th>Difference Proposed-Existing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8.7</td>
<td>0.9</td>
<td>4.5</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>63.1</td>
<td>2.4</td>
<td>4.5</td>
<td>2.1</td>
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<td></td>
<td>80.0</td>
<td>2.7</td>
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</tr>
<tr>
<td></td>
<td>235.0</td>
<td>4.5</td>
<td>4.5</td>
<td>0.0</td>
</tr>
</tbody>
</table>
The project elements for the Big Ditch Realignment include the following items:

- Channel Alignment, Profile and Section Geometry
- Inverted Siphon, Fisher Slough Crossing
- Big Ditch and Other Project Channel Crossings
- Existing Big Ditch Channel Fill and Regrade
- Channel Connections
- Local Farm Drainage Channel Regrade
- Existing Big Ditch Fisher Slough Culvert Crossing Demolition

### 2.2.3.2 Channel alignment, profile, section geometry

The Big Ditch Channel realignment is a 4,100ft long realignment (Figure 6). The realignment begins in the northwest corner of the Jungquist property north of the Fisher Slough site at parcel numbers P16854. The channel is diverted to the west with a 200ft radius curve, which is greater than the 2.5 times the channel width of 60ft ($R_c \geq 160ft$) per the Corps channel design guidelines (USACE, 1994) to prevent erosion from flow separation and helical flow. It is noted that Big Ditch is a low energy environment and that helical flow conditions on the outside of the bend are likely negligible, thereby limiting erosion resulting from too tight of radius of curvature.

The Ditch then travels in a westerly direction towards the Pioneer Highway, owned by Skagit County. The Ditch turns southward, with another 200ft radius curve, and travels adjacent to the Pioneer Highway. There is a 20ft offset between the new channel and the Pioneer Highway road embankment. This configuration will prevent settling of the roadway embankment due to construction, and long term maintenance access along the west side of the ditch.

Between station 20+00 and 23+00 the realignment crosses beneath Fisher Slough. The question is often asked why Big Ditch cannot drain into Fisher Slough and why do the water bodies cross paths? The irrigation drainage infrastructure (Big Ditch) was graded lower than all other surrounding farm areas and channels to drain the land and make it viable for farming. The invert of Big Ditch is currently 4ft lower than Fisher Slough at the point of the crossing. If the two water bodies were combined, levees would need to be extended upstream along both sides of Big Ditch to protect farm areas for several miles, similar to the levees that currently bound Fisher Slough and upstream Hill Ditch areas. 1,000ft downstream from the crossing, at Station 10+00, Big Ditch will reconnect with the existing channel where it currently turns from a westerly direction to the south, just south of the Smith A access point.
The profile of the channel is flat, for both the existing and proposed condition (Figure 7). The bed of the channel is located at an elevation 0.0ft NAVD88. The primary difference in the profile is that the existing channel has areas with sedimentation upstream from the existing crossing, and the crossing itself is elevated above the channel bed at 2.7ft. The proposed condition will remove these raised portions of the profile, and the inverted siphon will be buried to a depth of -11.0ft NAVD88 at the bottom of the pipe below Fisher Slough. This will provide adequate cover and scour protection for the pipes.

The channel cross section design uses channel dimensions similar to existing conditions with a typical 60ft top width, 2.5:1 side slopes, and an average depth from channel invert to top of bank of 6.0ft (Figures 8-9). The side slope angle recommendations are those made in the original Geotechnical Engineering report (TNC, 2009). Compaction requirements of the Big Ditch banks are 90% of maximum dry density per ASTM D-698, standard Proctor method. The upper portions of the Big Ditch banks (above 3.0ft NAVD88) will be hydroseeded upon completion.

The current estimate for earthwork excavation is 29,000CY of materials to be removed (cut) from the Big Ditch realignment channel. This material will be stockpiled in the Jungquist and Smith A staging areas, and along the margins of the proposed existing Big Ditch fill area. The excavated materials will be separated into topsoil and suitable subsurface levee fill materials. Topsoils will be hauled, graded and tilled into the Jungquist farm areas. The remainder of suitable materials will be used as pre-loading material for the levee setback structure. Unsuitable materials will be used in fill/grading activities throughout the project site.

The area of excavation is 4.4 acres of land. The right of way has a 40ft access buffer on either side of the ditch during construction (30ft for equipment, 10ft for topsoil storage) except along the west edge of the channel along the Pioneer Highway. The combined channel and construction easement area is 6.5 acres.
Upon approval of the alignment and crossing, legal descriptions and final acreages will be provided in the design plans. Preliminary acreages by parcel are described in the real estate section of the report.

### 2.2.3.3 Inverted siphon Fisher Slough crossing

The primary design feature for the Big Ditch realignment is the inverted siphon crossing. This feature will replace the existing sag culvert crossing located in the middle section of the Fisher Slough project area. The inverted siphon has several design elements that have been incorporated through comments, review and input by Drainage District No. 17, Dike District No 3, and WDFW (Figure 10-13). The inverted siphon design elements are described as follows from upstream to downstream.

- Sedimentation Basin & Sill
- Inlet and Outlet Transition Structures
- (2) 54” Coated Aluminum Vertical Slide Gates w/ Fish Passage Slot
- (2) 54” HDPE Inverted Siphon Pipes Beneath Fisher Slough

In addition to presenting the design basis for the inverted siphon structure elements, a detailed review of possible construction methods was performed to evaluate the most feasible methods of construction.

#### Sedimentation Basin & Sill

The sedimentation basin consists of a concrete cutoff wall sill at the head of the basin, with a 30ft long by 3ft-4in deep, by 15ft wide (less the width of the basin walls 18in) structure, with a volume of 56CY. The depressed section of the channel has an 18in concrete floor slab, for which sediment will deposit in the bottom of the sedimentation basin. The sedimentation basin will limit the amount of maintenance required to clear sediment from the pipes. The banks of the sediment basin are designed as earthen channel that match the top of the sediment basin short retaining walls. The earthen banks connect to a 15ft bottom width and 2.5:1 sideslope, to match the inverted siphon inlet bankline slope angles.

A SAMWin v1.0 sediment yield analysis was performed to size the sedimentation basin. The yield was estimated using the flood return interval flows including the 2, 10, 25, 50 and 100-year events. The discharges were then run in the Big Ditch HEC-RAS hydraulic model to obtain the hydraulic characteristics at the upstream end of the siphon for each of the flood flow frequencies and entered in the SAMwin program. A simple sediment gradation was developed assuming the channel consists of silts and sands based on our observations of bed conditions during hydrologic data sampling and cross section surveys. The sediment gradation consisted of 50% of the material ranging between 2mm – 0.05 mm (sand) and 50% <0.05 mm (silt) with maximum size particle size of 2mm. The Meyer-Peter-Mueller (MPM) 1948 sediment transport function was used to develop sediment transport rates and yield for Big Ditch.

The SAMwin sediment yield analysis calculated an annual sediment yield upstream of the siphon of 12 tons of material. The volume can be highly variable depending upon sediment density of the depositional material. Assuming a sediment deposition weight of 100lb/cf, a ratio of 0.61 tons/CY was used to estimate 7.3CY of material would be deposited in the basin on an annual basis. A safety factor of 2.0 was added to the sediment yield due to the uncertainty in the assumptions used resulting in a require basin size of 15CY. The sedimentation basin depth was increased, based on comments from the 50% design review. The depth of the basin was increased so that the bottom of the sediment basin floor slab was even with the bottom of the siphon inlet cutoff wall. The revised depth of the sediment basin is 3ft-4in, with a total sediment storage capacity of 56CY, more than 7 times the estimated volume of sediment (Figures 9-11).

Also during 50% design review, Drainage District 17 staff indicated that this reach of Big Ditch does not have significant or adverse sedimentation rates. This would tend to indicate that the proposed, sediment basin, which is more than 7 times the annual yield, is more than adequate. If costs become prohibitive in
this project, the basin could be shortened to save money. It is noted that the estimate is fairly simplistic in
the methods used to determine the annual sediment yield, and additional sedimentation capacity is
warranted to account for uncertainties in the design analysis.

The sediment basin will need to be cleaned annually to prevent sediment inflow to the siphon. The
sediment basin can be cleaned using a backhoe with smooth bucket (without teeth) and/or a vactor truck
to suction out the deposits. Drainage District 17 will be required to keep records of the amount of
sediment removed on an annual basis and adapt maintenance cleaning of the sediment basin per actual
site conditions, and report this information to WDFW per their drainage maintenance agreement (WDFW,
2005).

**Inlet and Outlet Transition Structures**
The next major feature within the inverted siphon intake is the inlet structure. This structure is made from
concrete and rebar and has the following design elements.

- Slanted floor slab to transition from sedimentation basin to pipe invert. Pipe is 6in above flood
  slab to allow for vertical slidegate connection and seating
- Slope angled slabs between headwall/wingwalls and the floor slab to provide efficient
  hydraulic transition
- Upstream trash rack affixed to inlet headwall and floors
- Vertical wingwalls
- Vertical 16.5ft headwall
- (2) 54in vertical gates affixed to headwall
- (2) 54in HDPE pipe penetrations through headwall

Design standards from the USBR Design of Small Canal Structures (USBR, 1978) and the Hydraulic
Engineering text (Roberson, 1995) for transition angles were referenced for designing the intake, pipe and
outlet structure. Minor modifications were made on the standard inverted siphon inlet/outlet structures, as
shown in the USBR manual, were made to allow for placement of vertical concrete wall and floor
elements, and hence ease of design and construction.

At the upstream end of the structure, there is a 4ft deep (and wide along the edges of the walls) cutoff wall
designed to prevent underseepage and piping of subsurface material along the concrete and soil interface.
The cutoff is actually the final wall in the sedimentation basin. The crest of the cutoff wall (invert of the
channel) is located even with the bed of the channel and inlet floor slab at 0.0ft. Downstream from the
invert of the cutoff wall the concrete channel transitions along the inlet structure floor slab and to the
headwall and siphon pipe penetrations.

The headwall of the structure is 16.5ft tall and 18in thick. At the top of the wall, there is a gravel pathway
along the margins of the headwall. An aluminum catwalk with guardrails spans the inlet structure to allow
for access and operation of the 54inch vertical slide gates.

The floors/foundation, cutoff, wingwalls and headwalls are designed with reinforcing steel using the
USBR and other headwall/wingwall design standards, as well as transportation standards for headwall and
pipe bar size and spacing. No. 4 and No. 6 bar, with 12in spacing are located throughout the structure.
The concrete and rebar inlet (and outlet) structures will be built using cast-in-place concrete construction
methods. It if often preferable to have pre-cast concrete structures delivered to the site. It may be the case
that the headwall panels could be pre-cast and delivered to the site. However, due to the specificity in the
design for the structure for hydraulic efficiency, fish passage and operations and maintenance, it is not
likely that a pre-cast solution is viable.
A trash rack will be installed to limit the amount of debris in the channel that could be entrained in the pipes. The trash rack is a custom design, that uses ~(60) 1/2in x 2in x 4in flat bar (grate), turned on edge and positioned a 6in spacing on center. A series of (3) flat bar cross members will be welded to the flat bar grate to provide structural rigidity. The trash rack grate bars will be spot welded to a 1” x 6” metal anchor plate flush mounted to the inverted siphon inlet floor slab and slanted floors using epoxy anchors and mollys. The top of the flat bar grate will be mounted to an angle iron affixed to the cat-walk. A series of 3ft by 4ft openings will be spaced along the bottom of the rack to allow for fish passage through the structure. These openings correspond with the approximate observed low water condition in Big Ditch and should remain submerged and not allow transport of debris into the structure. The contractor will need to finalize and field fit the customized trash rack design.
Figure 7. Big Ditch Proposed and Existing Profile
Figure 8. Big Ditch Typical Section A

Figure 9. Big Ditch Typical Section C
Figure 10. Inverted Siphon Sedimentation Basin and Inlet Isometric
Figure 11. Inverted Siphon Inlet (and Outlet) Plan Detail
(2) 54in Vertical Slide Gates
Two 54”, round, vertical slide gates will be affixed to the headwall (Figure 12). These gates will remain raised and in the fully open condition during much of the year. During low flow irrigation period operations, the gates may be lowered to allow for checking up the upstream water surface elevations along Big Ditch. The exact make and model of the gates will not be determined during the design engineer. Rather, an engineering specification with minimum performance requirements (equivalent or better to a Waterman 54in C-10 gate) is specified in the engineering contract specifications.

One unique modification that will need to be made to the gate(s) is the fabrication and cutting of a 9in by 18in vertical slot, located 5 ½in from the bottom rim of the gate (Figure 12). This slot has been sized to provide a maximum velocity of 8.0fps with 3ft of head on the upstream side of the gate with a flow rate of 8.7cfs. The fish passage and hydraulic design of the gates is discussed in detail in TM 2.0 – Big Ditch Design Analysis. Currently, the slot is designed as a permanent opening on the gate, with an attachment where the slot can be covered and closed (watertight) if necessary. The permanent slot size is the most simplistic for operating purposes. Having a variable slot size, or variable setting on the slot cover would increase the operating range of this feature and allow for future modifications in operations as necessary. It is recommended that the current configuration be utilized, and then velocity monitoring be performed during irrigation summer period operations.

![Figure 12. Vertical Slide Gate Schematic (design based on Waterman Gate Style C-10)]
(2) 54in HDPE Pipe Design
The next major structural element is (2) 54” High Density Polyethylene (HDPE) inverted siphon pipes. The pipes connect through waterstops at the (2) inlet/outlet headwalls. Design of the pipes included several analysis items.

- Pipe material type and thickness specification
- Pipe diameter and conveyance requirements
- Pipe clearance or burial depth requirements
- Deflection
- Buckling
- Bending stress/strain analysis
- Buoyancy
- Pipe connections, sealing and waterproofing
- Foundation and bedding requirements
- Filter diaphragm requirements

More detailed information on pipe design is included in the design Technical Memorandum – Inverted Pipe Design, Nov. 3, 2009 (App. A.3).

Pipe Material & Thickness
Two types of materials were originally considered for the project, including HDPE and concrete. Concrete pipes were historically used for construction of standard inverted siphon crossings. The materials are robust and the service life of the structure could range up to 100 years. One key consideration in the design of the structure is how it will be built. Using a concrete pipe system, sections of concrete would need to be lowered and connected in a trench or between a sheet pile water isolation system, where the trench is dewatered to allow installation. The water isolation and diversion system would be a significant element to allow the assembly and installation of a concrete pipe system.

Using HDPE provides a number of benefits above concrete for the inverted siphon project including site isolation and dewatering, building in the wet and submerging the pipes, and pipe jacking. In addition to the various construction methods, the HDPE pipe will be welded together and limit potential leaking from settlement or seismic events. The HDPE pipe also is flexible and can provide other benefits as they relate to seismic conditions. The HDPE also has minimal hydraulic roughness and highly efficient conveyance, which was a key consideration in maximizing the inverted siphon conveyance capacity.

The recommended pipe material type for the project is to use high density polyethylene (HDPE) for its flexibility during construction, low hydraulic roughness, and demonstrated effectiveness on other pipeline projects. The ability to fuse weld the pipe pieces in the field is a positive for installing a watertight system in poor soil foundation conditions. Pipe jacking or trenchless construction are both options for the contractor using HDPE.

Soil external pressures from dead and live loads were evaluated to assess the necessary pipe thickness and material specification. Soil pressures were derived from the deepest sections below the levees at a height of 18.0ft and live loads were evaluated using an H-20 loading pressure and the Boussinesque line load equation for an infinite strip at the shallowest soil depth above the pipes near the inlet and outlet structures where vehicles may drive and park for maintenance activities. The pipe loading analysis indicated a total pressure of 33psi, for which an HDPE pipe rated to 50psi was specified giving a factor of safety of 1.5.
Pipe Diameter and Conveyance Requirements
Per the hydrologic and hydraulic analysis, it was demonstrated that the inverted siphon pipes have significant increases in capacity due to the lowering of the channel invert and improvements in hydraulic efficiency, while meeting fairly rigorous fish passage design criteria (maximum velocity of 2.0fps for the fish passage design discharge of 63.1cfs) for juvenile salmonids and adult trout.

Pipe Clearance or Burial Requirements
Pipe clearances and scour protection were evaluated in detail using the methods described in the FHWA Manual HEC-18 (FHWA 2001). In addition a scour analysis, anecdotal evidence of scour at other tidal marsh restoration projects was evaluated and compared to the engineering calculations to check the predicted estimates. Overall, 3.1ft of scour was predicted to occur at the project site in the area near the inverted siphon. The pipe invert has been established at a minimum depth of -11.0ft, which would provide for 3.1ft of scour plus 3.0ft of cover on the pipe. There is a small probability that a maximum scour of 6.0ft could occur. The design addresses this condition by placement of coarse pipe backfill material above the pipe that would armor and protect the pipe (Figures 14-15). If maximum scour does occur, the pipe should not be dewatered at any time due to buoyancy concerns.

Pipe Deflection
Pipe deflections were evaluated using methods prescribed in an HDPE design manual (App. A.3). Pipe deflections were estimated at 3.0% of the pipe diameter which falls within the recommended maximum of 7.0% deflection.

Pipe Bending Stress and Bending Strain
Pipe bending stresses were checked to not exceed 3,000psi and bending strains not exceed 5% for polyethylene. The criteria for bending stress check was 856.8psi, and bending strain 0.787%, for which both meet the design criteria checks.

Buoyancy
In evaluating pipeline buoyancy, the standard methods are typically to evaluate a dewatered condition. This condition could only occur if the pipe was drained using a pump system, as the pipe is below the local groundwater table. Considering the fully drained condition, pipe buoyancy is determined by evaluating the downward saturated soil and pipe weight against the upward buoyancy force equivalent to the weight of the water displaced by the pipe.

For the minimum cover condition, the soil load on the 100ft section of pipe provides adequate protection, with a factor of safety of 2.0. This analysis does not account for additional resistance factors such as the pipe behaving as a singular structure connected to the headwalls with significantly more cover underneath the levees. Accordingly, the pipe is not expected to float when empty of water. However, if significant scour does occur, then the pipe should not be fully dewatered as this specific condition is not expected nor has it been evaluated.

Connections
The pipes will be connected via HDPE fusion welding which provides a completely watertight seal in the field. Pipe connections and waterproofing will be tested and inspected upon completion prior to initiating backfilling the excavated areas.

Sealing, Waterproofing & Waterstops
Waterproofing seals are required for the pipe penetrations through the concrete headwalls, and will be included in the specifications. A number of products are available for waterproof seals or connections at the headwall. The following types were reviewed for this project:
The Nature Conservancy  Final Basis of Design Report
Fisher Slough – Final Design and Permitting

- Hydraulic concrete grouts
- Rubberized grouting rings and gaskets
- Elastometric sealants
- Structural flanges/boots with grout and sealants

Standard hydraulic concrete grouts are typically filled around the pipe penetration through the headwall connection. Issues related to using waterproof concrete grout only are related to water seepage resulting from shrinkage of the concrete and grout, and shifting or settlement of the pipe, both can cause cracks in the grout.

Rubberized grouting rings are typically a gasket ring that slides around the pipe and is placed in the concrete form. These gasket rings are manually tightened around the pipe, and then concrete poured around the gasket, and filled with waterproof grout sealant. One of the problems with rubberized gaskets is that they can dry out and deteriorate if exposed to air or sunlight. The pipe will be submerged nearly full time, so air should not be an issue.

Another category of waterstops are structural flanges that are either connected to the headwall and then filled with grout and sealant, or welded to the outside of the pipe and placed in the headwall with concrete poured around the flange, and backfilled with grout and sealants. The structural flanges can provide excellent water sealant, but have limitations for flexibility due to pipe shifting and settlement.

Elastometric sealants are typically rolls of adhesive materials (Prostik and Synko-flex or Hydro-flex) that are wrapped sealants on the pipes. These gaskets are flexible and can accommodate some shifting and pipe settlement. However, some products can deteriorate over time if exposed to sunlight and air.

Due to the potential for settlement and shifting of the pipe, we are recommending an elastometric sealant and waterstop for the structure such as Hydro-flex, HF-302 product made by Henry.

In addition to the pipe penetration waterstop, bonding agents are required for all concrete cold joints in the structure.

**Foundation, Bedding and Backfill Requirements**

The foundation of the pipe will use a composite of geotextiles fabric laying on in-situ soils, and then a layer of pipe backfill material placed up to the mid-point or spring line of the pipe. Levee suitable fill material will be placed on top of the pipe and in the excavated levee areas. Towards both ends of the pipe, a filter diaphragm will be installed to prevent seepage along the pipe system, and limit the potential for soil erosion through the embankment. It is noted that if pipe jacking or trenchless construction methods are used, an engineering analysis will need to be provided showing stability of the pipe without bedding material or compacted backfill.

The underlying geotextiles fabric will be used as a filter to prevent seepage and erosion of underlying soils into the bedding and backfill layers, which could create adverse seepage and settlement in and around the pipe. The geotextiles fabric will also provide an initial working base for the construction contractor to begin to lay down the bedding material and create a working platform for pipe installation. The material specification for the underlying fabric is a Mirafi Non-Woven 180N equivalent or better.

The next layer of material is pipe bedding material to be laid along the foundation and bedding zone of the pipe. A typical specification is recommended using WSDOT pipe bedding material. WSDOT, for plastic and thermo-plastic pipes, specifies backfill of the pipe bedding and pipe backfill zones using the
pipe bedding material specification 9-03.12(3) (WSDOT, Standard Specifications 2008). The material will be compacted to 90% maximum dry density, per Standard Proctor.

The upper layers of materials will be suitable levee materials (as shown in other sections of the design plans and specifications) compacted to 95% maximum dry density per standard Proctor ASTM D-698.

Filter Diaphragm Design
A comment was provided by the TNC engineering design review if a seepage collar may be necessary along the pipes. Upon review, Tetra Tech and URS concluded that a filter diaphragm along the pipes in the levee embankment was warranted to reduce seepage velocities and protect from material piping and erosion. The filter diaphragm design method uses the NRCS, NEH Part 628 Chapter 45 Filter Diaphragm Design and 633 Chapter 26 Determining Filter Gradation Limits. The filter diaphragm design is configured with the filter diaphragm dimensions equal to 2D on the sides and top of the pipe, and 1D below the pipe. The materials for the filter diaphragm are specified as ASTM C-33 concrete sands, compacted to 90% maximum dry density, per ASTM D-698.

Inverted Siphon Pipe Construction Installation
The construction of the inverted siphon near the Pioneer Highway bridge crossing Fisher Slough is a complex design and construction engineering project. Two known methods of construction were evaluated for feasibility. They included pipe jacking/direction drilling (trenchless methods) and open cut construction methods. Per the original design reports, trenchless methods were evaluated and considered the preferred approach. Multiple trenchless construction contractors were consulted and the fee for trenchless construction appears to be higher than the current construction budget. At that point, a more detailed and comprehensive review of open trench excavation and dewatering of the pipe crossing installation was examined in more detail to determine if the construction was feasible so close to the bridge crossing. It was determined that both methods are feasible, and ultimately the contractor will need to decide what the most cost-effective solution is for installing the inverted siphon pipes. The plans currently show open trench construction (as the engineer thought this the most cost effective solution), but plan and bid notes are written in such a way that the installation method is per the recommendation of the construction contractor. Both of these options are described in more detail herein.

Trenchless Construction Installation
Trenchless construction would involve either pipe jacking or directional drilling and slip-lining and pulling a pipe through a casing underneath the slough. Trenchless construction technologies are considered to have less impact and effects on aquatic areas since the work is being performed underground and water control diversions are not necessary during pipe installation. A case study was examined that provided a technical basis that this type of activity was feasible for the Fisher Slough project.

Trenchless technology was used for a project in Indio California where a 54” HDPE pipe was pulled through a directionally drilled hole along a 1,500ft alignment (Bueno, 2004). The project involved multiple passes of directionally drilled holes that were sequentially enlarged for the 54” HDPE pipe that was being used as casing for other smaller water/wastewater pipes. After the alignments were drilled, the 54” pipe was attached to a cable pulling winch that pulled the pipe through the bore hole (Figure 13).

Use of such technology and equipment would require special contractor equipment and personnel. The current plan is not to specify either trenching or trenchless methods, rather the specifications and bid schedule will request the contractor provide bids for both options of trenching and full channel diversion, or trenchless methods, to evaluate which method is preferred and will provide the lower cost.

Some trenchless construction considerations include the following:
• Length of pipe and minimum bending radius (the Fisher Slough installation may be too short)
• Approach areas (long open areas are needed for trenching and jacking to line up pipe)
• Mud-wave and shallow soil disturbance from pulling through drill pigs and casings
• Lack of backfill material for maximum scour depth protection not provided in trenchless construction, therefore pipe may need to be placed up to 3ft deeper
• Cost of trenchless construction for (2) 240ft pipes
• Potential effects on bridge and foundations from installation and associated heavy equipment
• Resubmission of inverted siphon plans resulting from potential changes in design configuration and construction effects.

Figure 13. 54” HDPE Trenchless Inverted Siphon Installation

Open Excavation Construction Installation
A comprehensive review of an open excavation construction approach for the inverted siphon pipe installation was performed following the finding that the trenchless installation may be cost prohibitive. The following section of the report is the contents of a memorandum included in Appendix A.3 (Technical Memo – Inverted Siphon Open Excavation Construction Feasibility). The following is a summary of findings from the open excavation inverted siphon construction memorandum. A preliminary plan was developed (Figure 14), that will need to be finalized by the contractor, based on the following general investigative elements:
• OSHA Excavation and Trenching Requirements
• General Excavation Construction Plan
• Temporary Excavation Cut Slope Geometry
• Bridge Protective Measures
• Fisher Slough Diversion Dam & By-pass Structure
• Dewatering Operations
• Pipe bedding and settlement design
• Determination of construction feasibility

OSHA Excavation & Trenching Requirements
The current preliminary installation plan uses excavation as defined by OSHA, with wet conditions that will require special pumping and protective / shoring measures. The standards define the planned construction as an open excavation and not a trenching condition. The requirements by OSHA for a safe excavation area include several elements that will need to be fully addressed in the final inverted siphon construction plan to be submitted by the construction contractor, and approved by The Nature Conservancy and their consulting engineer.

Specific Excavation Requirements (CFR 1926.651)
• Surface encumbrance removal or protection
• Underground installation and utility locates
• Excavation area access, ingress and egress
• Traffic exposure
• Falling load exposure
• Excavation area warning system for mobile equipment
• Hazardous atmospheres
• Protection from water accumulation hazards
• Stability of adjacent structures
• Protection from loose rock or soil
• Daily Inspections
• Fall protection

Requirements for Protective Systems (CFR 1926.652)
• Protection of Employees in Excavations
  o Cave-in protection
  o Sloping and benching systems
  o Support and protective systems
  o Materials and equipment
  o Installation and removal of support systems
  o Sloping and benching work area limits and protections
  o Shield and protection systems

Each of these requirements will be addressed in the contractor final construction plan, as well as a health and safety plan.
General Excavation Construction Plan
The general open excavation construction plan and elements include the following:

- Open cut excavation through both levees and Fisher Slough
- Installation of protective bank stability and shoring measures, primarily for the Pioneer Highway Bridge side of the cut
- Installation and operation of Fisher Slough diversion dam and by-pass pipe to downstream of floodgates
- Installation and operation of dewatering system comprised of well points, pump hose and detention basin

Temporary Excavation Cut Slope Configurations
The excavation will use temporary cut slopes that run through the levees and Fisher Slough, contained within a water diversion and isolation system. Excavations will likely use a temporary cut slopes of 2H: 1V, to a depth 1ft below the pipe inverts, with a 15ft bottom width. The assumed 2H: 1V cut slope is flatter than all slopes specified in the OSHA manual (Type C Soil Slopes of 1.5H: 1V – poorest identified soil slope), and therefore considered a conservative estimate for excavated areas and offsets from the excavation area. Dewatering and minor slope stabilization activities will be necessary.

Using the 2H: 1V slope cuts, the western side of the temporary excavation area daylights at the edge of Pioneer Hwy and the bridge, specifically along the higher levee sections of the cut. It is likely that temporary sheet pile or soldier pile shoring will be necessary along the levee sections to provide embankment stability and protect the bridge. For the Slough area, the trench excavation daylights at the bottom of the channel, with an approximate ~27ft offset from the nearest bridge pile and deck of the bridge. The preliminary plan assumes sheet pile shoring along the entire bridge for a conservative design approach.

The contractor will likely determine a method that incorporates localized shoring near the levee and bridge areas, and possibly open cut slopes along the open area of the lower lying Slough. One potential design option would be to incorporate the shoring measures into the new inverted siphon inlet/outlet headwall and wingwall sections, to remain in place as part of the overall structure.

On the east side of the pipes, the excavation area is free to daylight without affecting other infrastructure. The only potential conflict or impact is the diversion by-pass dam that will be installed as part of the project. The contractor will determine if the proposed ~26ft offsets are adequate, or if changing position of the dam is necessary.

Overall, the cut slopes and offsets to the bridge are feasible, and will not likely affect the bridge infrastructure.

Bridge Protective Measures
The OSHA manual specifies a number of possible protective bank stabilization and shoring measures that can be used to provide stable slopes and protect employees from soil slumping or caving. Typical measures include the following:

- Excavation slopes
- Benching
- Shoring

It is likely for the inverted siphon installation, that each type of measure will be necessary in some form or another in the excavated area. For feasibility, it was assumed that protective shoring (sheet or soldier...
Figure 14. Preliminary Inverted Siphon Installation Open Excavation Plan
Figure 15. Inverted Siphon Plan and Profile
piles) would be used in combination with 2H: 1V excavated slopes. This provides a minimum offset of ~27ft from the top of trench to the Pioneer Highway bridge piers, and 54ft offset from the bridge piers to the bottom of the trench. This is considered adequate offset for protection of the bridge piers and superstructure. The piers are not exposed or in conflict with construction alignments or activities. The method of pile installation will be limited. Vibratory pile installations will not be allowed to protect the bridge.

Monitoring of the bridge will be performed daily during construction. Monitoring will include survey of bridge corner elevations and positions to assess settlement and deformation or changes in position. Daily monitoring during construction will be provided to The Nature Conservancy and Skagit County. Changes (deflection, deformation or settling) greater than 0.1 inch will be flagged for stoppage of construction to inspect and correct shoring measures and implement contingencies if necessary.

Construction contingencies should be set aside by The Nature Conservancy to make corrective changes in shoring and protective measures during construction, and possibly additional bridge shoring as a last resort and worst case scenario contingency item.

**Fisher Slough Diversion Dam and By-Pass Structure**
A diversion dam and by-pass structure will need to be installed during construction. The structure will likely be comprised of a dam built from sand bags, aggregate bags, porta-dam or hydraulic geomembrane dam to check the water and isolate the excavation area. The diversion dam and by-pass structure design are to be finalized by the construction contractor. One possible location that the Contractor should consider is the existing Big Ditch culvert crossing as an option for the diversion structure with a discharge point into the existing Big Ditch just downstream from the crossing. This area has a narrow point between the levees and is a likely water control and diversion location. Water will then be pumped from the upstream side of the dam, into Big Ditch. Pump operations would need to accommodate discharge from the Fisher Slough Tributary system. The estimated summer time high flow rates from Fisher Slough tributaries are estimated at 5cfs to 8cfs. This estimate was determined by documentation of shallow depths of flow 0.1ft deep over the Big Ditch culvert crossing, and application of a broad crested weir equation.

On the downstream side, it is recommended to keep the floodgates completely closed during the inverted siphon excavation and installation operations. Closure of the gates would need to be coordinated through the permitting process. In addition, Dike District #3 and the floodgate manufacturer would need to be consulted on the best method of closure. The floodgate manufacturer may need to be hired to close the gates, in order to maintain the warranty on these recently purchased items.

Part of the diversion dam and by-pass structure will require installing a block net and providing fish removal as specified by the permits.

The diversion dam and by-pass structure will be installed and operational during the in-water work window from July 15, through Sept. 30, 2010. This fish window is firm for the contractor, and they should plan their work to be completed by the end of September.

**Dewatering Operations**
The excavated trench areas will be dewatered using well-point or groundwater pump and sump installations. The dewatering discharge will be pumped out of the excavated area to a detention, infiltration and discharge return area. The most likely dewatering pond is to the south on Smith A property with a return point to Big Ditch. Another option is to pump downstream of the floodgates into Fisher Slough downstream. Unfortunately, this does not allow for settling or infiltration, and there are
some logistical considerations threading pipes and pump hoses underneath and through the floodgates, while keeping the floodgates closed per previous recommendations.

**Pipe Bedding & Settlement Design**
During design review, there was an issue and concern regarding pipe stability due to possible liquefaction and settlement of the pipe during a seismic event. It is estimated that 4” to 6” of settlement could occur during a seismic induced liquefaction event.

The options available for pipe settlement design were either to install piles affixed to the pipes that would prevent settlement, or allow for some settlement as part of the pipe design using standard pipe bedding methods. The pile option was originally considered in the design, but is more costly than the pipe bedding alternative.

The current plan is to use pipe bedding and backfill material, a 1 1/2 inch minus crushed rock, placed 1ft below the pipe invert, and fully embed the pipe to provide additional scour protection. The placement of a geotextile fabric beneath the pipe bedding material (Mirafi 180N equivalent or better) will ultimately improve the workability of the area and reduce the wet and muddy conditions. The pipe backfill material will be compacted and will provide a stable working surface for equipment and laborers during pipe installation. Use of the geotextiles and compacted bedding material will be more stable both during construction and reduce settlement potential in the long term.

An additional 6 inches was added to the headwall (18in) to allow for slippage of the pipe through the penetration. This in combination with a flexible waterstop will allow pipe settlement and shifting and accommodate seismic conditions, and limit the potential for disconnecting from the headwall and subsequent leaking.

**Inverted Siphon Operations & Maintenance**
The inverted siphon will require maintenance by Drainage District #17. The following are general maintenance items that will need to be performed.

- Annual cleaning of sediment basin
  - Perform during in-water work window Aug. 1 – Sept. 30 annually
  - Use an excavator with toothless bucket
  - Report dates and volume of materials dredged and disposed of from basin per drainage maintenance agreement requirements
- Periodic cleaning of trash rack
  - Trash rack will likely need to be cleared after flood events and debris begins to clog on rack
  - Clean trash rack grate using rake or pick from catwalk
  - Provide notification of amount of materials removed and disposed of from basin per drainage maintenance agreement requirements
- Irrigation operations of vertical slide gates
  - Summer irrigation period (June 1 – Sept 30) allows closing of vertical slide gates to check water in Big Ditch for upstream irrigation practices
  - Provide notification of gate closure per drainage maintenance agreement requirements
- Pipe sedimentation and cleaning
  - Pipe cleaning will only need to be performed periodically on semi-annual basis
  - Pipe cleaning and clearing may use the following methods
    - Closure of single gate (alternating) thereby flushing water through each pipe
    - Jet vacuum snaking and cleaning of pipe contents
    - Dragline or mechanical pig cleaning of pipes
At no time should people enter the pipe.

The specifics of operations and maintenance activities, procedures, timeframes and responsible individual parties are described in further detail in accompanying DRAFT - Big Ditch Inverted Siphon Operations & Maintenance Plan, 2009.

2.2.3.4 Big Ditch and Other Project Bridge Crossings

Five channel crossing structures are currently planned (or in existence) along the Big Ditch realignment and within the project bounds. The plan is to install four new crossings (bridges) over the Big Ditch realignment as part of the base contract. In addition to the Big Ditch base contract crossings there are three optional crossings that may be needed. The crossings are summarized as follows:

- Jungquist Northeast Crossing (salvaged from existing crossing) (BASE)
- Jungquist Northwest Crossing (BASE)
- Jungquist Southwest Crossing (BASE)
- Smith A South Crossing (BASE)
- Temporary Fisher Slough Crossing (OPTION)
- Smith B Access Crossing (OPTION)
- Sersland Access Crossing (OPTION)

For the four base bridge crossings, the primary function of these crossings is to allow for heavy farm equipment crossings to the fields, and also to provide vehicle and equipment access to the new setback levee. In addition to heavy farm equipment, it may be necessary to cross these structures using heavy construction equipment during construction. Large excavators, pile drivers and cranes may be accessing the site during various stages of construction. These types of equipment can have excessive vehicle loads. The crossings are designed for highway rated H-20 loads, and will be adequate for heavy equipment crossings.

The plan is to use manufactured steel or concrete bridge crossings based on performance specifications and loading requirements. The performance specifications include bridge deck width, span, load requirements, safety features, structural and decking materials. These performance specifications are provided to the contractor in the design plans and contract documents. The construction contractor will then be responsible for bidding, procuring and installing the bridges per manufacturer specifications. A generic crossing for the project is shown in Figure 16. The performance specifications of the bridges are as follows:

- Bridge Span: Varies by location 60ft – 80ft
- Load Rating: H-20
- Deck Width: 14ft
- Bridge Structural Materials: Steel or Concrete
- Bridge Decking Materials: Concrete
- Abutments and Foundations: To be specified by the manufacturer and provided by the contractor
- Appurtenances & Safety Features: Guard rails meeting AASHTO standards that provide access to farm equipment.
- Deck Elevations: Varies depending upon location, mainly located with bridge deck low chord equal to the elevation of adjacent farm elevations.
- Warranty Requirements: 10-year warranty
At one point in the design process, there was a discussion of salvaging/recycling the existing bridge crossing Big Ditch along the Jungquist property. The bridge and deck were in poor condition, and this concept of recycling the bridge has been eliminated from the project. The bridge in question will remain in place and is not included in the current project plan.

The Smith A crossing is an 80ft bridge. For the Smith A crossing, there are a variety of construction related issues. The construction sequencing of Big Ditch and the levee setback pre- and final loading will need to be considered by the Contractor. In addition to construction sequencing, there are grading and profile issues. The Smith A crossing needs to connect to Pioneer highway, ramp down to the bridge, cross Big Ditch and then ramp back up to the S. Levee Setback. Final grading has been developed with maximum 10% ramp grades, and connecting on top of the levee with the levee access road.

The Jungquist Northeast access bridge is a 60ft stand alone bridge without connections to a formalized road system. This will provide access for farm equipment crossing.

The Jungquist North and South access bridge crossings are 60ft crossings that connect with to Pioneer Highway. The North crossing is a fairly straightforward installation on flat ground. However, the Jungquist south crossing will need to be located on a mild grade down from the Pioneer Highway to the farm road. This structure also crosses at the head of the inverted siphon sedimentation basin.

The 60ft span Temporary Fisher Slough crossing was designed as an after-thought in the project when the total number of truck and heavy equipment trips were being realized. In particular, the designers became aware of the number of trips (4,500+ inter-site transfers) from between the north and south sides of Fisher Slough. This temporary crossing is at the option of the construction contractor to alleviate traffic concerns and minimize traffic control requirements for the project. If the contractor decides to utilize the temporary crossing, the dates of installation and operation are key considerations especially with respect to permit conditions. Also, the contractor will be required to reconstruct the north levee with suitable levee fill, regrade to existing conditions, place 6in of topsoil and hydroseed per other levee installation requirements. The temporary crossing should only be allowed during June 1 through Sept. 30 periods only to minimize potential flooding from cut needed in the North levee. Also, the no portion of the bridge should encroach below the 12.0ft ordinary high water (OHW) line to limit permit related issues.
Figure 16. Typical Bridge Crossing Section
2.2.3.5 **Big Ditch Channel Connections**

Once the inverted siphon and channel construction are complete, then the ditch and structure will need to be watered. Initially, this should be done slowly. A small backwater breach at the downstream end of the Big Ditch realignment would be a likely starting point. This breach would fill the inverted siphon fully, and backwater upstream to the upper channel connection. Then the upper channel connection could be breached. Breach materials should be stored nearby at the breach zones until the rewatered channel and inverted siphon structure operations are observed and confirmed in working order.

2.2.3.6 **Existing Big Ditch Fill and Regrade**

Upon rewatering of the new channel and inverted siphon, work will ensue on the filling of the existing Big Ditch. Soils that were excavated from the new channel realignment will be placed into the existing Big Ditch alignment. Some additional discussion on the level of compaction and methods for placement will be provided in the next phase of design. Filling existing ditches may require special treatments to avoid cracking and development of seepage fissures along the existing bank line.

Per Figure 3, the cross hatch and northern most section of the existing Big Ditch between the new connection and the North levee will be filled and regressed with a smaller drainage ditch along the centerline. The next downstream area to the south to the existing crossing will be fully filled and graded to match surrounding elevations. Downstream from the current culvert crossing, Big Ditch will remain open for Phase II construction to allow for emergency overflow in between Phase II and Phase III construction. The downstream segment will be filled in Phase III construction.

2.2.3.7 **Local Farm Drainage Channel Regrade**

The existing drainage ditch that parallels the north levee and farm road will be re-excavated and the profile redirected towards the west (Figure 17). The existing ditch is 20ft wide and 5-6ft deep. The regrade will increase the ditch width to 35ft wide, and lower the ditch bottom from an elevation of 2-3ft down to 1ft. The ditch profile will be graded flat. In addition to the ditch regrade, the existing buried culvert will be replaced with a 30ft long, 48-inch corrugated metal pipe (CMP) culvert (Figure 18). Also, a 48” pipe will be installed at the confluence of the realigned Big Ditch near the Jungquist South crossing.

One of the key concerns on the site are potential cultural resources that are potentially located within the local drainage channel regrade. 45SK50 was a shell/midden site identified in previous archaeological investigations. Any grading and excavation work to be performed on the local drainage channel regrade will require coordination with the Owner’s representative and request for a cultural resource observer on site during construction activities. If historical or archaeological resources are discovered during construction, the discovery protocols included in the contract specifications will be implemented by the contractor, The Nature Conservancy, owner’s representative and other associated parties.
Figure 17. Regrading Profile of Existing Drainage Channel

Figure 18. Removal and Replacement of Existing Culvert
2.2.3.8 Existing Big Ditch/Fisher Culvert Crossing Demolition

The final element of the Big Ditch Realignment will be the demolition and removal of the existing Big Ditch culvert crossing. This structure is large and likely has significant amounts of concrete and sheet pile (Figures 19-20). The structure will be completely removed and disposed of off site.

In order to demolish this structure, installation of a full water diversion and by-pass of Fisher Slough will be required. This can likely be managed with the S. Levee demolition work. It is recommended that the contractor coordinate closure of the floodgates to minimize tidal inflows, and then pump and divert tributary flows. The contractor will be required by the contract documents to submit a care and diversion of water plan for removing this structure using the guidance put forth in Section 2.2.2.4 TESC and Water Diversions.

Associated design elements with the demolition of this structure include removing a monolithic concrete structure, H-pile and cut underlying support piles down to grade. The line of demolition is approximately the inside toe of the levee. The remaining structure will be plugged with concrete and then buried into the levee prism. The levee reconstruction will be built to match the existing grade and topography. The north levee will be permanently rebuilt.

The demolition of the existing Big Ditch culvert crossing must occur in Phase III, 2011. The reason for this is that the structure is part of the emergency spillway and will need to remain operational between Phase II and Phase III and can be removed when the S. Levee realignment and spillway installation are complete in 2011.
2.2.3.9 **Big Ditch O&M Requirements**

The Big Ditch and inverted siphon operations and maintenance requirements are summarized as follows:

- Annual cleaning of sedimentation basin. Report sediment removal volumes to WDFW.
- Annual and periodic cleaning of trash racks as necessary after storm events.
- Optional closure of gates from June 1 through Aug. 30 for irrigation management. Report closures or changes to gate settings to WDFW.
- Full opening of gates from Sept. 1 through May 31. Report gate opening to WDFW.
- Periodic mowing of channel banks to maintain flow conveyance.
- Periodic dredging of channel due to long term sedimentation.

The operations and maintenance requirements will be provided to The Nature Conservancy, DD#3 and DD#17 as a separate report with more detail on the activities involved.

2.2.4 **South Levee Setback**

This section of the report describes the general configuration and technical design criteria used in developing the levee setback design. For reference the South Levee Setback construction will occur in both Phase II and Phase III construction. The dual phases of construction are planned to allow for differential settlement along the levee, and other construction sequencing issues such as spillway construction and decommissioning of the existing spillway on the current levee. The levee design approach used the methods and criteria as described in the Army Corps manual for Engineering Design.
and Construction of Levees (USACE, 2000). The general approach for design investigations includes the following elements:

- Geologic setting
- Geotechnical soils investigations
  - Subsurface explorations and soils testing
  - Estimated soil layer permeabilities
  - Existing borrow soil suitability
  - Levee settlement estimates
- Hydrologic and hydraulic design analyses
  - Groundwater conditions
  - Surface water hydrology
  - Hydraulics
    - Levee profile elevations
    - Embankment protection
- Seepage
- Embankment Stability
- Interior drainage effects
- Special design considerations - pipeline crossings and penetrations
- Access ramps and roads
- Closure structures
- Levee vegetation management

Each of these levee design elements are presented herein.

2.2.4.1 Geologic setting

The 2002 USGS Geologic Map of Washington Northwest Quadrangle (Dragovich et. al., 2002) indicates that the Fisher Slough Restoration Project site lies within an area of Quaternary alluvium (Qa) consisting of sorted combinations of silt, sand and gravel deposited in streambeds, alluvial fans and locally including peat and lacustrine deposits. Near the eastern border of the site, the alluvium transitions to undifferentiated glacial outwash (Qgo), characterized as recessional and proglacial stratified sand, gravel, and cobbles with minor silt and clay interbeds deposited in delta, ice-contact, beach, and melting water stream environments.

The Fisher/Carpenter Creek landscape straddles the border of two physiographic regions, the North Cascade Mountains on the east and Puget Lowland to the west. Headwaters of the three watersheds that make up the Fisher Slough watershed (Little and Big Fisher Creeks and Carpenter Creek) originate in the terraces and foothills bordering the western edge of the Devil’s and Cultus Mountains ranging between 800 and 1000 feet in elevation, that comprise the western slopes of the North Cascades. The area has been greatly influenced by glacial actions that created terrace and plateau features that trend north-northwest and range in elevations from 40 to 340 feet. The soils are moderately deep and moderately well drained and are formed in volcanic ash and glacial till. The creeks and their small tributaries cut through these terraces forming a rolling topography on the terraces. The creek channels form deeper ravines before entering the valley floodplain. The creeks converge to form Fisher Slough after entering the level to nearly level Skagit delta floodplain before terminating at the South Fork Skagit River. Underlying geology of the delta is comprised of glacial deposits and alluvium. Soils of the delta are very deep and naturally poorly drained.

Seismic conditions were evaluated at the site and per the 2006 International Building Code provisions (IBC, 2006), the project was evaluated for site classification and potential effects of liquefaction on levee
and pipeline designs. In general, the site was characterized as Site Class E (soft soil profile) and a special consideration for Site Class F for 2B stratum soils along the Pioneer Highway and bridge as described in following sections. These seismic conditions are considered in the design with more details provided in App. A.1 – Technical Memoranda - Geotechnical Studies.

### 2.2.4.2 Geotechnical soils investigations

A comprehensive set of subsurface soils investigations have been performed for the project. All soils investigation information and geotechnical analyses are included in App. A.1 – Technical Memoranda - Geotechnical Studies. Surface soils are mapped as Skagit Silt Loam (NRCS, 2008). These surface soils provide excellent, nutrient rich growing media for agricultural operations, and relatively poor drainage.

**Subsurface Soils Investigations**

Information on subsurface conditions were obtained from a previous report prepared by WSDOT for the Pioneer Highway bridge (WSDOT, 1984), and from borings drilled by URS during this phase of the work. The geotechnical investigation included drilling eight bore holes (B-1 to B-4, AB-1 to AB-4), excavating 28 test pits (TP-1 to TP-6, ATP-1 to ATP-3, and TP-7 through TP-25), and drilling and installing three groundwater monitoring wells (GW-1 to GW-3) at locations shown on CAD plan Sheet B2 (Figures 21-22). All test pit data and resulting soil profiles are included in App. A.1. The following is a brief summary of the soil characterization for the project site.

**Stratum 1: FILL- Soft to medium stiff SILT [ML/MH]**

This layer was encountered in all of the borings drilled for this project, and is typically 4 to 9 feet thick. The soil is generally brown in color and soft to medium stiff in consistency. N-values range from 1 to 7 blows per foot. Approximate soil strength measured in the field from pocket penetrometer testing indicated that the undrained shear strength ranges between 250 and 750 pounds per square foot (psf) except at occasional locations where it increases to more than 750 psf, such as at Boring AB-2/GW-1. Laboratory sieve analysis tests on four samples indicate fines contents ranging from 73 to 98 percent. Proctor compaction tests on samples from TP-1 and TP-6 indicate that the optimum moisture content is roughly 18 to 25 percent, which is considerably higher than the existing moisture content.

**Stratum 1A: FILL- Gravel to silty gravelly fill GRAVEL [GP/GM]**

A secondary set of test pits were taken in late summer 2009 due to concerns of possible sand and gravel deposits along the Smith B property. These additional soil test pits discovered a shallow 2ft zone of gravel or silty gravelly fill (GP/GM) between soil testing station 24+00 to 25+50, which corresponds to levee setback realignment stationing 34+00 to 35+50. This limited gravel/silty gravel zone will be referred to as “Stratum 1A” to distinguish it from the brown silt fill previously identified as Stratum 1 at the site. This “Stratum 1A” gravel zone was encountered in Test Pits Tp-14, TP-17, TP-18 and TP-21, and was typically 2 feet thick. This material was medium dense in character and is expected to be free draining. The design plan for this section of the levee is for full removal (and disposal or treatment) of Stratum 1A with inspection and approval of the stripped areas prior to installing levee fill lifts.

**Stratum 2A – Very soft to soft clayey SILT [ML]**

This unit was encountered below Stratum 1, and extended in some borings to the maximum depth drilled. It was dark gray to gray in color, and very soft to soft in consistency. The SPT sampler N-values that were recorded during drilling range from 0 to 10 blows per foot. Approximate soil strength measured in the field from pocket penetrometer testing indicated that the undrained shear strength ranges between 50 and 500 pounds per square foot (psf). Sea shell fragments were encountered in this layer. This layer was encountered in all of the borings drilled for the project. Laboratory tests on five samples indicate fines contents ranging from 54 to 98 percent, but with an average above 93 percent. Other tests showed this soil to be low to medium plasticity (Plasticity Index values ranging from 5 to 24), moderately high compressibility, and low permeability.
Stratum 2B – Very loose to loose SILT/sandy SILT/silty SAND [ML/SM]
This unit was interpreted to be a sandier version of the Stratum 2A deposit described above. It was dark gray to gray in color, and very loose to loose in character. The SPT sampler N-values that were recorded during drilling range from 0 to 6 blows per foot. Approximate soil strength measured in the field in fine-grained zones using pocket penetrometer testing indicated that the undrained shear strength range between 110 and 250 pounds per square foot (psf). Sea shell fragments were encountered in this layer. This layer was encountered at shallow depths less than 5 feet in borings B-1 and AB-4, and at depths greater than 9 to 12 feet in Borings AB-1, GW-2 and GW-3 and in Test Pits ATP-1, ATP-3 and TP-5. Laboratory tests on four samples indicate that the fines content of the Stratum 2B is typically in the range from 28 to 89 percent. One sample from Boring AB-1 showed a low permeability value similar to that of Stratum 2A, although there appeared to be a greater amount of fines (85 percent) in the test sample than is typical for this stratum. This stratum has caused significant concern related to settlement and seepage in particular along the Smith A, Pioneer highway section of the S. Levee Setback.

Stratum 3 – Loose to medium dense silty SAND [SM]
This unit was encountered below Stratum 2A. It was dark gray to gray in color, and loose to medium dense in character. The SPT sampler N-values that were recorded during drilling range from 8 to 17 blows per foot. Sea shell fragments were encountered in this layer. This layer was encountered in three of the borings (B-3, B-4, and AB-2/GW-1) drilled for the project. Laboratory test indicates that this soil contains up to 28 percent fines.

Stratum 4 – Medium dense to dense silty SAND/sandy SILT with gravel [SM/ML]
This unit was encountered below either Stratum 2A or Stratum 3. It was dark gray to gray in color, and medium dense to very dense in character. The SPT sampler N-values that were recorded during drilling range from 24 to 63 blows per foot except at a particular location where the N-values increased to more than 100 at Boring AB-3. This high N-value might be attributed to presence of gravel. The gravel in this deposit ranges in shape from rounded to subangular. Four of the borings (B-3, B-4, AB-2/GW-1, and AB-3) drilled for this project were terminated in this material. A laboratory sieve analysis test on one sample showed 79 percent fines.

Existing Embankment Fill – South Levee
Most of the fill soil encountered in the test pits on the existing levee (TP-1, TP-2 and ATP-2) consists of a moist, low plasticity clayey silt (ML/CL) with sand content ranging from negligible to roughly 27 percent. This layer is typically 10 to 13 feet thick, and is generally brown in color and medium stiff to stiff in consistency. Approximate soil strength measured in the field from pocket penetrometer testing indicated that the undrained shear strength ranges between 500 and 1500 pounds per square foot (psf). At one location, only in ATP-2, the fill soil consisted of a silty gravelly sand to silty gravel (SM/GM) down to 10 feet depth. The soil sample in ATP-2 is attributed to historic dredging practices where Big Fisher Creek bed materials were disposed of on the top and backside of the levee, hence the high gravel content.

Embankment Fill – Old Railroad Grade
Most of the fill soil encountered in the test pits on the existing rail embankment (TP-3 and TP-4) consists of moist, low plasticity clayey silt (ML/CL) with sand content ranging from negligible to roughly 20 percent. This layer is typically 8 feet thick and is generally brown in color and stiff to hard in consistency. Approximate soil strength measured in the field from pocket penetrometer testing indicated that the undrained shear strength ranges between 2000 and 4000 pounds per square foot (psf).
Figure 21. B-1 Soil Mapping Locations
Figure 22. B-2 Levee Setback Alignment and Estimated Soil Profiles.
General Soil Conclusions
In general, the soil profile in the west half of the site is dominated by the deep alluvium deposited by the Skagit River, while the soil profile in the east half of the site apparently transitions to the glacial outwash soil described in the Geologic Setting section. Borings drilled by WSDOT in 1984 indicate that the soft or loose alluvium at the bridge location extends to at least 80 feet depth. CAD plan sheet B-2 shows an estimated soil profile along the future setback levee alignment, illustrating the reduced thickness of alluvium and the appearance of shallow dense glacial soils towards the eastern end of the levee.

The soil encountered within the area to be occupied by the new facility can be used for levee fill material and foundation support with certain precautions, and with the expectation that total and differential settlements may occur after construction due to the soft nature of the soil (Stratum 2A and 2B). A table of estimated soil parameters for each of the soil strata encountered at the site is provided at the end of the text (Table 5) to assist design, site preparation and construction of the proposed facility. The values provided in the table have been estimated using a combination of current measured field and laboratory data together with published data on similar soils. It should be noted that in most cases the values listed in Table 5 are intended to represent average or slightly on the conservative side of average conditions. Natural variations in stratigraphy and soil parameters are expected throughout the site, and the Table 5 values may not be strictly representative of all locations.

Estimated Soil Layer Permeability
Materials and their permeability that were used in the seepage model are as follows:
- Stratum 1 Fill - SILT with a permeability of $1 \times 10^{-5}$ cm/sec
- Stratum 2A – clayey SILT with a permeability of $1 \times 10^{-5}$ cm/sec
- Stratum 2B – SILT/sandy SILT/silty SAND with a hydraulic conductivity of $1 \times 10^{-4}$ cm/sec
- Stratum 3 – SILT/sandy SILT/silty SAND with a permeability of $1 \times 10^{-3}$ cm/sec
- Existing Fill (Railroad Embankment)– SILT with a permeability of $5 \times 10^{-5}$ cm/sec
- New Fill – sandy SILT/silty SAND with a permeability of $1 \times 10^{-4}$ cm/sec
- Seepage Berm Fill–fill with permeability of $1 \times 10^{-6}$ cm/sec

The permeability was obtained from laboratory data for this project, published literature for similar soils, and from URS geotechnical engineering judgment. The lab values of permeability measured for this project in strata 2A & 2B were judged to be too low for accurate representation of field conditions. A conservative higher value was used for the analysis.

Existing Borrow Soil Suitability
An assessment of existing Big Ditch borrow, and existing levee materials was performed during the initial phases of design work to determine if the soils coming from other project features would be suitable for use in the new south levee setback structure. The results of the geotechnical investigations generally concluded that the on-site materials are suitable for use in the new levee structure. These soils are likely the same as those that are found in the existing levee structures that were built using borrow ditches that run along the margins of the Fisher Slough levees. There are a few notable exceptions where suitable fill materials were not found, and should not be used in the new levee, unless treated to meet the project levee fill specifications.

Test pit ATP-2 is a silty gravelly sand to silty gravel (SM/GM) down to 10 feet depth, likely associated with past gravel dredging activities in Big Fisher Creek. This material does not have adequate sufficiently high percentage of fines, and therefore is not in its present condition suitable for use as fill for the levee setback structure. The soil shall either be disposed of at other fill/grading or off-site locations, or treated by the contractor to meet the project levee fill specifications.
In addition, a zone of gravel or silty gravelly fill (GP/GM) was encountered in the upper approximately 2 feet of the soil profile between station 24+00 and 25+50 (as shown in B-1) or station 34+00 to 34+50 as shown on the remaining project CAD plan sheets referring to the S. Levee Setback alignment. The samples that correspond to this area are TP-14, TP-17, TP-18 and TP-21. At a minimum this soil shall be removed and either be disposed of at other fill/grading or off-site locations, or treated by the contractor to meet the project levee fill specifications.

The levee will be constructed in a series of fill materials consisting of silt, clays, sands and/or gravels with a minimum of 25 percent fines passing the #200 sieve (considering only materials with diameters less than 3 inches). Levee embankment fill should be placed in lifts approximately 6 to 8 inches thick, moisture-conditioned as necessary (moisture content of soil should be within 2 percent of the optimum moisture), and compacted to 95 percent of the maximum dry density based on ASTM Test Method D-698 (TNC, 2009a, App. A.1). This method is prescriptive for also providing adequate seepage protection.

One key consideration for the construction contractor is the availability of suitable levee fill material throughout the life of the project. In theory, the total cut/fill volumes for the project are fairly close in quantities. However, a majority of the suitable cut material is contained within the existing levee that will be removed at the end of the project. This creates a gap in the availability of suitable levee fill material until the existing levee is removed. Due to this project sequencing constraint, the contractor will likely need to import up to 40,000cy of material for placement in the new levee and will have a somewhat similar amount of material remaining at the end of the project. All extra soils at the end of the project shall be removed and disposed of at an off-site location. The contractor shall take particular care in developing their bid estimates to consider interim soil stockpiling and handling, with a fairly large load of available fill material towards the end of the project.

Levee Settlement Estimates
Due to the soft nature of the underlying soil (Layer 2a and 2b), the dike will experience considerable settlement, both during and after construction. The estimated settlements for a proposed new setback levee at three locations, are listed in Table 5.

<table>
<thead>
<tr>
<th>Levee Section</th>
<th>Stationing Based on B-1 Map</th>
<th>Location of Settlement with Cross Section</th>
<th>Estimated Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>Likely</td>
</tr>
<tr>
<td>Along Pioneer Highway</td>
<td>Station 10+00 to 17 + 50</td>
<td>Center</td>
<td>3.9 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Toe - both</td>
<td>1.3 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.7 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.9 ft</td>
</tr>
<tr>
<td>Along Railroad Grade</td>
<td>Station 17 +50 to Station 29 + 00</td>
<td>Center</td>
<td>2.3 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Toe – Field side</td>
<td>na</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.6 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>na</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Toe – Over old ditch</td>
<td>1.0 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.7 ft</td>
</tr>
<tr>
<td>Southeast Section</td>
<td>Station 29+00 to 48+00</td>
<td>Center</td>
<td>2.3 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Toe - both</td>
<td>0.5 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.6 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.4 ft</td>
</tr>
</tbody>
</table>
Approximately 50 to 60 percent of the above settlements are expected to occur within 12 months following the construction of the dike, with the remainder occurring over a period of roughly 6 to 10 years or more. The full levee cross-section can be constructed in one season for all sections of the setback levee except that area along Pioneer Highway and new ditch excavation.

The settlement evaluation assumed the settlement occurred at one time and did not consider phased construction described in the stability section above. One result of phased construction is that slightly reduced total settlements may occur.

As was previously mentioned, the contractor is allowed two soil settlement adjustment payments based on observed conditions at the project site. In order to receive these payments, the contractor must install, care for, and survey the settlement plates throughout construction. If plates are damaged during construction, then this will negate the settlement rate estimate, and the contractor will not get paid without proper documentation.

2.2.4.3 Surface Water and Groundwater Hydrology and Hydraulic Conditions

The project site surface water and groundwater hydrology and associated hydraulic conditions play a significant role in levee embankment seepage, embankment erosion, and general stability characteristics. These site characteristics are discussed further to describe how the data were analyzed in associated engineering assessments on levee stability, and other engineering design issues.

Groundwater Conditions

A range of groundwater data have been collected at the project site, including during digging of test pits, borings, as well as well installations (Figure 23). The groundwater conditions are of interest as they affect seepage conditions through the levees and drainage conditions of surrounding farm areas. The following is brief summary of available groundwater data and observed conditions at the project site.

- Initial groundwater monitoring in summer 2006 inside Fisher Slough and outside fisher slough at ORIG_GW1 and ORIG_GW2. Monitoring results show the interior groundwater conditions as a result of daily tidal flushing, and adjacent exterior farm conditions for a well located outside the levee (ORIG_GW2) only 300ft from the interior well (ORIG_GW1). Observations show that the groundwater response at a distance of 300ft was a function of the underlying Skagit River aquifer and only minimally affected by Fisher tidal inflows in July through Sept. 2006 (Figure 24). This is likely a result of the Stratum 1 Fill soft to medium stiff silt, Skagit silt loam surface materials, and the existing embankment South Levee fill comprised of similar types of materials from local borrow and compacted into the levee structure. It is noted that the GW-2 well went dry in mid-August, 2006.

- During the geotechnical soils investigations, several groundwater elevations were documented. Groundwater was encountered both in the test pits and borings during the early October 2008 investigation period. Water was encountered in two of the test pits (TP-5 and TP-6), and was encountered at depths of about 10 feet (Elevation -6) and 5 feet (Elevation 0) below the ground surface respectively. The ground water levels did not stabilize before backfilling of the test pit occurred. Seepage inflow rates ranged from slight to rapid.

- Water levels measured in the wells GW-1, GW-2 and GW-3 in November 2008, at least 2 months after installation, indicated that the water table was at depths of approximately 0.5 feet (Elevation 6.0ft) to 3 feet (Elevation 3.0ft) below the ground surface. There was some indication that the well water levels may have been influenced by the presence of ponded water on the ground surface at the well locations. Local residents typically report that water levels are near the ground surface in winter.
Recent groundwater data were collected during January to September 2009. This data is available from TNC. Of particular interest to the levee setback design is the data from GW-1, which is located midway along the north-south levee section in the Smith B area (Figure 25). The data indicate that for the 2009 monitoring period, groundwater was about 2ft below the ground surface in January, then 3ft below the ground surface in Feb. through May timeframe, and then steeply declined from May to Sept. with a final elevation of -10.0ft which is ~17.0ft below the ground surface. This is the data set referred to in the transient seepage analysis, groundwater boundary condition.
Figure 23. Groundwater Monitoring Locations
Figure 24. Original Groundwater Monitoring of Fisher Slough Inside and Outside the Levees
Figure 25. Groundwater Elevations at GW-1 for Jan. to Sept. 2009 Monitoring of Fisher Slough Inside and Outside the Levee
Levee Design Surface Water Hydrology
The primary function of the Fisher Slough levees is to provide tributary cross drainage from the tributary watershed to the east including Carpenter, Sandy, Johnson, Bulson, Big Fisher and Little Fisher Creeks. In addition, the S. Levee provides some level of protection for downstream flood areas along the Skagit River eastern floodplain along the Skagit Delta towards Milltown and Stanwood Washington. In addition to tributary and floodplain flow along the interior floodplains of the Skagit Delta, the Skagit River itself backwaters up Tom Moore Slough and floods Fisher Slough from the downstream outlet end of the site. The flooding hydrology and hydraulics are complex at the project site. In general conservative design criteria include flood inundation elevations, fully saturated levee conditions on both sides of the levee, and future buildout runoff estimates for upstream tributary runoff conditions were used in evaluating levee performance. The following discussion outlines the hydrologic criteria used in designing the structure that were developed in other phases of the study.

The initial Fisher Slough Preferred Restoration Plan and the Initial Fisher Slough Tributary Flooding and Sediment Investigations (TNC 2007a, 2007b) evaluated tributary and flood hydrology, hydraulics and sedimentation rates for the project. For the tributary watersheds a HEC-HMS model and HEC-RAS model were developed to predict the incoming unsteady flood flow hydrographs into Fisher Slough. These inflow hydrographs were coupled with downstream tidal and flood water surface elevation and inflow boundary conditions using a MIKE-11 model.

Flood peaks inflows were estimated for both existing and future watershed buildout conditions, per the Skagit County Comprehensive Plan and assumptions of full build out in the associated urban growth areas within the watershed. Table 6 is a summary of estimated 1% exceedance flood peak flows into Fisher Slough. The Future $Q_{100}$ peak flow condition was used for the spillway design inflow coupled with flood water surface elevations from FLO-2D (USACE 2005), levee probable likely failure point modeling conditions.

Table 6. 1% Exceedance Tributary Flood Inflows to Fisher Slough

<table>
<thead>
<tr>
<th>Tributary</th>
<th>Existing $Q_{100}$ (cfs)</th>
<th>Future $Q_{100}$ (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hill Ditch</td>
<td>630</td>
<td>700</td>
</tr>
<tr>
<td>Big Fisher Creek</td>
<td>260</td>
<td>270</td>
</tr>
<tr>
<td>Little Fisher Creek</td>
<td>700</td>
<td>710</td>
</tr>
<tr>
<td><strong>Total Inflow</strong></td>
<td><strong>1,590</strong></td>
<td><strong>1,680</strong></td>
</tr>
</tbody>
</table>

A 100-year flood water surface elevation of 16.7ft (NAVD88) was selected from evaluating the results of the Corps UNET and FLO-2D model as published at the time of this phase of the Fisher Slough study (USACE, 2005). The Corps study has much deeper floodplain depths as compared with FEMA maps, 16.7ft versus 12.7ft, and therefore the selected inundation condition is likely conservative. The 16.7ft elevation (per the Corps Probable Likely Failure levee condition for the Dike District #3 levees south of Mt. Vernon) was used for evaluating levee, steady state seepage and embankment stability conditions, as well as emergency spillway overtopping flow conditions.

Other fall/winter and spring/summer flow conditions, as monitored during the 2006 through 2008 hydrologic data collection periods and as documented in the Fisher Final Design Recommendations Report (TNC 2009a), were used to evaluate inundation more typical flood conditions that will be experienced at the project site. It is understood that the tributary levee setback will have little to no effect on the larger Skagit River flood conditions. However, tidal flow conditions and smaller flood events (5-10 year event in Fall 2008) have more effect on velocity, shear stress, scour and erosion conditions as compared with large flood conditions where a majority of the floodplain is within ponded backwater.
areas. These data collected at the site provide an excellent range and basis for typical tidal and frequent flood conditions at the site.

Levee Design Hydraulics
The levee design hydraulic analysis for Fisher Slough area used MIKE-11. The MIKE-11 model was selected for its ability to develop rule curves to simulate floodgate opening and closures for a self-regulating structure. Inflow nodes and boundary conditions for the project include unsteady water surface elevations and flows for the Skagit River, and each of the three tributaries Hill Ditch, Big Fisher and Little Fisher Creeks.

100-year flood event modeling was performed using predicted Skagit River stage from the UNET modeling output (USACE 2005) at a node proximate to the project site. The results showed clearly that the levee setback alternatives provide minimal benefits for flooding in that the Slough still flooded, but was delayed by a period of 3 to 6 hours for 100-year tributary inflows and a range of Skagit River downstream stages (TNC, 2007a). The effects from flooding typically involve overtopping and spilling from the Slough into Big Ditch at the current culvert crossing, as well as upstream overtopping of the levee at several upstream locations from Slough backwater conditions. The upstream overtopping locations are near a private bridge just south of Lake McMurray road, and also at several other low lying levee profile locations along Stackpole Road.

With this understanding that the project provides only incremental improvements in flood storage, a majority of the modeling efforts focused on small, more frequent flood events. It is during these events that increased velocities and shear stresses are observed during flood and ebb conditions as compared with the larger events where the backwater flooding can raise water surfaces for extended periods.

Fortunately, the project hydrographic data collection series was able to capture a representative 5-10 year flood event in November 2008 that was used as the project design event. The MIKE-11 model was calibrated to this event, and a number of downstream boundary conditions, alternatives of floodgate closure elevations, levee setback locations, filling and grading, and pilot channel excavation for improved drainage were performed as part of the final floodgate and final tidal marsh restoration recommendations study (TNC, 2008a and TNC 2009a).

The data collection series in 2008 was evaluated for both spring juvenile chinook and summer irrigation periods and fall/winter flooding conditions (TNC, 2009a, Appendix C.2). Table 7 is the summary of hydraulic flow conditions for the active alternative at key section locations in the marsh restoration area. Figure 26-30 show the existing and predicted water surface and velocities for both modeling periods.

The following are a few of the key results from the hydraulic modeling analyses:

- Additional flood storage of 311 acre-ft reduces the flood water surface elevations by 2.8 ft for the Nov. 2008 flood event (approximately 5-10 year event).
- Construction of pilot channels and minor grading along levee toe areas increases drainage and reduces the potential for shallow ponding which could cause water temperature increases and stranding of fish.
- Construction of pilot channels and minor grading of the active restoration approach increases the diversity of species with more high marsh and riparian species expected to become established on the site as a result of natural recruitment, inundation frequencies and expected vegetation composition.
- Pilot channel excavation reduces the likely timeframe for full establishment of a dendritic tidal channel network from a 10 yr to 30 yr period down to a 2 yr to 10 yr period of development.
• Tidal flushing is the primary mechanism for channel development and erosion, whereas during flood conditions, channel and floodplain velocities and shear stresses are negligible as Fisher Slough becomes a large backwater ponded area.

• Channel shear stresses from tidal flushing are fairly low in general with average shear stresses well below the permissible shear stresses in the range of 0.024 to 0.074psf. Peak shear stresses occurring on the order of 1% of the time, and mostly associated with ebb tide conditions are on the order of 0.071psf to 0.123psf (for Fisher main tidal and Smith B respectively). These peak shear stresses will help to develop the tidal drainage network, but have limited ability to erode vegetated levee areas. The potential for levee embankment erosion is discussed further in the following sections of the report.

• For the Fall 2008 flood event, the existing condition would spill whereas with the levee setback project, the peak stage of 11.4ft is 1.6ft below the 14.0 spillway crest elevation, thereby reducing the potential for spilling into the Big Ditch system.
<table>
<thead>
<tr>
<th>Table 7. Hydraulic Summary, Fisher Slough Active Alternative</th>
</tr>
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<tbody>
<tr>
<td></td>
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<tr>
<td></td>
</tr>
<tr>
<td>Flood Peak Elevation</td>
</tr>
<tr>
<td>% Time Gates Open</td>
</tr>
<tr>
<td>Avg. Hours Per Day Gates Closed</td>
</tr>
<tr>
<td>Hours Closed Per Period</td>
</tr>
<tr>
<td>Percent of Year Closed</td>
</tr>
<tr>
<td>Interior marsh</td>
</tr>
<tr>
<td>Average Floodplain/Channel Elevation (ft)</td>
</tr>
<tr>
<td>Average Depth of Flow Over Floodplain (ft)</td>
</tr>
<tr>
<td>Maximum Floodplain Depth (ft)</td>
</tr>
<tr>
<td>Minimum Water Surface Elevation (ft)</td>
</tr>
<tr>
<td>Max Flood Velocity (ft/s)</td>
</tr>
<tr>
<td>Avg. Flood Velocity (ft/s)</td>
</tr>
<tr>
<td>Max Ebb Velocity (ft/s)</td>
</tr>
<tr>
<td>Avg. Ebb Velocity (ft/s)</td>
</tr>
<tr>
<td>Max Shear (lb/sf)</td>
</tr>
<tr>
<td>Avg. Shear (lb/sf)</td>
</tr>
</tbody>
</table>
Figure 26. Fall/Winter Tidal & Flood Hydraulics (1 of 3)
Figure 27. Fall/Winter Tidal & Flood Hydraulics (2 of 3)
Figure 28. Fall/Winter Tidal & Flood Hydraulics (3 of 3)
Figure 29. Spring/Summer Tidal & Flood Hydraulics (1 of 2)
Figure 30. Spring/Summer Tidal & Flood Hydraulics (2 of 2)
2.2.4.2 Levee alignment, profile, section geometry

The levee setback alignment is 3,929 feet in length and will replace the existing south levee which is 5,470ft long. The levee setback begins at the intersection between the Fisher Slough Pioneer Highway Bridge Crossing and the existing south levee. The levee setback footprint is 6.5 acres of cut/fill activities.

Levee Design Profile

The profile of the levee is being rebuilt to an elevation of 18.0ft (similar to existing conditions except for portions of the levee that have settled) and is 1.3ft higher than the 16.7ft, 1% flood exceedance stage on the mainstem Skagit River as estimated in the Corps’ Probable Likely Failure point scenario for the DD3 levee. This does not meet FEMA freeboard criteria of 3ft for the 1% exceedance event, but in general the DD3 levees do not meet 1% exceedance flood protection and freeboard criteria. Matching the mainstem river levee and north levee profiles is appropriate for this design. Increasing the height is not acceptable, as the S. Levee is a downstream levee on the floodplain and could cause significant changes in flooding and depths by creating upstream backwater conditions for the levee failure flood scenario as described earlier.

The levee setback design profile is set at a constant elevation of 18.0ft along the entire length of the levee, with the exception of the overflow spillway, which has a spill elevation of 14.0f. The 18.0ft levee setback profile elevation is representative and slightly higher than some existing levee locations that have experienced settlement for the existing structure towards the Pioneer Highway crossing.

Further upstream, the levee profile rises above 18.0ft in elevation in particular near the Big Fisher Creek and Hill Ditch confluence, which is likely a result of dredging and placing materials on the levee. Further upstream the profile dips down to 16ft and then back up above 18ft along Little Fisher Creek. The levee along this area acted to force Little Fisher Creek across the valley hillslope to maintain grade, and channelize the stream towards Big Fisher Creek and the Hill Ditch confluence. With the new levee alignment, Big Fisher and Little Fisher Creek profiles will fall 5ft to 6ft onto the tidal marsh floodplain, and be much less confined, thereby limiting the potential for levee overtopping as their base flood elevations will be significantly reduced, depending on Skagit backwater conditions and gate operations.

Overall, the levee will have a slightly higher profile than existing conditions with a similar overflow spillway elevation, and match the North Levee profile and connect with the mainstem Skagit River DD3 levee system. The levee setback will provide increased flood storage and reductions up to 3.0ft in elevation for frequent flooding (5-10yr) events in Hill Ditch (Carpenter), Big Fisher and Little Fisher Creeks.

Levee Design Section Geometry

The levee section design geometry is ultimately a function of seepage and embankment stability conditions. Comprehensive geotechnical investigations of the levee design were developed for this study, and are discussed in further detail in the following report subsections. The outcome of these designs was a levee setback design section with the following dimensional characteristics (Figure 31):

Height – Approximately 12ft (ground elevation =6.0-7.0ft, top of levee elevation = 18.0ft)
Levee landward sideslope = 3H:1V
Levee waterward sideslope = 2.5H:1V
Top Width = 12ft (for access road)
Bottom Width = Varies Typical 80ft
Minimum offset from Big Ditch w/ Geogrid = 15ft
Sideslope were evaluated in detail in the levee embankment stability analysis, for which the landward and waterward slopes design recommendations were developed with reasonable factors of safety. The levee top width of 12ft was selected to allow for maintenance vehicle access. The bottom width is a function of levee profile elevation, existing ground elevation, top width and side slope configuration. In general the proposed levee has a base width of 80ft and is generally 20ft wider than the existing levee.

Of certain concern was short term settlement of the levee, especially along the Pioneer Highway roadway embankment. A minimum offset of 15ft from the edge of Big Ditch was specified if a geogrid is used underneath the levee to provide structural stability along the Pioneer Highway section of the structure.

The general levee setback structure has an approximate cut volume of 16,600CY associated with stripping materials and prepping the levee base. These materials will be replaced as part of an overall fill volume placement of volume of 44,000CY.

As the levee fill material is placed, differential settlement will occur along the length of the structure as the weight of the additional soil compresses underlying soils and forces water out from pore spaces. Settlement estimates are discussed further in the following Section 2.2.4.3. The amount of settlement will affect the overall total quantity of fill material necessary to build the levee to the specified design height. The plans and specifications are set up such that the contractor is allowed two points during construction (end of each Phase II and Phase III) to make adjustments in the quantities of levee fill using settlement plate measurements. Essentially this process involves installing bearing plates with steel tubes that will measure levee settlement over time. This amount of material for settlement is then added to the embankment, end area estimate of the levee fill quantity.

Due to the potential for settlement, Pre-Load and Final loading approach will be used where soils are placed in two distinct phases: Phase II Pre-Loading and Phase III Final Loading elements of work. Phase II Pre-Loading will occur in 2010 and Phase III Final Loading in 2011. The initial section of the levee Stations 10+00 to 17+50 will be built in two stages, namely Pre and Final Loads. Pre-Load maximum heights are to an elevation, with full width, of 15.0ft and then Final Loads will be built to an elevation of 18.0ft. All other sections of the levee beyond station 17+50 can be built to full height, as project sequencing allows. There are not structural settlement issues along the areas beyond the Pioneer Highway section of the setback.

The setback levee travels south 750ft paralleling the Big Ditch Realignment and the Pioneer Highway. The levee location has been offset a minimum distance of 100ft from the roadway embankment to limit the effects on Pioneer Highway roadway embankment, as described further in the following Section 2.2.4.2. The 100ft minimum is adjusted from the original offset distance estimates published in previous design reports.

The base of the levee along the Big Ditch realignment will have a 35ft bench, with crushed gravel driving surface to provide additional stability on the backside of the levee next to Big Ditch. This area can be used as a maintenance access area for the emergency spillway and the outlet of the inverted siphon.

In addition to the general levee structure, there is a seepage blanket and cutoff trench of low permeable glacial till clays that will limit seepage through the levee. An additional 12,500CY of earthwork will be necessary for installation of a levee clay slope blanket and cutoff trench. The details on the need and expected performance of the seepage cutoff trench are also included in the following geotechnical report sections. The seepage blanket and cutoff trench will be constructed of low permeable (<1e10^-6cm/s) glacial till.
An overflow spillway is located along this section of the levee between station 15+00 and 16+00 with a crest elevation of 14.7ft. The discharge point of the spillway will occur at an elevation of 14.0ft, through porous rock materials, and is similar in elevation to the existing overflow spillway that currently exists at the Big Ditch culvert crossing of Fisher Slough. The spillway design is described further in Section 2.2.4.8.

The next levee segment runs from station 17+50 to 31+50. This setback segment runs along the abandoned railroad alignment. Topsoils stripped from this section of the levee will not be reused as fill material, due to the extensive amounts of invasive species in this area. Construction will require extensive clearing and grubbing along the abandoned railroad embankment. In addition, minimal soil testing has been performed along the abandoned railroad segment of levee. Soil test pits are required every 100ft along this section of the levee to confirm existing materials conditions. Several large trees on the embankment will be removed from the abandoned railroad. The removal of these trees will require fairly significant grubbing of root systems to ensure the new levee embankment is in good condition. The initial design considered salvage and placement of the large wood debris in the marsh as a restoration element. This option is no longer considered as described in Section 2.2.5.4.

The final segment of construction is station 31+50 to 49+00, which is a Phase II pre-loading section, entirely located on historical farm areas. The levee runs south to station 42+00 and then turns east. The key in of the levee will be to the hill slope at station 49+00. There is a special consideration for stripping depth and material removal and disposal between stations 34+00 and 36+00. Gravelly and sandy soils were discovered in the top 2-3ft of soil samples and need to be completely removed as part of the soil stripping operation. These soils should not be disposed of in the 20,000cy of topsoils on the Jungquist farm area due to high gravel content. The gravelly soils can be used in other backfill areas such as Little Fisher and Big Fisher Creek fill areas. In addition, the short levee segment between 34+00 and 36+00 will also need a 6in thick layer of 4in diameter quarry spall embankment erosion protection. This is described further in Section 2.2.5.10.

Clearing, grubbing and excavation of the key-in area on Moyer Hill will be necessary to fully connect the levee with existing terrain. When the key-in structure is being excavated, the contractor must notify TNC, so that a cultural resource monitor can be present.
Figure 31. Typical Levee Sections for Pre-load and Abandoned Railroad Areas
2.2.4.4 Levee seepage design

A critical element of the levee design is to evaluate seepage through the levee. Excessive seepage rates during flood conditions can cause piping and erosion of levee embankment materials. A particular seepage concern for Fisher Slough is the potential local effects the levee will have on adjacent farm areas and groundwater wells as a result of setting back the levees, and allowing more tidal marsh inundation and changes in flood operations. To address these design issues, the levee seepage analyses are discussed in the following categories:

- Steady state flooding levee seepage analysis
  - Evaluate vertical exit gradients and Corps design criteria
  - Develop input for levee embankment stability analysis
- Unsteady state late spring/early summer seepage analysis to document potential effects on local farm drainage

A separate memorandum has been included with the report that addresses the levee setback project with respect to possible effects on local groundwater wells, for which the reader is referred to App. A.9.

Steady State Flooding Levee Seepage Analysis

Two types of seepage analyses were performed for the project, steady state analysis for fall/winter flooding and unsteady state, transient seepage analysis to assess affects of late spring and early summer flooding on adjacent farm areas. Seepage analysis was performed using SEEP/W, a two-dimensional, finite element model with the ability to evaluate groundwater and seepage flow through numerous soil layers along a design section. SEEP/W provides output necessary to evaluate Corps design criteria data, including vertical exit gradient criteria. Additionally, the SEEP/W modeling pore water pressures are used as input parameters for the embankment stability model SLOPE/W, which is described in the following section of the report.

One of the key design criteria in assessing levee stability is to evaluate vertical exit gradient criteria using the Corps risk factors. For this study, a low risk vertical exit gradient threshold of 0.5 (ft/ft) was selected per the levee design manual (USACE, 2000).

Three cross sections were identified for the SEEP/W and SLOPE/W (SEEP/W & SLOPE/W, 2007) analysis, they are:

- Smith A – North-south levee section representing CAD plan design alignments STA 10+00 to 17+50 (or STA 0+00 to 7+50 per geotechnical alignment)
- Smith A – East-west abandoned railroad section representing CAD plan design alignments STA 17+50 to 32+00 (or 7+50 to 22+00 per geotechnical alignment)
- Smith B – Levee section CAD plan design alignments STA 32+00 to 49+50 (or 22+00 to 59+50 per geotechnical alignment)

Each modeling section contained topographic, subsurface soil strata and permeability input parameters derived from original soil testing. In addition to geometric and soil data, hydrologic data are needed as input to the model. An array of hydrologic inputs was identified for the modeling effort and includes the following:

- Steady state flood conditions (static, no seismic)
  - Fisher Slough Flooding – Fisher Slough water at El. 16.7ft (NAVD88) (Q_{100} WSE maximum), Big Ditch and farm field side of the new levee – water at ground surface (~7.0ft ground elevation and top of Big Ditch water surface elevation). This is the most
likely, worst case flood condition with Fisher Slough water stage high for a significant period of time with lower stages in Big Ditch occurring.

- Big Ditch Flooding - Fisher Slough water at ground surface (~7.0ft ground elevation), exterior farm field side of the new levee – water at El. 16.7ft (NAVD88) (Q_{100} WSE maximum). This condition is not likely to occur but was evaluated for potential Big Ditch flooding with lower Skagit River conditions.

- End of construction conditions – Fisher Slough water elevation at 16.7ft (NAVD88) (Q_{100} WSE maximum) and Big Ditch and farm field side of the new levee water at 1.0ft (minimum measured water surface elevation in Big Ditch). This would represent a worst case early flood in Fisher Slough with minimal water in Big Ditch.

- Steady state flood conditions (seismic)
  - Fisher Slough average MHHW – Fisher Slough water at elevation 9.5ft (MHHW), Big Ditch and farm field side of the new levee at ground surface elevation 7.0ft. This case examined levee stability during a seismic event for typical high water conditions.

- Unsteady state conditions (static, no seismic)
  - Rapid drawdown conditions of Fisher Slough side of new levee – Drop in water surface elevations from 16.7ft to 10.7ft over 24 hour period. 6.0ft head differential is equivalent to downstream floodgate head differential structural design. This would simulate a rapid drawdown event for Fisher Slough where the Skagit drops a significant amount in 24 hours. Typical falling limbs on Skagit are on the order of multiple days to weeks, so this evaluation is considered conservative.

The key findings are summarized as follows (Figures 32-39, Table 10):

- Smith A along the Pioneer Highway, vertical exit gradients exceed the Corps design criteria due to the shallow underlying soil layer 2B. The high exit gradients exist at the toe of the levee for both Fisher flooding and Big Ditch flooding conditions. Seepage control measures are necessary to reduce exit gradients for this levee section.

- Smith A along the abandoned railroad section of the levee has low exit gradients and meets the Corps 0.5 (ft/ft) vertical exit gradient criteria and does not require seepage control based on the modeling results.

- Smith B has low exit gradients and meets the Corps 0.5 (ft/ft) vertical exit gradient criteria and does not require seepage control based on the modeling results.
In response to the need for seepage control, a number of measures were identified that could improve seepage conditions through the levee. These included evaluating a sheet pile cutoff, a clay slope blanket and cutoff trench, and low permeability soil blankets on the levee waterward side and a Geosynthetic Clay Liner (GCL) and weighted berm on the levee landward side that extending out from the toes of the levee. These low permeable alternatives were examined and compared for performance in the Smith A area, as shown in Table 10 as a summary table at the end of the following report Section 2.2.4.5 Levee Embankment Stability Design.

Dike District #3 prefers the use of a clay blanket and cutoff trench as a standard design feature being used on their structures, and this alternative is being brought forward as the preferred design solution. There is some concern regarding the need for the clay blanket and cutoff trench along the entire structure, and the associated costs. The current design has included the clay slope blanket and cutoff trench, as a preferred method and higher cost, even though the toe blanket and geosynthetic clay liner may be more effective at treating localized exit gradients on the landward levee toe. The reason for this selection is that the GCL and berm methods are not currently a local design standard, and use of unknown technology had a level of unacceptable uncertainty. This may become an issue and could be revisited if bid estimates are more than the project budget.

Unsteady State Late Spring/Early Summer Levee Seepage Analysis
A secondary seepage analysis was performed to evaluate the effects of flooding on local farm fields during late spring and early summer runoff events on the Skagit River. During levee design seepage design review, concerns were brought forward by the DD3 and local landowners reviewing the project regarding possible groundwater mounding effects on adjacent farm properties that may occur from setting back the levees. The understanding of the concerns at hand were the development of high groundwater tables during late spring or early summer farming periods that could effect plowing, planting and seed/root saturation and could cause crop damage.

The key period and type of runoff event that is of concern are when the Skagit River has peak flows occurring in the May through July period. Specifically for the Fisher Slough project, the June and July peak flows cause concern for the levee setback project as the downstream floodgates are fully opened on June 1, each year.

A review of Skagit River, Mt. Vernon USGS gage data, and Fisher Slough project hydrologic data were evaluated to characterize how often, and what types of flow occur during these events. During the Spring/Summer of 2008, Fisher Slough had late spring/early summer floods, for which the project was collecting surface water data at the site. This data was correlated to the Mt. Vernon gage, for which the record was reviewed from 1988 through 2009 to identify the number of occurrences when the Skagit rises above a target elevation (stage 24.0ft at Mt. Vernon gage ~ 10.3ft NAVD88 at Fisher Slough) that would be considered problematic from a seepage perspective. The following four flow events were observed in the recent 25 year period:

- May 18-21, 2008
- June 30-July 4, 2008
- June 29, 2002
- June 1, 1997

The initial screening analysis indicate that the late season peaks occur 1 in every 5 to 10 years.
The Mt. Vernon and the Fisher Slough gage data were analyzed in detail to develop “representative” inflow hydrograph conditions for the transient seepage analysis based. This involved averaging the rising limb, peak, and falling limb durations, and estimating the average peak flow elevation. The following late season flow hydrograph was developed for the transient model input:

- Average rising limb duration – 3 days
- Average peak stage – 12.0ft (equivalent to the OHW)
- Average peak duration – 4 days
- Average falling limb duration – 4 days
- Total hydrograph duration – 11 days

The flow hydrograph was then used as input to the SEEP/W model to evaluate the groundwater response in the adjacent farm fields. More details on the model configuration and boundary conditions are provided in App. A.5.1 Levee Design – Transient Seepage Analysis. For reference, the starting groundwater surface elevation is at 2.1ft, which is approximately 4.9ft below grade.

The transient seepage modeling analysis evaluated four alternatives including the following configurations at Smith B:

- Smith B without clay blanket and cutoff, and without a toe drain
- Smith B with clay blanket and cutoff, and without a toe drain
- Smith B without clay blanket and cutoff, and with a toe drain
- Smith B with clay blanket and cutoff, and with toe drain.

In general, all configurations show a 4ft to 5ft rise in groundwater adjacent to the levee for a distance of 45 to 100ft out into the farm field from the toe. The results are described herein. Figure 40 shows the water surface profile response to the late spring, early summer 11-day flood event.

**Smith B (without clay blanket/cutoff and without toe drain)**
The levee only alternative does show that groundwater mounding occurs and the groundwater gradient exceeds the ground surface, indicating that full saturation and ponding would occur on the farm side of the levee. The model predicts more than 6in of ponding would occur. The distance from the toe of the levee where the groundwater mound is more than 1ft below the ground surface is on the order of 77ft.

**Smith B with clay blanket/cutoff and (without toe drain)**
The levee with clay blanket and cutoff (without a toe drain) alternative also shows that groundwater mounding occurs and the groundwater gradient exceeds the ground surface, indicating that full saturation and ponding would occur on the farm side of the levee. The model predicts 1in of ponding would occur. The distance from the toe of the levee where the groundwater mound is more than 1ft below the ground surface is on the order of 61ft.

**Smith B (without clay blanket/cutoff) and with toe drain**
The levee (without clay blanket and cutoff) and with a toe drain alternative shows that groundwater mounding occurs, but the groundwater gradient remains below the ground surface. This indicates that the soils are nearly fully saturated, with the groundwater table less than 1in from the ground surface. The distance from the toe of the levee where the groundwater mound is more than 1ft below the ground surface is on the order of 62ft.

**Smith B with clay blanket/cutoff and with toe drain**
The levee with clay blanket and cutoff and with a toe drain alternative shows that groundwater mounding occurs, but the groundwater gradient remains the lowest below the ground surface. This indicates that the soils are partially saturated, with the groundwater table less than 5in from the ground surface. The distance from the toe of the levee where the groundwater mound is more than 1ft below the ground surface is on the order of 45ft.

In addition to the transient analysis, the original seepage exit gradient and embankment stability analysis were updated to reflect the changes in the levee configuration. This information combined with cost information has also been included in the Table 8 seepage design summary table at the end of Section 2.2.4.5 Levee Embankment Stability Design.

Based on the outcome of the steady state and transient seepage analysis, it has been decided to use a clay slope blanket and cutoff trench along the entire structure. For the Smith B, north-south section of levee, a toe drain will be installed to lower groundwater tables during late spring/early summer flood conditions. It is also noted that a recommendation is included to evaluate the potential for engaging the downstream floodgate during late spring and early summer flood events at key farm periods. This would require additional coordination and consultation with the project team and regulatory agencies prior to initiation and is a good adaptive management topic for DD#3, DD#17 annual drainage fish and maintenance review meetings.
Figure 32. Steady state seepage analysis for Smith A along Pioneer Hwy without seepage protection

Average Exit Vertical Gradient = 0.54
Figure 33. Steady state seepage analysis for Smith A along Pioneer Hwy with seepage protection, marsh side toe blanket and ditch side GCL and berm
Figure 34. Steady state seepage analysis for Smith A along Pioneer Hwy with seepage protection, sheet pile cutoff wall
Figure 35. Steady state seepage analysis for Smith A along Pioneer Hwy with seepage protection, clay slope blanket and cutoff trench
Figure 36. Steady state seepage analysis for Smith A along Abandoned Railroad without seepage protection
Figure 37. Steady state seepage analysis for Smith A along Abandoned Railroad with seepage protection, clay slope blanket and cutoff trench
Figure 38. Steady state seepage analysis for Smith B without seepage protection
Figure 39. Steady state seepage analysis for Smith B with seepage protection, clay slope blanket and cutoff trench
Figure 40. Unsteady state seepage analysis for Smith B alternatives
2.2.4.5 **Levee embankment stability design**

Slope stability analyses were performed using SLOPE/W (SEEP/W & SLOPE/W, 2007), which is a computer program for the general solution of slope stability problems by two-dimensional limiting equilibrium methods. The calculation of the factor of safety against instability of a slope can be performed using one of the following methods: Bishop Simplified Method (applicable to circular shaped failure surfaces), Ordinary Method, Janbu Simplified Method (applicable to failure surfaces of general shape), or Spencer's Method (applicable to any type of surface) The minimum factors of safety (FS) for static conditions required by the US Army Corps of Engineers (USACE, 2000) are shown in **Table 8**. The required minimum FS under seismic conditions is generally 1.1 (WSDOT, 2008).

<table>
<thead>
<tr>
<th>Design Condition</th>
<th>Minimum Factor-of-Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of Construction</td>
<td>1.3</td>
</tr>
<tr>
<td>Rapid Drawdown</td>
<td>1.0 to 1.2 *</td>
</tr>
<tr>
<td>Long Term (Steady Seepage)</td>
<td>1.4</td>
</tr>
</tbody>
</table>

* Sudden drawdown analysis. F.S. = 1.0 applies to pool levels prior to drawdown conditions where these water levels are unlikely to persist for long periods preceding drawdown. F.S. = 2.0 applies to pool level likely to persist for long periods prior to drawdown.

SLOPE/W features unique random techniques for generation of potential failure surfaces for subsequent determination of the more critical surfaces and their corresponding factors of safety. These techniques generate circular failure surfaces, surfaces of sliding block character, or more general irregular surfaces of random shape. For the purposes of these analyses, URS utilized Spencer’s Method. The pore pressures generated in the SEEP/W analyses were used in SLOPE/W stability analysis.

The same section locations and boundary conditions listed above in the seepage analysis section were used for the stability analysis. Soil parameters (friction angle, cohesion and unit weight) that were used in the analyses were based on field data, laboratory results, and engineering judgment, and are summarized in **Table 9**.

<table>
<thead>
<tr>
<th>Soil Unit</th>
<th>Unit Weight (pcf)</th>
<th>Cohesion (psf)</th>
<th>Friction Angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stratum 1&lt;sup&gt;b&lt;/sup&gt;, SILT</td>
<td>110</td>
<td>500&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0</td>
</tr>
<tr>
<td>Stratum 2A&lt;sup&gt;a&lt;/sup&gt;, clayey SILT</td>
<td>95</td>
<td>250&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0</td>
</tr>
<tr>
<td>Stratum 2B&lt;sup&gt;a&lt;/sup&gt;, SILT/sandy SILT/silty SAND</td>
<td>100</td>
<td>250&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0</td>
</tr>
<tr>
<td>Stratum 3, silty SAND</td>
<td>110</td>
<td>0</td>
<td>28</td>
</tr>
<tr>
<td>Existing Railroad Fill</td>
<td>120</td>
<td>2000</td>
<td>0</td>
</tr>
<tr>
<td>New Fill</td>
<td>120</td>
<td>500</td>
<td>36</td>
</tr>
</tbody>
</table>

<sup>a</sup> used as minimum strength in this analyses

<sup>b</sup> S<sub>k</sub>/p’ ratio of 0.33 was used in this analyses

The proposed setback levee consists of a compacted earthfill with the following cross section: side slopes of 2.5H:1V on the Fisher Slough side, side slopes of 3H:1V on the Big Ditch side, with a top width of twelve feet, and a profile elevation of 18.0ft.
Overall, the results indicate that the proposed slopes meet the required factor of safety. However, the results of the end of construction stability analyses indicate that the section of levee along Smith A, Big Ditch and Pioneer Highway is in an area of soft silts and silty sands with low strengths (Appendix A.1 – Geotechnical Tech Memo). However, the section of the levee setback along Smith A, Big Ditch and Pioneer Highway, CAD plan alignment station 10+00 to 18+00 (geotechnical sampling stationing 0+00 to 8+0) were lower than the accepted factor of safety of 1.3. Because of soil conditions near Fisher Slough additional measures are required for the levee to meet the stability requirements. The maximum height of construction allowed in the first construction season is to Elevation 15.0 (approximately nine (9) feet of fill) with a minimum levee setback 15 feet from the edge of the Big Ditch excavation, using an underlying geogrid for foundation reinforcement. This reduced levee height will allow initial dike settlement to occur and some increase in foundation strength prior to completing the levee fill to the design elevation.

The results of the long term and rapid drawdown stability analyses satisfactorily meet the minimum acceptable FS in all cases except along Pioneer Highway - Big Ditch section, which was borderline acceptable. The stability of the dike slope in this area will be improved by the following options:

- Maximum fill height of 15ft for Phase 2 fill placement,
- Reinforce the embankment (placing geotextile/geogrid layers at the base of the levee) with the a minimum 15 foot levee setback from Big Ditch and,
- Maintain a minimum 30 foot berm between the toe of the levee and Big Ditch without geogrid foundation reinforcement.

These options were evaluated and will provide adequate factors of safety. The original approach was to use a 15ft offset and geogrid to maximize the tidal marsh area. Recent design revisions have moved the levee to a 35ft offset, and the geogrid is remains for an additional factor of safety in this difficult design and construction area.

Seismic stability of the levee considered the “post-shaking” factor of safety. In this analysis, a residual post-earthquake undrained strength was assigned to potentially liquefiable layer stratum 2B. A reduction of 15% for the shear strength for strata 2A due to shaking induced elevated pore pressures.

The residual post-earthquake undrained strengths of the Stratum 2B below the groundwater table were estimated based on equivalent Standard Penetration Test (SPT) blow counts (N). Based on Idriss and Boulanger (2008) approach, a residual post earthquake strength of 165 psf was estimated for the stratum 2B deposits.

The results of the post-shaking stability analyses indicate that calculated FS met the minimum acceptable FS in all cases, except parallel to Pioneer Highway. At the north end of the setback levee where factors of safety of less than one were obtained for the “Post-shaking” stability analyses for the design seismic event, potentially large vertical and lateral deformations of the levee could occur due to liquefaction in the Stratum 2b sandy native foundation soil. The liquefaction is likely to be discontinuous due to the variable fines content of this sublayer.

Estimating the magnitude of deformations that could occur as a result of the liquefaction is beyond the scope of this effort. However, experience in California (Miller and Roycroft, 2004) and (T. Kokusho, 2006) have shown that portions of the levee could settle and move laterally perhaps less than 10 to more than 30 percent of the height of the levee when foundation liquefaction occurs, which is a lateral range of 1 to 4ft.

The use of geogrid layer in the base of the levee is expected to reduce the seismic deformations. Substantial post-earthquake repair could be needed. Alternative ground improvement methods such as
vibrocompaction, vibroreplacement or soil mixing could be employed during construction to the vulnerable zones of the foundation to minimize the magnitude of seismic displacement. However, due to settlement and deformation concerns for Pioneer Highway and the Fisher Slough Bridge, vibratory compaction and installation methods are not allowed along the highway or bridge areas to prevent settlement or deformation of the adjacent structures.

The results and figures for the embankment stability analysis are included in App. A.1.

### 2.2.4.6 Pioneer Highway Embankment Stability

The purpose of this analysis is to evaluate the effect of settlement of the new levee and the excavation of Big Ditch on the Pioneer Highway embankment. Calculations show that the minimum distance between the toe of the levee and the toe of highway embankment must be 80 feet to limit the settlement of the edge of the highway embankment to less than 0.5 inch. This includes the assumed top distance of 45 feet for Big Ditch (2.5H:1V side slopes, 15 feet bottom width), and a minimum 15ft offset between Big Ditch and the levee toe.

Based on stability calculations of Pioneer Highway, the minimum required offset of Big Ditch from Pioneer Highway is twenty (20) feet from the toe of the highway embankment to the edge of the Big Ditch excavation in the area south of the bridge at Fisher Slough (STA 10+00 to 17+50). A minimum ten foot offset is required for Big Ditch from the Pioneer Highway embankment to the north of the Fisher Slough bridge.

All embankment offset criteria are met or exceeded in the current design plans. If settlement of the edge of roadway does occur, a contingency for asphalt cracking and repair could be provided. In order to assess project effects, the edge of roadway condition should be documented (photographed) prior to the start of construction.
Table 10. Summary of seepage modeling results

<table>
<thead>
<tr>
<th>Location</th>
<th>Seepage Design Alt.</th>
<th>Avg. Exit Gradient</th>
<th>Meets Corps Criteria</th>
<th>Long Term Embankment Stability FS</th>
<th>Transient Farm Drainage Conditions</th>
<th>Distance From Levee Toe to GW Depth @ 1ft</th>
<th>Additional Cost²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>W/O Seepage Protection</td>
<td>0.54</td>
<td>No</td>
<td>1.45</td>
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<td>-</td>
<td>-</td>
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<tr>
<td></td>
<td>Impermeable Berm/Blanket</td>
<td>0.36</td>
<td>Yes</td>
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<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Sheet Pile Cutoff</td>
<td>0.43</td>
<td>Yes</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Smith A</td>
<td>Clay Blanket &amp; Cutoff Trench</td>
<td>0.50</td>
<td>Yes</td>
<td>1.72</td>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>W/O Seepage Protection</td>
<td>0.21</td>
<td>Yes</td>
<td>1.52</td>
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<td>-</td>
<td>-</td>
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<tr>
<td>Abandoned</td>
<td>Clay Blanket &amp; Cutoff Trench</td>
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<td>Yes</td>
<td>1.58</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>RR</td>
<td>W/O Seepage Protection</td>
<td>0.38</td>
<td>Yes</td>
<td>1.57</td>
<td>-6</td>
<td>Above 77</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Clay Blanket &amp; Cutoff Trench</td>
<td>0.36</td>
<td>Yes</td>
<td>1.61</td>
<td>-1</td>
<td>Above 61</td>
<td>-</td>
</tr>
<tr>
<td>Smith B</td>
<td>Toe Drain</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>Below 62</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Clay Blanket &amp; Cutoff Trench &amp; Toe Drain</td>
<td>-</td>
<td>-</td>
<td>5</td>
<td>Below 45</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

1 Corps criteria for low risk for vertical exit gradients is 0.5ft/ft. Evaluated Fisher Slough flooding WSE at 16.7ft with Big Ditch levee backside WSE at 7.0ft.

2 Additional costs beyond base include 7.8% tax and 10% contingency, except Smith A cutoff includes 20% contingency for unstable soils and high groundwater seepage conditions.
2.2.4.7 Levee embankment erosion protection

It is often the case that certain locations along the levee could experience high velocities, and embankment protection may be necessary to prevent erosion of the levee embankment. For Fisher Slough, the two most likely locations are the areas where the main tidal channel abuts the levee, which coincidentally is located where the floodplain and marsh areas constrict to a narrower width.

Design of embankment protection evaluates the potential for erosion of the embankment materials directly or scour of the embankment toe, which could cause undermining and further erosion of the embankment. The maximum predicted velocity in the system was 2.9fps with a maximum shear stress of 0.071 for the Fisher Slough main tidal channel and 0.123psf for the Smith B pilot channel. Permissible shear stresses for silty/sandy material range from 0.024psf to 0.072psf (FHWA, 2005). Using a mid-range permissible shear stress of 0.050psf, the criteria are exceeded ~1.2% of the time. Therefore, some minor scour and erosion is expected in a few hot-spot areas.

The project site was screened for potential hot spots and likely scour/erosion areas of the levee. These areas were specifically identified as those where channels flow along the toe of the levee and where the levee and floodplain constrict a significant distance. Three specific areas were identified in the screening process:

- Main channel of Fisher Slough, 200ft upstream of Pioneer Highway Bridge in Smith A where floodplain necks down to main tidal channel only. S. Levee setback STA 10+00 (CAD Sheet C11)
- S. Levee setback between Smith A and Smith B, STA 31+00 to 32+00 (CAD Sheets C12 and C16) where the entire floodplain necks down.
- S. Levee setback STA 35+00 (CAD Sheet C16) where the Smith B pilot channel is closest to the levee.

The peak shear stress at the Smith A levee sections near STA 10+00 was 0.071psf, which corresponds to a critical diameter of 0.14in using the Shields equation for incipient motion analysis.

The S. Levee setback area between Smith A and Smith B, STA 31+00 to 32+00 will be located behind a stand of dense trees and vegetation that will be preserved during construction. Erosion and encroachment on the levee is not expected in this area.

The short section along S. Levee setback STA 35+00 has beach shear stresses of 0.123psf, which corresponds to a critical diameter of 0.31in using the Shields equation for incipient motion analysis.

An additional 6in layer of 4in quarry spalls extending from an elevation of 10.0ft down to an embedded elevation of 0.0ft can be applied at the areas of concern. The locations are:

- Smith A inverted siphon crossing (both north and south levee reconstruction areas) and extending the protection on the south levee setback protection to STA 10+50.
- The same type of material and embedment is also recommended along STA 34+00 to 36+00 to provide protection from the Smith B Pilot Channel.
- The erosion protection material shall be placed on top of and waterward of the clay seepage slope blanket, and buried beneath the 6in topsoil and hydroseed mulch layers of materials.
2.2.4.8 Levee interior drainage

Typically when designing a levee, the interior drainage needs to be assessed. The Fisher Slough levees drain all flow to the floodgates at the Pioneer Highway. Localized ponding is not likely to occur on the site that would be problematic for levee soils. This is because the toes of the levees will be graded gently towards pilot channel and tidal marsh drainage channels. Interior drainage design is not required for this levee system.

As for interior farm drainage areas to the north and south of the levees are concerned, the Big Ditch inverted siphon structure provides significant improvements in drainage as compared with the existing culvert crossing. To the south, drainage will be improved (or maintained) through the installation of a clay slope blanket and cutoff trench and backside levee toe drain.

2.2.4.9 Levee closures and connections

Other areas that can be of concern for a levee system are closures and connections. The floodgate is the primary system closure and has been retrofit and had engineering design completed on Phase I work. The downstream connection is to high ground and the main levee system at the Pioneer Highway Bridge. The upstream connection is to high ground in the southeast corner of Smith B. The levee setback is fully connected with openings at the floodgates and temporary overflow periods.

The only other penetration in the levee system in the project area is the north levee Skagit River flood return flow gates, which will not be affected by the project.

2.2.4.10 Levee access, ingress, egress

Access to the levee can be gained through the Smith A railroad bridge crossing, from Pioneer Highway over Big Ditch, and from ingress/egress ramps at the levee corner on the Smith B segment of the levee. All access ramps are designed with a maximum grade of 10% slope. The north end of the levee setback can also be accessed in emergency situations at the Pioneer Highway bridge crossing of Fisher Slough. This may be necessary in the future if access is to the northern most portions of the levee is needed, and the emergency spillway is flowing. In addition, access gates will be located at the Smith A and Smith B ingress entrances onto the levee to provide security from random vehicle access.

2.2.4.11 Levee Toe Grading

The levee toe will be graded from an elevation of 8.0ft down to the marsh floodplain surface at a minimum slope of 2%, and drop in elevation for 1-2 ft, with an approximate width of 30ft along the disturbed work areas at the levee toe. Areas below 8.0ft in elevation will be seeded with high marsh species, due to the frequent tidal inundation occurring in the area at the toe of the levee. The toe grading will promote drainage away from the levee toe, and reclaim/restore areas where soils may be compacted from construction equipment. In addition, the toe grading areas will provide high marsh habitat.

2.2.4.12 Levee Overflow Spillway

The levee setback will have a 160ft long overflow spillway similar in elevation and width to the existing overflow spillway between Station 14+55 and 16+24 (CAD Sheet C11). The existing spillway is currently located at the existing Big Ditch Culvert Crossing Fisher Slough, and has a crest elevation of 14.7ft. Due to the porous material and spillway configuration, spills will occur at an elevation of 14.0ft (through the porous material).

An engineering design analysis was performed on the proposed emergency spillway on the South Levee Setback structure for the Fisher Slough Project. The design analysis includes the following elements:

- Evaluation of existing spillway capacity
- Sizing and evaluation of the S. Levee Setback spillway capacity
- Design of protective riprap for spillway elements and material specifications
- Design of filter fabric installation
- Grading design of spillway structure

**Evaluation of the existing spillway capacity**

The emergency spillway is located at the current Big Ditch culvert crossing **Figure 41**. When Fisher Slough reaches flood stage (at approximately an elevation of 14.0ft) the overflow spillway is engaged and spills from Fisher Slough, and plunges onto the downstream concrete apron of the Big Ditch sag-culvert crossing. The spillway was originally designed to have 12”x12” stop logs placed in slots at the top of the structure. The boards were removed (taken) from the project site, and have not been used in the last 15-20 years. The current spillway is allowed to overflow at an elevation of 14.0ft.

The dimensions of the spillway include a 34ft wide section, as shown on the 1935 as-builts (Skagit County, 2009), with (2) 2ft wide concrete stop log walls. The effective width of the existing overflow spillway is 30ft wide. It is noted in **Figure 41** that the crest of the spillway is overgrown with thick grass and weeds, and limits the flow across the spillway. The estimate provided herein is likely generous to the actual amount of flow across the spillway. The existing spillway discharge capacity was determined using a broad-crested spillway, and estimated at 400cfs (**App. A.3**).

![Fisher Slough Spillway](image)

**Figure 41. Fisher Slough Existing Emergency Spillway and Big Ditch Culvert Outlet**
Sizing of the S. Levee Setback spillway capacity

An emergency overflow spillway is being designed for the Fisher Slough South Levee Setback structure. This structure is being designed to exceed the existing spillway capacity. A number of methods were proposed by the Diking and Drainage District on the configuration of the spillway, including long, depressed levee sections with sheet flow on the backside of the structure.

Due to site and grading constraints, a shorter rock spillway section is proposed by the design engineers. A main spillway length of 120ft was selected. The estimated total discharge of 1,722cfs for the proposed spillway is 4.0 times the estimated discharge of the existing spillway of 400cfs. The spillway design capacity is slightly greater than estimated 24hr, 100year, routed flood peak estimate for Hill Ditch, Big Fisher and Little Fisher Creeks combined which is 1,680cfs for the future build out conditions in the watershed.

Design of protective riprap for spillway embankment

The spillway design includes placement and protection of rock in three key areas:

- Spillway crest/dip crossing
- Spillway slope embankment
- Spillway toe rock scour depth

For each of these areas, the following design criteria were evaluated:

- Rock size
- Filter size and ratios
- Layer thickness

Rock Sizing Method Comparisons

A number of methods were used to determine the rock size, filter and gradation ratios, and layer thicknesses. These methods are described in detail in App. A.5. For the embankment slope spillway rock the estimated D$_{50}$ weight of 2,332 lbs which is roughly a 3.0ft diameter rock (1.16tons). The WSDOT specifications were reviewed to determine if a standard specification would be appropriate for use at the embankment. The heavy loose riprap specified in WSDOT Standard Specifications Section 9-13.1(1) was examined for use at the site. Unfortunately, the heavy loose riprap has no maximum material sizes, which can lead to having extremely large material being placed on site. A spillway embankment specification was therefore developed for this project. The specification has broad gradation coefficients (Gc) and coefficient of uniformity (Cu) to assist with filter design requirements. At a minimum, these shall not fall below a value of 2 to maintain a well graded material and provide filtering (Table 11, Figure 42).
Table 11. Spillway Embankment Riprap Rock Specification

<table>
<thead>
<tr>
<th>Rock Dia. (in)</th>
<th>Rock Dia. (ft)</th>
<th>Rock Weight (lbs) ( (\gamma=165\text{pcf}) )</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Max</td>
</tr>
<tr>
<td>48</td>
<td>4.0</td>
<td>5,529</td>
<td>100%</td>
</tr>
<tr>
<td>36</td>
<td>3.0</td>
<td>2,333</td>
<td>90%</td>
</tr>
<tr>
<td>24</td>
<td>2.0</td>
<td>691</td>
<td>60%</td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
<td>1</td>
<td>10%</td>
</tr>
</tbody>
</table>

Gc = 6.7  
Cu = 8.0

Figure 42. Fisher Slough Spillway Embankment Riprap Material Specification

Spillway Slope Embankment Rock Layer Thickness
Rock placement layer thickness is typically evaluated using $1.5 \text{D}_{50}$ to $2.0 \text{D}_{50}$ with a minimum thickness of $1.0 \text{D}_{100}$ (USACE 1994, USBR 2007). The $\text{D}_{50}$ has been selected as 3.0ft. A thickness factor of $1.5 \times \text{D}_{50}$ of 4.5ft (also meeting the $1.0 \times \text{D}_{100}$ criteria) of 4.5ft is considered adequate for the design (Table 12).

### Table 12. Rock Layer Thickness Evaluation

<table>
<thead>
<tr>
<th>Thickness Factor</th>
<th>Layer Thickness (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$1.5 \text{D}_{50}$</td>
<td>4.5</td>
</tr>
<tr>
<td>$2.0 \text{D}_{50}$</td>
<td>6.0</td>
</tr>
<tr>
<td>$1.0 \text{D}_{100}$</td>
<td>4.0</td>
</tr>
</tbody>
</table>

**Filter and bedding material design for riprap**

The placement of riprap will need bedding and filter materials to prevent soil erosion and piping from beneath the riprap placement. A filter fabric layer will be placed on the levee embankment soils to allow for both filtering of soil materials, as well as maximizing drainage of the embankment. A Mirafi 180N geotextile material has been specified to be placed between the embankment soils and the rock bedding material.

Bedding materials lying between the geotextiles and the riprap spillway embankment materials must provide a foundation for which to place and seat the riprap, while providing some level of filtering so that the bedding materials are not lost through piping through the riprap (FHWA, 1995b).

The WSDOT Gravel Borrow material specification 9-03.14(1) was identified as a likely candidate for good riprap bedding material and possible filter layer. However, due to the size of the riprap material, a slightly coarser material was needed for the smaller size fractions. The following table summarizes the specified riprap bedding/filter material for the embankment. This material will be laid a minimum thickness of 6" deep (more than $\text{D}_{100}$ or $2\text{D}_{50}$). The filter criteria are 3.0 and 12.6 respectively for the bedding to spillway embankment material (Table 13, Figure 43).

### Table 13. 4” Minus, Quarry Spall Riprap Bedding/Filter Material Specification

<table>
<thead>
<tr>
<th>Rock Dia. (in)</th>
<th>Rock Dia. (ft)</th>
<th>Rock Weight (lbs) ($\gamma=165$pcf)</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Max</td>
</tr>
<tr>
<td>4</td>
<td>0.33</td>
<td>3.1998</td>
<td>100%</td>
</tr>
<tr>
<td>2</td>
<td>0.16</td>
<td>0.4000</td>
<td>100%</td>
</tr>
<tr>
<td>0.5</td>
<td>0.04</td>
<td>0.0000</td>
<td>20%</td>
</tr>
</tbody>
</table>
Spillway Crest Rock Sizing Check

Rock along the spillway crest will need to be installed in such a manner that vehicles can travel across the spillway during dry periods. A separate design analysis was performed to check the use of quarry spall, 4” minus riprap bedding/filter material. A Shield’s incipient motion analysis was performed to size the critical diameter of rock during the estimated 100-year flood event. Shear stresses were determined by evaluating the water surface slope across the spillway using the upstream 16.7ft water surface elevation and the assumed downstream critical water surface elevation. Using this analysis, a $D_c$ of 2.2ft is needed along the spillway crest which is assumed equivalent to the $D_{100}$. For specification purposes, this will be assumed to be 2.5ft rock. The $D_{50}$ will be selected as 2.0ft diameter rock. This rock specification is too large to drive across. Instead, the approach will be to sweep and compact the riprap bedding, filter and quarry spall materials into the spillway rock. These rocks will need to be periodically replaced after spill events occur (Table 14, Figure 44).
Table 14. Spillway Crest Rock Material Specification

<table>
<thead>
<tr>
<th>Rock Dia. (in)</th>
<th>Rock Dia. (ft)</th>
<th>Rock Weight (lbs) (γ=165pcf)</th>
<th>Percent Passing</th>
<th>Max</th>
<th>Min</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>2.50</td>
<td>0.0000</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
</tr>
<tr>
<td>24</td>
<td>2.00</td>
<td>0.0000</td>
<td>100%</td>
<td>50%</td>
<td>50%</td>
</tr>
<tr>
<td>12</td>
<td>1.00</td>
<td>0.0000</td>
<td>20%</td>
<td>0%</td>
<td>0%</td>
</tr>
</tbody>
</table>

Figure 44. Fisher Slough Riprap Bedding/Filter Material Specification

Spillway Crest Rock Layer Thickness
Rock placement layer thickness is typically evaluated using 1.5 D₅₀ to 2.0 D₅₀ with a minimum thickness of 1.0D₁₀₀ (USACE 1994, USBR 2007). A thickness factor of 1.5 D₅₀ of 2.3ft is considered adequate for the design (Table 15).
Table 15. Spillway Crest Rock Layer Thickness Evaluation

<table>
<thead>
<tr>
<th>Thickness Factor</th>
<th>Layer Thickness (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 D$_{50}$</td>
<td>2.3</td>
</tr>
<tr>
<td>2.0 D$_{50}$</td>
<td>3.0</td>
</tr>
<tr>
<td>1.0 D$_{100}$</td>
<td>4.0</td>
</tr>
</tbody>
</table>

**Toe down design of protective riprap for spillway**

In designing the spillway embankment, several studies were reviewed regarding toe down requirements (App. A.5). One study specifically evaluated toe rock performance in relation to the embankment and spillway rock size. The study consistently found that the rock placed on the steep embankment was much less stable than toe rock of similar size. The design guidelines therefore specify that the toe rock be similarly size to the embankment spillway rock. For this study, toe rock will be sized to a D$_{50}$ of 3.0ft, similar to the spillway rock.

For the S. Levee Setback, the depth of scour was estimated at the bottom of the spillway at a depth of 3.2ft, as measured from the water surface. Using a factor of safety of 2.0, the depth of scour from the water surface is 6.4ft, which is slightly (0.4ft) lower than the bed of Big Ditch if a 6.0ft tailwater condition exists. One of the complicating factors in designing the toe down is that the downstream tailwater condition is not known. Likely it will be fairly high and at a minimum bankfull during runoff conditions (which is essentially 6.0ft deep). This would tend to indicate that a toe down is not required.

During engineering review by Tetra Tech, senior design engineers recommended as a standard of practice that toe-downs should be placed a minimum depth of 3.0ft, which has been added to the project design plan. Rock will be keyed into an elevation of -3.0ft, which is below the invert elevation of Big Ditch at 0.0. The key in elevation refers to the top of the rock blanket at the end of the structure.

**Emergency Spillway Rock Design Elevation Targets**

The final step in design of the emergency spillway is establishment of the target elevations of the spillway rock (Figures 36-37). The intent is for the spillway rock to begin spilling at an elevation of 14.0ft. The top of the spillway will be constructed slightly higher, but will still discharge at an elevation of 14.0ft through the porous spillway crest material. The discharge elevation will be controlled by the underlying seepage clay blanket, built up to an elevation of 14.0ft along the spillway crest. In order to meet thickness requirements of the spillway rock, mixed and compacted with quarry spalls, it will be placed up to an elevation of 14.7ft. These materials will effectively “leak” due to their porous nature at elevation 14.0ft, but surface flows will begin at 14.7ft. An 8.0in layer of 4” minus bedding materials will be spread and compacted into the spillway crest to form a driving surface across the spillway. These materials will need to be periodically replaced after spill events.
Figure 45. Emergency Spillway Plan & Profile
Figure 46. Emergency Spillway Section
2.2.4.3 **Levee and Spillway O&M Requirements**

The levee setback operations and maintenance requirements are as follows:

- Annual mowing (number of times and schedule to limit RCG infestations)
- Annual herbicide applications for RCG control
- Annual removal of trees and shrubs in levee prism, and buffer zones as specified in recent Corps levee vegetation guidance (USACE, 2009) that could promote adverse seepage and piping through the levee.
- Annual removal of rodents and burrow holes that could promote adverse seepage and piping through the levee.
- Annual repair of erosion.
- Annual inspections by the Diking District documenting information necessary to maintain PL84-99 Corp program eligibility
- Profile survey each year for years 1-3, year 5 and then every 5 years to measure settlement. Provide survey to TNC and project partners.
- Periodic replacement of levee embankment materials to raise the profile back to design grade accounting for settlement

All levee features to be inspected per the Corps Levee Owner’s Manual (USACE, 2006) to remain compliant with the Corps PL84-99 program.

An additional item is updating the status of the levee in the PL84-99 program. Mr. Doug Weber of the Seattle District Corps of Engineers was consulted on what activities were necessary to update the project in the PL84-99 program. Dike District #3 will need to submit a letter describing the changes in the project and levee configuration, and submit simple plans, design cross sections of levee prism, emergency spillway, emergency spillway and floodgate to update the project. The levee will remain active and acceptable in the program, as the levee was designed using Corps guidelines and the project status needs to be updated upon completion of construction.

2.2.5 **Tidal Marsh Restoration**

The tidal marsh restoration elements include the construction, realignment and connection of several pilot channels, tributaries and tidal channels. In addition to the channel work, native plantings and limited placement of salvaged large woody debris will be performed.

The plan for the tidal marsh restoration is to have the restoration elements constructed and ready when the S. Levee Removal (demolition) begins. As the breach and removals are performed, the restoration channels will sequentially be inundated as the levee removal work progresses.

2.2.5.1 **Pilot Channels**

Two pilot channels will be constructed, one in the Smith A area and a second in the Smith B area. The pilot channels are simple channels that are designed to maximize tidal exchange in the marsh areas, and ultimately erode and evolve into sinuous channels over time.

These channels will be excavated using a backhoe and will be approximately 3ft to 6ft wide and 2-3ft deep. The maximum depth of excavation of the pilot channels is a function of the tidal marsh floodplain elevation, and the downstream main tidal channel bed elevation of 3ft. The tidal marsh floodplain areas in Smith A and Smith B is ~6-7ft. The maximum excavation depth is therefore 3-4ft deep. The profile of the channel will start at an elevation matching the bed of the main tidal channel, and then the profile
excavated at even grade daylighting at the end of the alignment. The excavated spoils will be removed from the adjacent floodplain, and disposed of in other levee or Big Ditch fill areas, or made available for reuse.

The tidal channel alignments were revised in the 90% design to have additional sinuosity. The rationale for increasing sinuosity is that the erosion and sedimentation predictions for full tidal channel development are on the order of 10-years. The channels will not likely be able to actively erode the channel margins, due to the strategy to maintain floodplain marsh vegetation to outcompete reed canary grass. The additional channel length and curvature provides increased tidal rearing habitat area, for which it could take a longer period of time to develop naturally. The planned pilot channel sinuosity is 1.35 for Smith A and 1.25 for Smith B pilot channels respectively.

Sinuosity of restored and natural tidal marsh channels has been measured by a number of investigators. The range of published values of tidal channel sinuosity is broad and varies between 1.07 and 1.72 depending upon channel order. Fisher pilot channels are 2\textsuperscript{nd} order channels, for which the Hall study and compilation of salt marsh tidal channel data indicated that sinuosity ranged between 1.15 and 1.44. This range brackets the proposed riverine tidal marsh channels proposed for Fisher Slough. The overall long term function and shape of the channels will ultimately be a function of sediment deposition, vegetation establishment, for which the proposed sinuosity falls within the expected range of channel sinuosity.

In Smith A, the pilot channel would be \(\sim 1,000\text{ft}\) long, and in Smith B the pilot channel along the west edge of the property would be \(\sim 1,600\text{ft}\) long. The amount of material to be excavated for the pilot channels is approximately 1,500\text{cy} of material.

The design plans will use a single alignment table layout and provide a small number of typical sections for the contractor to use as a guide for constructing in the field. Pilot channel grading does not need to be very precise. The channel can be slightly perched relative to the main channel, or have a pool at the confluence. Or, the bed elevation could stay relatively flat over long sections and could end abruptly at the blind end. These will be “rough” features and do not need a high level of precision to function effectively per the restoration objectives. Unacceptable construction would be an uneven, overexcavated profile, that would leave pockets or deep pools where fish could be stranded. The profile should in general drain towards the Slough without major ponding areas.

It is recommended that the field engineer and field biologist work with the construction contractor and crew during excavation of the channels. A basic stakeout alignment should be provided with offsets should be laid out in the field. Then excavation can be performed with technical input during construction.

2.2.5.2 Tributary Realignments

Big Fisher and Little Fisher tributaries will be realigned across the historical fan areas onto the Smith B alluvial fan and tidal marsh areas. The realignments are designed using similar channel geometry to the existing stream channels (Figures 47-48).

The Big Fisher Creek realignment is 720\text{ft} long above the confluence with Little Fisher and 780\text{ft} long below the Little Fisher confluence downstream to the Main Tidal channel connection. The total footprint of the feature is 0.69\text{ acres}. The channel bed profile of Big Fisher Creek will be graded to roughly match the bottom of the main tidal channel realignment.

The Big Fisher cross section of the channel is approximately 15\text{ft} to 20\text{ft} wide and 2-3\text{ft} deep along the alluvial fan area, and then will change with steeper banks and be slightly deeper along the tidal marsh plain area. The realignment will require approximately 2,000\text{CY} of earthwork, and will connect with the
Little Fisher Creek realignment at the base of the historical fan. The excavated materials from the Smith B area can likely be salvaged for use as topsoil exchange. Much of the excavated material from higher elevations (towards the east) will become more gravelly in nature. These should be removed and disposed of, and are not to be used in the levee setback design. The gravelly materials can be used to fill in the existing Big Fisher Creek after diversion and in the realignment berm. Currently there is a historical berm on the west side of Big Fisher Creek that was built to redirect Big Fisher towards the north in its current alignment. This berm will be breached. A similar type of berm will be installed across the existing stream and redirect the stream channel back towards the south.

In addition to the base realignment of Big Fisher Creek, an option is shown on the design plans for removing the entire historical realignment berm, and filling in Big Fisher Creek (see CAD sheets C19 and C22). This activity would restore then entire channel and floodplain areas to a condition found prior to the ditching and realignment of Big Fisher Creek. This is a line item option in the bid sheets.

The Little Fisher Creek realignment is 1,200ft long, and approximately 6ft wide, and 3ft deep. The total footprint, not including the levee removal is 0.17 acres. The realignment will follow what is thought to be the historical channel towards the fan. Little Fisher will connect with Big Fisher in the middle of the Smith B tidal marsh area. Little Fisher bed profile will match the bed channel elevation of the Big Fisher realignment at their confluence.

The cross section geometry in the plans and profile are relatively simplistic. Additional pool-riffle geometry can be field fit during construction to a general plan and profile design per the guidance of the project manager, engineer or biologist. They would show the operator where to excavate slightly deeper to form a pool. Typical pools form at the base of riffle slopes and on the outside of meander bends. For reference, typical spacing of pools is on the order of 6 to 7 channel widths. For Little Fisher Creek this would be 24ft to 28ft, and should also correspond with the outside of meander bends. For Big Fisher Creek the spacing would be approximately 120ft spacing, and also would be located towards the outside of meander bend features.

The creeks will flow across the fan area, it are expected that there will be dynamic and shifting features channel configurations over time. The plan is to provide minimal maintenance to maintain these alignments and to let the channels form naturally over time. The exception to this approach where maintenance will be performed is if levee integrity becomes an issue due to channel erosion and migration. The proposed profiles will be slightly steeper than the existing channel profiles, across the face of the fan, and there will likely be adjustments in the slope and channel dimensions including signs of deposition and erosion along the active channel adjustment areas. These should not be “fixed” unless it is demonstrated that there are clear reasons to do so. The following conditions would be considered for post-project corrections:

- Active erosion encroaching on levee or private structures.
- Blockages of sediment and debris causing adverse effects or impacts on local private property or flood control levees or infrastructure.

Otherwise, the fan area should be considered dynamic where active sedimentation, debris deposition, channel migration and bank erosion are likely to occur.

The construction timing and sequencing of the tributary realignments is such that the channels can be mostly constructed in the dry behind the existing levee. As the South Levee is removed, then the tributaries can be reconnected through simple levee breaches. Once they are reconnected, the previous channels can then be plugged, filled and regarded as necessary.
Figure 47. Big Fisher Creek Near Head of Realignment

Figure 48. Little Fisher Creek Near Levee Removal and Realignment Area
2.2.5.3 Main Tidal Channel Realignment

The realignment of the main tidal channel along the Smith B property will provide for a longer channel and more habitat area and improved tidal exchange and flow connectivity to the low energy Smith B area. The improved tidal connectivity will result in increased fish access and ecological response for the pilot channels in the Smith B marsh area. It also reduces some of the risks associated with damages to the north levee, and may reduce long term O&M needs, although minimally as Big Fisher Creek was the primary culprit for long term sedimentation, erosion and levee related damages.

The main tidal channel section dimensions are based on the width and depth of Hill Ditch and the existing main tidal channel through Fisher Slough (Figure 49). The channel is 35ft wide, with 2H:1V sideslopes, and a depth of 4ft to the top of bank. The realignment is 1,300ft long and approximately 4,000CY of materials will be removed. These materials can be used for both topsoil and for levee construction materials, if they meet the requirements of the soils specifications. If not suitable, these materials can be salvaged for farm topsoils.

The realignment begins immediately upstream from the Big Ditch existing culvert crossing demolition area, and extends upstream to the current Hill Ditch/Big Fisher Creek confluence, along the margin of the wetland inside the levee. At station 13+00, the realigned channel crosses the existing South Levee removal area into the Smith B tidal marsh area. This will be a breach location during the levee removal. Upstream from this location, the tidal channel again crosses the existing South Levee removal at station ~18+50, another breach location, and then reconnects upstream at the Big Fisher Creek/Hill Ditch confluence. The order of the breaching should be downstream first, and then upstream. This requirement will be checked during review of contractor care and diversion of water submittals.

The plan is to place fill material at the head of the existing channel as a plug to deflect the channel into the Smith B area. It is recommended that the excavated materials from the Smith B portion of the main tidal channel be used for sediment plug at the upstream end of the existing channel.

The timing and sequence of construction for the Main Tidal Channel Realignment is similar to the tributaries in that a majority of the work can be performed in the dry. For the areas within Smith B, and outside of the levee, the channel can be constructed wholly in the dry. For the sections of the channel inside the levee, work will be located in wet areas. Construction in these areas will need to be isolated to prevent water quality pollution. The Main Tidal Channel Realignment can be connected as the Existing South Levee is removed.
2.2.5.4 **Large Woody Debris Salvage and Placement**

Large wood debris removal will be necessary in many clearing areas on the project. Several large trees will be removed from the project. A technical memorandum is on file with The Nature Conservancy if future large wood debris placement in Fisher Slough is acceptable to the project partners (App. A.6). At the time of this report, all LWD will be removed and disposed of and hauled to an off-site disposal or restoration location, or chipped and used as mulch on the project site. No LWD will be installed in the project.

2.2.6 **Existing South Levee Removal**

The existing south levee removal is the final element of project construction. This structure must remain in place until all other levee setback and tributary, main tidal and pilot channel dry construction activities are completed. Otherwise significant work areas would become wet and difficult to construct, as well as increasing the potential for water quality pollution. In addition to the channel realignments, there are some other project elements that will occur with the Existing South Levee Removal. The sequencing of these elements is a critical part of the design and described in more detail herein.

2.2.6.1 **Existing Big Ditch culvert crossing demo**

The Existing Big Ditch culvert demolition is included in Phase III instead of Phase II work primarily because the levee spillway needs to be maintained during the winter season between construction phases. If it were removed in Phase II work, the overflow would need to be reconstructed, and then eventually removed. It will be less expensive to perform this work in Phase III.

Removal of the existing Big Ditch culvert will require a water diversion structure, for which the contractor will need to submit a diversion plan.
The structure will be completely demolished to a minimum depth 3ft below the Main Tidal Channel invert to an elevation of 0.0ft NAVD88. Remaining portions of the structure below this elevation can remain in place, as it is unlikely that the site will scour below this elevation due to the downstream controls at the site. When demolishing the structure, the contractor will be required to report if the entire structure is being removed, or if only a portion of the structure. An inspection of the removal will be required. This could be a construction contingency item, as the subsurface structure is not well known.

2.2.6.2 Existing Big Ditch downstream section fill and grade
The downstream section of the Existing Big Ditch (now former in Phase III) will be filled and regarded during Phase III work. There is little to no risk of the spillway structure being overtopped during the construction season. The ditch will be sloped and tied to the levee toe similar to the remainder of the levee toe grading at a 5% slope for 20ft to 40ft out from the levee. For the Big Ditch, the sloping will need to span the entire ditch and match flush with the interior floodplain elevations.

One consideration that will be addressed by the geotechnical engineer is if the ditch should be cleared and scarified prior to filling. The roughening of the banks may allow for better installation of the soils and prevent cracking along the fill and ditch seem. A preliminary recommendation for clearing, scarifying and 90% compaction has been made, but is under review by the geotechnical engineer.

2.2.6.3 Existing S. Levee removal
The levee removal will occur along a 4,500ft alignment that extends from the Pioneer Highway Bridge to the Moyer property driveway along Little Fisher Creek on the southeast most corner of the project site. The levee removal will be 5.78 acres in length. 45,000CY of material is planned for removal.

Levee removal materials can be reused on the site for construction of a number of features. Suitable materials can be used to construct upper and remaining portions of the S. Levee Setback Realignment. Unsuitable materials can be disposed of in non-essential fill features (such as ditches), or mixed with suitable materials to meet levee soil construction specifications.

Figure C-29 (App. B) in the design plans shows the cut/fill approach for the S. Levee removal, assuming that the floodgates will be closed during construction.

The levee removal materials from station 35+00 to 39+00 (and possibly further) will need to be disposed of off of the site (or remixed to meet levee fill suitability requirements) due to the significant amount of gravel materials found in the existing levee. The remainder of the S. Levee removal will likely be usable for the Levee Setback Realignment, if the materials meet the geotechnical specifications. If not, they may be disposed of in other grading/fill project areas. Excess materials shall be disposed of at an off-site upland disposal location.

The removal of the levee will require the contractor to develop a work schedule based on tidal conditions. Excavations can occur above the tide at any time during the fish work window. One strategy would be to remove the upper portions of the levee first, and then come back and remove the lower elevations of the levee during a succession of low tide events.

During low tide excavations, the structure will be demolished to match the existing tidal marsh floodplain grade along the entire length of the levee. Soils being removed from the levee are suitable for other levee construction, however the levee setback will be completed at this time. Therefore the soils will need to be hauled and disposed of off site. TNC will provide a list of approved persons who can haul the material
away upon request. The contractor will document all hauling activities off the site, both for the contractor and other local persons approved by TNC.

One of the key design considerations for utilizing Existing S. Levee materials is the water control issue. In order to make soil reuse feasible, the site needs to be isolated from tidal inundations, such that exposed excavated soils are kept relatively dry for continual equipment access and removal. If the levee were removed and tidal flows allowed through the floodgates, then the tidal marsh and levee setback construction areas would be exposed to daily tidal inundation and water surface fluctuations. This is considered unfeasible from a construction perspective and cannot be allowed.

It is recommended to close the floodgates for the entire Phase III construction period (July 15 – Sept. 30, 2011 coinciding with the in-water work window) and pump tributary flows downstream from the floodgates or into Big Ditch. This would allow for full excavation and removal of the existing levee, without compromising construction work associated with tidal flows on exposed construction surfaces. The contractor shall coordinate closure of the floodgates as part of the water control plan for the Phase III levee removal portions of the project.

As the levee is excavated, it is expected that the exposed sections of the levee breach will have minor turbidity plumes as these bare surface are exposed to tidal flows.

### 2.2.6.4 Tributary, Pilot Channel and Main Tidal Channel Connections

Channel reconnections will most likely occur as the contractor removes the levee. In certain instances advanced connections may be made, with temporary stream crossings or access from a variety of haul routes may be used. Channel reconnections should be included on a general sequencing schedule and notification from the contractor in advance of the reconnections should be provided. One of the key logistical issues will be connecting the channels while maintaining access to other areas of the project. Again, temporary crossings may be involved for this work.

During channel reconnections, there may be brief plumes of turbidity as the newly constructed channels become wet. In order to manage these releases, it is likely best that the channels be connected in a progressive sequence and not all at the same time.

### 2.2.6.5 Access Haul Route and Staging Area Removal

The haul routes and staging areas will need to be removed in a logical sequence. All staging areas will be removed from tidal marsh restoration areas in advance of plantings and channel connections. Removal of rock work areas after tidal inundation is not desirable. Certain haul routes can be removed in advance of the South Levee Removal, while those directly adjacent will likely be removed as the structure demolition progresses. This will allow trucks to have access for removal and disposal of the levee soils. Again haul route and staging area removal activities will need to be on a clearly defined construction schedule.

### 2.2.7 Vegetation & Planting Plan

The vegetation and planting plan has been developed with a few key strategies. First, preservation of existing vegetation is a primary objective at the site. Second, establishment of vegetation using native seed sources from the Skagit River and tributaries to Fisher Slough was originally assumed to be adequate to establish native marsh plant communities. However, recent monitoring by The Nature Conservancy and the Skagit River System Cooperative has shown minimal marsh or native seed sources available in the Skagit River, which will therefore not likely meet restoration objectives using the original native seeding approach. In lieu of this finding a more rigorous seeding and planting plan will be undertaken.
The primary areas that will be planted or seeded are as follows:

- **Hydroseed exposed surfaces on the new levee setback structure and disturbed riparian/upland areas after completion of grading on both phases to discourage colonization by reed canary grass.**
- **Plow and seed disturbed areas along the toe of the levee and within the marsh areas to discourage colonization by reed canary grass.**
- **Plant native woody riparian and upland species at higher and appropriate areas around the site, especially on exposed and graded land surfaces. Suggest somewhat sparse planting of container stock for trees, spread alder seed or allow natural deposition of alder seed.**
- **In ungraded areas where there are dense stands of reed canary grass that occupy greater than 500 square feet, conduct mowing and localized application of herbicide.**
- **Preserve existing vegetation and limit access and grading areas to the fullest extent possible, which needs to be clearly communicated to the contractor and controlled to a certain level, otherwise the contractor may drive all over the tidal marsh restoration areas. Notes should be added to the site plans clearly indicating this approach.**

Planting activities should occur after completion of grading activities and 1-month prior to breaching of the levee and opening of the floodgate (in late September and early October). Limited watering will be required to establish the riparian and marsh seeding areas prior to tidal marsh inundation to increase plant survival, otherwise the seed and topsoils could be eroded and washed away. Watering will include application of 1 inch of water on all seeded areas, on a weekly basis, unless there is an equivalent amount of precipitation during that time period. Planting of container stock and cuttings should not occur until after October 1 and when rain is predicted within approximately 1 week.

The vegetation and planting plan is designed to allow natural establishment of native seed sources in the tidal marsh areas, while limiting the potential for invasive species infestations. The areas that will be planted are delineated as follows:

- **Levee hydroseed areas**
- **Tidal marsh seeding of disturbed areas**
- **Riparian seeding and planting areas**

**Figure 50** is a summary of estimated quantities and species for the vegetation and landscaping plan and shows weight distribution of seed species and planting distribution of riparian species. The weight distributions were determined using seed weight worksheets provided by NRCS 2009 (*App. A.7 – Landscape Plans, Tables 16-19*).

The levee setback side slopes will be seeded with a fescue and ryegrass mix that has been applied on other northwest levee setback projects. One issue that has been observed in plant mixes on levees is that natives species do not work well on levee/dike areas. The compacted soils need good grass cover that can be mowed as part of operations and maintenance activities. An effective mix includes tall fescue with annual ryegrass for quick cover (Kilcoyne and Chaney 2009). The levees will have 6in of clean topsoil placed on the fill material surfaces. Then the seed mix will be applied using hydroseed with a soil binding agent and tackifier along all exposed areas of the levee from the toe to the top of levee embankment. The access and roadway areas along the top of the levee will not receive hydroseed treatments, as they will be covered with crushed road subsurface and surface materials. The seed mix is specified as follows:
Table 16. Levee Hydroseed Mix

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name</th>
<th>Unit Type</th>
<th>Units</th>
<th>Units/Acre</th>
<th>Acres</th>
<th>Total Units (lbs) or (ea)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Agrostis spp.</td>
<td>Bentgrass</td>
<td>Seed</td>
<td>lbs</td>
<td>3</td>
<td>6.98</td>
<td>20.9</td>
</tr>
<tr>
<td>Festuca rubra</td>
<td>Fescue, Creeping Red</td>
<td>Seed</td>
<td>lbs</td>
<td>15</td>
<td>6.98</td>
<td>104.7</td>
</tr>
<tr>
<td>Festuca trachyphylla</td>
<td>Fescue, Hard</td>
<td>Seed</td>
<td>lbs</td>
<td>10</td>
<td>6.98</td>
<td>69.8</td>
</tr>
<tr>
<td>Festuca arundinacea</td>
<td>Fescue, Tall</td>
<td>Seed</td>
<td>lbs</td>
<td>15</td>
<td>6.98</td>
<td>104.7</td>
</tr>
<tr>
<td>Lolium multiflorum</td>
<td>Ryegrass, Annual or Italian</td>
<td>Seed</td>
<td>lbs</td>
<td>20</td>
<td>6.98</td>
<td>139.6</td>
</tr>
</tbody>
</table>

The second vegetation and planting area are seeding of the disturbed areas of the tidal marsh. The likely exposed areas will include narrow strips along the toe of the levee, margins of the pilot channel and tributary realignments, and ditch fill areas. Typically in tidal marsh restoration projects, these areas are not seeded or planted as there is enough native seed supply for natural plant establishment to occur. However, for the purposes of this project, the strategy for controlling invasive infestations is to maintain as much existing vegetation as possible on the marsh plain areas, and then seed and plant those disturbed areas to promote rapid growth of native species to outcompete invasive Reed Canary Grass (RCG). Marsh seeding areas will use the following species:

Table 17. Marsh Seed Mix

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name</th>
<th>Unit Type</th>
<th>Units</th>
<th>Units/Acre</th>
<th>Acres</th>
<th>Total Units (lbs) or (ea)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carex lyngbyei</td>
<td>Lyngbei's Sedge</td>
<td>Seed</td>
<td>lbs</td>
<td>18</td>
<td>13.00</td>
<td>227.5</td>
</tr>
<tr>
<td>Aster subspicatus</td>
<td>Douglas aster</td>
<td>Seed</td>
<td>lbs</td>
<td>5</td>
<td>13.00</td>
<td>65.0</td>
</tr>
<tr>
<td>Deschampsia cespitosa</td>
<td>Tufted hairgrass</td>
<td>Seed</td>
<td>lbs</td>
<td>1</td>
<td>13.00</td>
<td>8.1</td>
</tr>
<tr>
<td>Triglochin Maritimum L.</td>
<td>Seaside Arrowgrass</td>
<td>Seed</td>
<td>lbs</td>
<td>1</td>
<td>13.00</td>
<td>8.1</td>
</tr>
<tr>
<td>Hordeum brachyantherum</td>
<td>Meadow barley</td>
<td>Seed</td>
<td>lbs</td>
<td>1</td>
<td>13.00</td>
<td>8.1</td>
</tr>
<tr>
<td>Potentilla pacifica</td>
<td>Pacific silverweed</td>
<td>Seed</td>
<td>lbs</td>
<td>1</td>
<td>13.00</td>
<td>8.1</td>
</tr>
</tbody>
</table>

Many of the seeds are commercially available. However, Carex lyngbyei is a special product and will require lead time to obtain the necessary amount of seed for the project. The contractor will likely need to coordinate procurement of seed 1-year in advance of the hydroseed application (during Phase II construction).

The third vegetation and planting areas are the riparian areas adjacent to the tributary realignments, and higher elevations of the levee removal near the upstream end of Fisher Slough. The recommendation is to first hydroseed grass and clover species and then plant woody species to outcompete invasive RCG. The riparian zones will be planted generally on the Moyer parcel where there are more gravelly soils or at a minimum as 100 foot wide buffers following along the tributary realignments of Big Fisher and Little 125 December 2009
Fisher creeks. Hydrosedding should occur after levee removal and 1-month prior to breaching to allow time for establishment of seed root systems. Plantings should be postponed as long as possible into October to improve plant mortality rates. The following seed mixes are specified for the project (Tables 18 and 19).

### Table 18. Riparian Seed Mix

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name</th>
<th>Unit Type</th>
<th>Units</th>
<th>Units/Acre</th>
<th>Acres</th>
<th>Total Units (lbs) or (ea)</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Festuca rubra</em></td>
<td>Fescue, Creeping</td>
<td>Seed</td>
<td>lbs</td>
<td>8</td>
<td>3.12</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>Red</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Festuca trachyphylla</em></td>
<td>Fescue, Hard</td>
<td>Seed</td>
<td>lbs</td>
<td>5</td>
<td>3.12</td>
<td>15.6</td>
</tr>
<tr>
<td><em>Festuca arundinacea</em></td>
<td>Fescue, Tall</td>
<td>Seed</td>
<td>lbs</td>
<td>8</td>
<td>3.12</td>
<td>25.0</td>
</tr>
<tr>
<td><em>Lolium multiflorum</em></td>
<td>Ryegrass, Annual</td>
<td>Seed</td>
<td>lbs</td>
<td>10</td>
<td>3.12</td>
<td>31.2</td>
</tr>
<tr>
<td></td>
<td>or Italian</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Trifolium pratense</em></td>
<td>Clover, Red</td>
<td>Seed</td>
<td>lbs</td>
<td>5</td>
<td>3.12</td>
<td>15.6</td>
</tr>
</tbody>
</table>

Woody riparian plantings include the following species:

### Table 19. Woody Riparian Plantings

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name</th>
<th>Unit Type</th>
<th>Units</th>
<th>Units/Acre</th>
<th>Acres</th>
<th>Total Units (lbs) or (ea)</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Cornus stolonifera</em></td>
<td>Red-osier dogwood</td>
<td>1-gal</td>
<td>ea</td>
<td>150</td>
<td>7.19</td>
<td>1078.5</td>
</tr>
<tr>
<td><em>Picea sitchensis</em></td>
<td>Sitka spruce</td>
<td>1-gal</td>
<td>ea</td>
<td>75</td>
<td>7.19</td>
<td>539.3</td>
</tr>
<tr>
<td><em>Thuja plicata</em></td>
<td>western red cedar</td>
<td>1-gal</td>
<td>ea</td>
<td>75</td>
<td>7.19</td>
<td>539.3</td>
</tr>
<tr>
<td><em>Lonicera involucrata</em></td>
<td>Twinberry</td>
<td>1-gal</td>
<td>ea</td>
<td>200</td>
<td>7.19</td>
<td>1438.0</td>
</tr>
<tr>
<td><em>Malus fusca</em></td>
<td>Pacific crabapple</td>
<td>1-gal</td>
<td>ea</td>
<td>100</td>
<td>7.19</td>
<td>719.0</td>
</tr>
<tr>
<td><em>Rosa nutkana</em></td>
<td>Nootka rose</td>
<td>1-gal</td>
<td>ea</td>
<td>200</td>
<td>7.19</td>
<td>1438.0</td>
</tr>
<tr>
<td><em>Salix hookeriana</em></td>
<td>Hooker willow</td>
<td>cutting</td>
<td>ea</td>
<td>100</td>
<td>7.19</td>
<td>719.0</td>
</tr>
<tr>
<td><em>Salix lasiandra</em></td>
<td>Pacific willow</td>
<td>cutting</td>
<td>ea</td>
<td>100</td>
<td>7.19</td>
<td>719.0</td>
</tr>
<tr>
<td><em>Populus Trichocarpa</em></td>
<td>Black Cottonwood</td>
<td>cutting</td>
<td>ea</td>
<td>100</td>
<td>7.19</td>
<td>719.0</td>
</tr>
</tbody>
</table>
Figure 50. Fisher Slough Landscaping Plan
3.0 QUANTITIES & COST ESTIMATE

The quantity takeoffs are listed in the cost estimate and bid sheets, by each individual build item. The engineer’s cost estimate has been submitted under separate cover to The Nature Conservancy to preserve bid integrity.
4.0 REVISED PROJECT SCHEDULE

A project schedule was developed that includes project design, permitting and estimated construction durations. There are multiple drivers behind the schedule including design, partner coordination, permit requirements, in-water work windows and funding. The typical in-water work window for Fisher Slough is Aug. 1 through Sept. 30th each year. The proposed in-water work window is July 15 through Oct. 15, for both 2010 and 2011. These extended in-water work windows will be necessary to accomplish major construction tasks. These dates need to be confirmed with WDFW as part of the HPA permitting process. Table 20 is the revised project and construction schedule.
<table>
<thead>
<tr>
<th>Item</th>
<th>Duration</th>
<th>Completion Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Final Design Start</td>
<td></td>
<td>01-Mar-09</td>
</tr>
<tr>
<td>Permit Submittal</td>
<td>2 months</td>
<td>01-Jul-09</td>
</tr>
<tr>
<td>Project Final Design Completion</td>
<td>5 months</td>
<td>01-Dec-09</td>
</tr>
<tr>
<td>Phase I – Floodgate Retrofit Completion</td>
<td>6 months</td>
<td>01-Oct-09</td>
</tr>
<tr>
<td>Permits Awarded</td>
<td>6 months</td>
<td>01-Apr-10</td>
</tr>
<tr>
<td>Construction Contracting Completion</td>
<td>2 months</td>
<td>01-Jun-10</td>
</tr>
<tr>
<td>Phase II Construction Start</td>
<td></td>
<td>15-Jun-10</td>
</tr>
<tr>
<td>Site Preparation, TESC &amp; Haul Routes</td>
<td>2 weeks</td>
<td>30-Jun-10</td>
</tr>
<tr>
<td>Big Ditch Channel Construction</td>
<td>3 months</td>
<td>30-Sep-10</td>
</tr>
<tr>
<td>Levee Setback Pre-load</td>
<td>3 months</td>
<td>30-Sep-10</td>
</tr>
<tr>
<td>Big Ditch Inverted Siphon Water Diversion Construction 1</td>
<td>1 month</td>
<td>15-Jul-10</td>
</tr>
<tr>
<td>Big Ditch Inverted Pipe Installation 1</td>
<td>2 months</td>
<td>15-Sep-10</td>
</tr>
<tr>
<td>Inlet/Outlet Construction</td>
<td>2 months</td>
<td>15-Sep-10</td>
</tr>
<tr>
<td>Pilot Channel, Tributary Pre-Excavations</td>
<td>1 month</td>
<td>15-Sep-10</td>
</tr>
<tr>
<td>Big Ditch Channel and Inverted Siphon Rewatering</td>
<td>1 week</td>
<td>15-Sep-10</td>
</tr>
<tr>
<td>Existing Big Ditch Fill &amp; Regrade U/S</td>
<td>2 weeks</td>
<td>01-Oct-10</td>
</tr>
<tr>
<td>Phase III Construction Start</td>
<td></td>
<td>01-Jun-11</td>
</tr>
<tr>
<td>Final Load and Emergency Spillway S. Levee Realignment</td>
<td>2 months</td>
<td>01-Aug-11</td>
</tr>
<tr>
<td>Big Ditch Crossing Demo and D/S Fill</td>
<td>2 weeks</td>
<td>01-Sep-11</td>
</tr>
<tr>
<td>Marsh and Riparian Restoration Seeding</td>
<td>2 weeks</td>
<td>15-Sep-11</td>
</tr>
<tr>
<td>South Levee Removal (Existing)</td>
<td>1 month</td>
<td>15-Sep-11</td>
</tr>
<tr>
<td>Tributary, Pilot, Main Tidal Channel Connections</td>
<td>1 week</td>
<td>15-Sep-11</td>
</tr>
<tr>
<td>Riparian Plantings</td>
<td>2 weeks</td>
<td>15-Oct-11</td>
</tr>
<tr>
<td>Phase III Site Cleanup &amp; Closeout &amp; Floodgate Opening</td>
<td>2 weeks</td>
<td>15-Oct-11</td>
</tr>
</tbody>
</table>

1 Inverted pipe water diversion construction is contingent upon contractor proposed construction methods (trench vs. trenchless)
5.0 OPERATIONS, MAINTENANCE & MONITORING PLANS

The DRAFT Operations, Maintenance and Monitoring Plans are submitted as accompanying documents to this Basis of Design Report Appendix F.
6.0 PROJECT OUTREACH, COORDINATION AND CONSULTATION

Since the inception of the Fisher Slough Tidal Marsh Restoration project, The Nature Conservancy has utilized a collaborative planning, design and implementation approach for developing the project. This has included active participation in all phases of work by their project partners, review of work by a Technical Advisory Committee, coordination with adjacent property owners, and coordination with multiple regulatory agencies. Multiple meetings, presentations, project reviews were performed throughout the project. Documentation of these meetings, presentation, and the comment review, project outreach, coordination and consultation process are included in a series of notes and spreadsheets in Appendix G.
7.0 PERMITS OVERVIEW

A number of permits and supporting documentation have been submitted for this project. Permits and supporting documentation that have been submitted for this project include the following:

7.1 SECTION 404 DREDGE AND FILL PERMIT (CORPS)

Upon review of the permit application, the Corps will make the determination as to whether this will be covered by a NWP or an individual permit. As a wetlands restoration project, this project will likely be covered under a Nationwide Permit (NWP). The Corps has requested Section 7 Endangered Species Act consultation with NMFS-USFWS, and complete documentation has been provided to them in this regard. The Corps has also requested confirmation of consultation with the WA SHPO and interested tribes under Section 106 of the National Historic Preservation Act, a process that has also been completed. At the time of this report, a determination has not yet been made as to whether this project will be covered under a Nationwide Permit or an Individual Permit.

Application Submitted: 6/15/2009
Review Period: 6-12 months
POC: Randel Perry, USACE

7.2 RIVERS AND HARBORS ACT, SECTION 10 PERMIT (CORPS)

Because this project is occurring within navigable waters, it is subject to Section 10 of the Rivers and Harbors Act. Permits under this act are issued by the Corps. There is no maintenance exemption for Section 10. This permit is generally applied for as part of the Section 404 permit application.

Application Submitted: 6/15/2009
Review period: 6-24 months
POC: Randel Perry, USACE

7.3 SECTION 7 CONSULTATION (NOAA, USFWS)

An informal consultation with USFWS to determine possible effects to listed terrestrial, avian, or non-anadromous fish species has been completed. The main focus of the USFWS consultation was the bull trout, and USFWS determined that there would be no effect to this species as long as in-water work was performed during the appropriate work window. The project falls within the SRFB programmatic consultation agreement (SRFB, 2008), which covers anadromous species. A SRFB streamline permit process requires the submittal of a self-certification document that is submitted with the JARPA and also sent to the SRFB regional grant manager. If work will occur outside of that window, the informal consultation or BO will be revisited.

Review Period: 60-180 days
POC (NMFS): Tom Sibley
POC (USFWS): Ginger Phelan
Concurrence Letter Received: 10/21/2009

7.4 SECTION 401 WATER QUALITY CERTIFICATION (WDOE)

This permit will be issued by WA Dept. of Ecology. It is required if the conditions below are met, or if the project cannot be covered by a NWP under which projects are pre-certified under Section 401:
The project or activities are below the OHWM with new work being proposed outside the original footprint, or
The proposed project or activity increases the original footprint of the structure by more than 10 percent in wetlands.

Application Submitted: 6/15/2009
Review Period: 30-180 days
POC: Rebecca Piaget (425) 649-7129

7.5 SKAGIT COUNTY SHORELINE PERMIT

The Shoreline Permit for Substantial Development, Conditional Use or Variance permit is necessary for the project. Prior to submitting the permit, a lot certification, critical areas review, and pre-application meeting were completed. The permit conditions were reviewed with Skagit County staff and an initial determination has been made that a conditional use or variance are not required for the project. The permit application addresses how the project falls within the shoreline master plan, and the counties zoning and comprehensive plan. Of particular interest for this permit is restoration within areas zoned as agricultural natural resource areas. The shoreline application also requires re-submittal of the JARPA at the local level. Due to the extenuating circumstances regarding the initial FHEP Streamline JARPA approach, and outside developments that have affected the permit process, Skagit County (a project partner) is working to expedite the Shoreline Permit process.

Application Submitted: 11/30/2009
Review Period: 4 mos
POC: Betsy Stevenson

7.6 SKAGIT COUNTY SPECIAL USE PERMIT

The special use permit requires review of zoning ordinances for uses that are not typically allowed in certain zoning designations. The approach for developing the special use application will be to review Skagit County zoning ordinances and address application questions regarding zoning compliance, and effects on neighboring properties and effects on natural and rural resource areas. In Skagit County, the Board of County Commissioners has issued an ordinance for review of restoration projects in agricultural, natural resource zoned areas. Of particular interest for this project are the demonstration of the habitat restoration projects effects on adjoining properties specifically for flooding and drainage. The process for vetting these issues and making a determination is the hearing board examiner process.

Application Submitted: 11/30/2009
Review Period: 4 mos
POC: Betsy Stevenson

7.7 SKAGIT COUNTY GRADING PERMIT

A fill/grade permit is required by Skagit County. A full set of plans has been provided to Skagit County for review of project grading plans. In addition to the grading permit, a temporary erosion and sediment control plan (TESC) is a supplement to the application submittal.

Review Period: 4 mos
POC: Shane Whitney
7.8 SKAGIT COUNTY RIGHT OF WAY AND UTILITY PERMIT
Access from the Pioneer Highway for construction related activities will require a right of way and utility permit. This permit will be obtained by the construction contractor and is a requirement of the construction contract documents.

Review Period: 1 mos  
POC: Ronny Audette

7.9 WDNR FOREST PRACTICES PERMIT
Due to the project’s proximity to riparian management zones and existing forestry resources, a forest practices permit is required for the project. The permit assesses the removal and harvest of forest resources as a result of project construction. The development of the application requires mapping of existing forested areas, and then identifying harvest areas, which then need to be flagged in the field for forester review.

Review Period: 3 mos  
POC: Boyd Norton

7.10 HYDRAULIC PROJECT APPROVAL (WDFW)
This permit, issued by WDFW, is responsible for preserving, protecting, and perpetuating all fish and shellfish resources of the state. To assist in achieving that goal, the state Legislature in 1949 passed a state law now known as the "Hydraulic Code" (Chapter 77.55 RCW). Provisions of this law require that any person, organization, or government agency wishing to conduct any construction activity that will use, divert, obstruct, or change the bed or flow of state waters must do so under the terms of a HPA issued by WDFW. State waters include all marine waters and fresh waters of the state, except those watercourses that are entirely artificial, such as irrigation ditches, canals and storm water run-off devices. Applications for streamlined processing of fish habitat enhancement projects must include the application form for these projects that is attached to the JARPA.

Application Submitted: 6/15/2009  
Review period: 45 days  
POC (WDFW): Brian Williams

7.11 GENERAL CONSTRUCTION STORMWATER PERMIT (WDOE)
Due to the overall size of the project, larger than 1-acre and discharging from the site, the construction contractor will be required to submit for a general construction stormwater permit. This permit was reviewed with Ecology and given the information on the size and type of the project, it is highly likely to fall into the general construction category. As part of this permit, the construction contractor is also required to develop a stormwater pollution prevention plan (SWPPP), which shall be kept on file at the site at all times during the project. Construction contractors are familiar with these permits and associated documents. Obtaining the permit is included in the construction contract documents.

Review period: 60 days  
POC (WDOE): Charles Gilman

7.12 OTHER PERMIT RELATED DOCUMENTS
A biological assessment for Bull Trout is required for Section 7 consultation. The USFWS has completed this document and it has been filed with the appropriate permit review agencies.

A wetland delineation and jurisdictional determination has been performed for the project and was submitted as part of the JARPA.

A level 1 HTRW study has been completed and is addressed in the SEPA checklist and narrative.

An archaeological/cultural resources study was performed to support the SEPA initial study. The DAHP has issued a letter of concurrence regarding the project. Tribal review by the Skagit, Swinomish and Stilliguamish tribes has been completed. The Stilliguamish has provided comments regarding periods when they recommend providing a monitor, in addition to the project construction archaeologist.
8.0 CONSTRUCTION CONTRACTING

The construction contract will be managed by The Nature Conservancy with support from their project partners and engineering consultant. The project involves a variety of different types of earthwork, structural, geotechnical and fish habitat restoration construction activities. The construction contract specifications will include provisions for several items that will provide Quality Assurance and Quality Control review of constructed works.

8.1 CONSTRUCTION CONTRACTOR QUALIFICATIONS & SELECTION

Selection of a construction contractor can be done through a variety of methods. Low bid is a typical method for selecting contractors. However, the lowest bidder is not always the most experienced. Also, other contractors use low bid strategies to win the work, and then modify the contract during construction.

It is recommended that the construction contractor be selected on qualifications, expertise, project implementation plan approach, and cost. The combination of these elements would be a best value approach, rather than low bid.

The Request for Proposals (RFP) would be tailored to request this information from the contractor.

8.2 BONDING

Requirements for bid bond, payment and performance bond, and an insurance bond will be required for the project.

The bid bonds will provide assurance to TNC that the contractors who bid on the project have the financial stability to complete the work. A bid bond can also help manage the selection and replacement of a contractor if they default or stop work on the project. Requirement of a bid bond ensures that the contractors provide clean bids.

A payment and performance bond for the full contract bid value is required to assure that the contractor performs the work according to schedule and the contract specifications. In addition, it protects TNC from financial risks if the contractor should become financially insolvent and negligent on their fiduciary duties to vendors and subcontractors who they are responsible to pay for products and services. The payment and performance bonds will protect TNC from third party claims arising from contractor negligence. As part of the payment and performance bond, a 1-year warranty will be requested on the project. The payment and performance bond and 1-year warranty will cover from Phase II – 2010, Phase III – 2011, and 1-year warranty 2012 (3 years).

A maintenance bond of 2 years and $500,000 value is requested for the project. The maintenance bond is being requested to insure against potential damages from project startup and operation. The maintenance bond will extend from 2013 through 2015 (2 years).

8.3 CONSTRUCTION MANAGEMENT PLAN AND SCHEDULE

Considering the complexity of construction sequencing, the contractor will be required to submit a construction management plan and schedule prior to starting the work. The plan will be approved and revised as necessary throughout the project.
8.4 SUBMITTALS

The construction contractor will be required provide a number of submittals documenting the construction process and completed (as-built) conditions. Examples of submittals will include sampling and lab testing of soil conditions during levee setback construction. The submittals will be defined in the specifications. The following is an outline of planned construction contractor submittals.

- Construction schedule
  - Phase II Work
  - Phase III Work
- Health and safety plan
- Water control plan and stormwater pollution control plan (SWPPP) and final temporary erosion and sediment control plan (TESC)
  - Diversion and return locations
  - Construction sequencing
  - Schedule of operations
  - Diversion and pollution control contingencies
  - Fish screening and exclusion details
- Final contractor inverted siphon construction plan
- Final Big Ditch culvert crossing demolition plan
- Manufacturer specifications and shop drawings
  - Final trash rack design plan
  - Cat walk
  - 54in vertical slide gates
  - Anchor and connection details
  - Manufactured bridge crossings
  - Access gates shop drawings
  - 54”, schedule 4710, HDPE pipe specifications
  - Geotextile fabrics
  - Soil and aggregate import testing specifications
    - Levee fill import from off-site
    - Spillway aggregates
    - Levee road subsurface and surface aggregates
    - Clay blanket and cutoff materials
  - Disposal and landfill locations
  - Landscaping and planting seed germination specification sheets
- Contractor required permits
  - Ecology general construction stormwater permit and SWPPP
  - BNSF railroad temporary occupancy permit (start this process immediately)
  - Skagit County right of way and utility permit
- Project management reports
  - Weekly progress reports and preliminary pay estimates
  - Monthly progress reports and payment invoices
  - Special inspection requests
- Contract closeout
  - As-built plans
  - Construction acceptance documentation
  - Closeout checklist approval
8.5 NOTIFICATIONS

The construction contractor will be required to provide notifications to TNC prior to completion of major works for inspection, and startup of next phases of work. This will ensure that TNC and their construction technical support resources will have time to plan for inspections, review and approvals. A notification request form has been included, as well as a list of inspection and coordination points where the contractor shall notify TNC of a scheduled inspection, check-point or test.

- Pit tests every 100ft along S. Levee Setback along abandoned railroad embankment STA 18+50 to 31+50
- Geotechnical inspections and testing
  - Levee inspection of all stripped and cleared foundation areas of S. levee setback prior to embankment fill installation
  - Levee testing of every 1,000cy of placed fill
  - Inverted siphon pipe backfill and levee reconstruction
  - Inverted siphon filter diaphragm materials
- Concrete testing
  - Inlet, outlet, sedimentation basis of inverted siphon structure
  - Bridge footings and abutments
  - Rebar inspections for inverted siphon inlet and outlet structure
- Inverted siphon inspections
  - HDPE pipe installation testing/inspection
  - Rebar inspections
  - Concrete testing and inspections
  - Trash rack and catwalk installation inspections
- Cultural resources archaeological observer and tribal monitor
  - Farm ditch regrade (required archaeological observer, notify tribal monitor)
  - S. Levee setback connection with Moyer Hill in Smith B (notify tribal monitor)
  - Existing south levee removal (notify tribal monitor)

8.6 QUALITY ASSURANCE INSPECTIONS, TESTING, REVIEW AND APPROVAL

Inspections and testing will occur throughout the project. A number of scheduled tests or inspections will be programmed into the contract specifications. The construction contractor is responsible under the terms of their contractor to perform their own testing. TNC will be performing quality assurance testing of numerous items, as listed in the previous section.

Plans should also be made to have on-site inspectors during the entirety of construction to document progress and identify any potential problems during construction. This can be done by providing daily field reports and review of construction plan documents. The inspectors should be under contract directly to TNC separate from the construction contract to assure independent review of the constructed project works. Quality control documents and approvals will also be provided to Skagit County, per their requirements of the permitting process.

8.7 CONSTRUCTION AS-BUILTS

Construction as-builts will be required for the contract. Actual surveyed plans, and redline markups of field changes will be fully documented and submitted to TNC as part of the contract. The as-builts will be referenced for measurement and payment of completed works. Stamped plans will be provided by the
contractor. As-builts documentation will also be provided to the TNC project partners for records of changes to the flood control and drainage works.

8.8 MEASUREMENT AND PAYMENT
Measurement and payment will require monthly payment invoices with quantity takeoff estimates from the contractor. TNC and their owner’s representative will review and provide independent calculations of cost items and accept or decline the completed works payment request. Additional measures can be taken to provide contract payment assurances. Retention is a method that can be used in construction contracting for withholding funds until project completion.

8.9 PROJECT FINAL INSPECTION AND CLOSEOUT
A final project inspection and closeout checklist will be included with the construction contract. The intent of the project closeout is to document and finalize all as-built conditions, clean up loose ends on the site, and make arrangements for final payment and closing out the contract.
9.0 REAL ESTATE, EASEMENTS AND RIGHT OF WAY

All real estate, easements, rights of way and land owner agreements have been coordinated by The Nature Conservancy and are on file. The Nature Conservancy has landowner agreements for all properties within the project footprint and adjacent to the project footprint.

It is noted that a number of property line boundary adjustments were made for the project as a result of title, deed and on the ground survey of actual property lines. The property lines as shown on the plans are approximate and it is the responsibility of the contractor to identify all necessary property boundaries in the field during construction. A copy of all real estate documentation used for the project is located on file with The Nature Conservancy as part of this report.
10.0 REFERENCES


Kilcoyne, K and Chaney, M. 2009. Personal communication with Jenny Baker (TNC) regarding levee seed mix.

Kokusho, T. 2006. “Recent developments in liquefaction research learned from earthquake damage”, Journal of Disaster Research, V1, N2


Occupational Safety & Health Administration, 1990. Excavation Standard (29 CFR 1926, Subpart P)


U.S. Army Corps of Engineers (USACE), 2000. EM-1110-2-1913 Engineering Design and Construction of Levees

U.S. Army Corps of Engineers (USACE), 2005. Skagit Flood Study, DRAFT Hydraulic Technical Documentation


U.S. Army Corps of Engineers (USACE), 2008. ETL 1110-2-571 Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams and Appurtenant Structures


U.S.D.A. Natural Resources Conservation Service (NRCS), 2009. Seeding Rate Calculator Worksheets Provided by NRCS.

Washington Department of Fish and Wildlife (WDFW), 2005. Skagit Drainage and Fish Initiative, Drainage Maintenance Agreement by and between the WDFW and Skagit County Drainage and Irrigation Improvement District #17.

APPENDIX A – TECHNICAL DESIGN MEMORANDA
APPENDIX A.1 – GEOTECHNICAL INVESTIGATIONS
APPENDIX A.2 – PRELIMINARY TESC PLAN
APPENDIX A.3 – INVERTED SIPHON DESIGN MEMORANDA
APPENDIX A.4 – TEMPORARY FISHER SLOUGH CROSSING DESIGN MEMORANDUM
APPENDIX A.5 – LEVEE DESIGN MEMORANDA
APPENDIX A.6 – LARGE WOOD DEBRIS DESIGN MEMORANDUM
APPENDIX A.7 – PLANTING PLAN INFORMATION
APPENDIX A.8 – SURVEY, REAL ESTATE, TITLE REPORT INFORMATION

SUBMITTED AS SEPARATE SET OF COPIES ON FILE WITH THE NATURE CONSERVANCY
APPENDIX A.9 – WATER SUPPLY WELL REPORT
APPENDIX B – FINAL DESIGN PLANS

SUBMITTED AS SEPARATE SET OF 11” x 17” SET OF PLANS (NOT ATTACHED)
APPENDIX C – CONSTRUCTION CONTRACT SPECIFICATIONS

UNDER DEVELOPMENT (SEPARATE SUBMITAL)
APPENDIX C.1 – FISHER SLOUGH CONTRACT SPECIFICATIONS
APPENDIX C.2 – SCHEDULES & FORMS
APPENDIX C.3 – MANUFACTURER SPECIFICATIONS & VENDOR INFORMATION
APPENDIX D – ENGINEER COST ESTIMATE & EXAMPLE BID TAB

ENGINEER’S COST ESTIMATE IS FOR THE NATURE CONSERVANCY ONLY

DO NOT DISTRIBUTE
APPENDIX E – BID SHEETS

BID SHEETS ARE EXCEL SPREADSHEETS. ELECTRONIC FILES SHOULD BE DISTRIBUTED TO CONTRACTORS AS PART OF RFP
APPENDIX F – OPERATIONS & MAINTENANCE PLANS

O&M MANUALS ARE UNDER DEVELOPMENT
APPENDIX G – DESIGN REVIEW COMMENTS & PROJECT MTG. NOTES