Bay View Watershed Stormwater Management Plan - Phase 2

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Prepared for Skagit County Public Works
and Skagit County Drainage Utility

Consultants

Semrau Engineering & Surveying, PLLC
TetraTech/KCM, Inc.

Technical Contributors

Dike & Drainage District No. 12
Drainage District No. 14
Drainage District No. 19
Port of Skagit
Skagit Conservation District
Padilla Bay National Estuarine Research Reserve

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Chapter 1

Introduction and Executive Summary

The Bay View area is located in the western part of Skagit County. The area derives it name from the community of Bay View, which is located along the shoreline of Padilla Bay. The area is characterized as primarily rural in nature, but portions are planned to have urbanized type development. This urbanized development has resulted in a need to address existing and future stormwater drainage needs to protect property and the environment.

The Bay View watershed stormwater planning efforts have been a series of planning projects by the Skagit County Drainage Utility (herein call the Drainage Utility) as well as others with a stake in stormwater drainage in the area. Planning efforts by other stakeholders are summarized in **Chapter 2 - Planning Data.** The stormwater planning efforts by the Drainage Utility have expanded upon these previous planning efforts. A summary of the planning efforts performed by the Drainage Utility are summarized below:

- **Aerial Survey.** The Drainage Utility contracted with Walker & Associates to perform an aerial survey of the Bay View watershed, which was conducted in April 2002. This information was compiled with another aerial survey performed from August 1998.

- **Watershed Study.** This was the initial stormwater drainage study performed by the Drainage Utility. The study extended from November 2002 through June 2004. The primary focus of this study effort was a ground survey and inventory of drainage structures within the primary four drainage sloughs within the Bay View watershed. This information was used to develop four hydraulic models that represent the unique hydraulic characteristics of each slough. These hydraulic models were used to identify drainage facilities that had insufficient capacity for existing and/or future development. Other tasks included the acquisition and review of existing planning information, the development of planning data that would be used within the watershed, and the development of a preliminary list of possible drainage improvements.

- **Stormwater Management Plan.** The *Bay View Watershed Stormwater Management Plan* expanded upon the Watershed Study. The draft of this document was completed in September 2005, which included a Capital Facility Plan for drainage improvements. Some stakeholders had concerns that all of the proposed drainage improvements were within the existing drainage sloughs and not located on future developed properties. There was a desire among some stakeholder to examine drainage options that prevented stormwater from the Bay View ridge from entering the existing drainage sloughs.

- **Bypass Channel Assessment.** An additional study was conducted that examined the design, permitting and land acquisitions issues that would be involved in the construction of new drainage channels that would intercept stormwater drainage from the Bay View ridge and prevent this upland drainage from entering the existing drainage sloughs. The results of the assessment were presented in technical memorandums in December 2005.

The assessment indicated that there were considerable design and permitting obstacles. In addition, land acquisition costs from numerous property owners were high, adding to the conclusion that the bypass channel concept was not feasible at this time, but that elements of the concepts could be considered for existing and future drainage planning efforts.

- **Phase 1: Bayview Ridge Urban Growth Area.** Skagit County was experiencing pressure to complete the *Bayview Ridge Subarea Plan*, which was to establish urbanized development policies for a 3,633 acres area of the Bayview Ridge Urban Growth Area [UGA]. The approval of the *Bayview Ridge Subarea Plan* in December 2006 included developing drainage improvements to accommodate the expected urban development. To address this immediate need, the Drainage Utility divided the *Stormwater Management Plan* into two phases. Phase 1 would address drainage improvements that are needed to address expected development within the UGA. This document was completed in February 2007 and underwent a public hearing process through the Skagit County Planning Commission and adoption by the County Commissioners.

  The Bayview Ridge UGA was challenged before the Western Washington Growth Hearings Board under Case No. 07-2-0002. The Board issued a Compliance Order in August 2007. In its order, the Board stated that the *Bay View Watershed Stormwater Management Plan Phase 1: Bayview Ridge Urban Growth Area* and the County Commissioner Resolution that adopted the Plan fulfill the requirements of RCW 36.70A.070(3)(b) and (c) for storm drainage facilities for the Bayview Ridge UGA.

- **Phase 2.** The *Bay View Watershed Stormwater Management Plan Phase 2* planning effort presented in this document is intended to expand upon the recommendations presented in the draft *Bay View Ridge Stormwater Management Plan* (September 2005 draft) to address development and drainage issues outside of the influences within the Bayview Ridge UGA. It is the intent that the Phase 1 and Phase 2 documents provide a complete stormwater drainage plan for the Bay View watershed.

The purpose of the *Bay View Watershed Stormwater Management Plan Phase 2* is to evaluate the stormwater impacts due to development on the Bay View ridge outside of the Bayview Ridge UGA. This evaluation involves: 1) evaluating stormwater facility improvements; and 2) proposing stormwater management strategies to manage drainage within the Bay View watershed using the planning data and hydraulic models previously developed for this project. Skagit County funded the preparation of this Phase 2 Plan from its Drainage Utility fund.

The Bay View Watershed Study Area (herein referred to as the Study Area) is bounded to the west by Padilla Bay, to the north and northeast by Joe Leary Slough and its tributaries, and to the south and southeast by Big Indian Slough. The Study Area is approximately 11,277 acres.

For the purposes of this Plan, the Study Area was divided into three basins; the No Name Slough Basin, the Joe Leary Slough Basin, and the Indian Slough Basin. The Indian Slough Basin was further divided into two separate basins, Little Indian Slough Basin and Big Indian Slough Basin, to perform separate hydraulic analyses. Stormwater drainage facilities within these three basins use a combination of drainage ditches and sloughs, culverts and storm drain pipelines, and ponds and detention facilities.

Past development in the Study Area has been considered to be rural in nature. More concentrated residential development has occurred in the community of Bay View on the west side of the Study area and around the Skagit Golf and County Club on the east side of the Study Area. Industrial and commercial developments, which are all within the proposed Urban Growth Area, have occurred around the Skagit Regional Airport and along Farm-to-Market Road just north of State Route 20.

There are several stakeholders within and surrounding the Study Area that will be directly or indirectly impacted by recommendations presented in this Plan. These stakeholders include Skagit County, the dike and drainage districts, Port of Skagit, City of Burlington, and property owners within the Study Area. Other federal and state agencies will have input into recommendations through regulatory requirements.

There are several existing reports and documents that provide information relative to stormwater drainage planning and facility design in the Bay View watershed. These documents include:

- *Padilla Bay/Bay View Watershed Non-Point Action Plan (1995),*
- *Port of Skagit Stormwater Management Master Plan (1998),*
- *Hydrologic and Hydraulic Model of the No Name Slough Drainage (2000),*
- *Bayview Ridge Subarea Plan (2006),*
- *Joe Leary Slough Drainage Study (2002),*
- *Bay View Watershed Stormwater Management Plan Phase 1: The Bayview Ridge Urban Growth Area (2007),*
- *Inventory and Evaluation of Tide Gates and Pump Stations related to Alternatives #5 and #7 of the Skagit River Flood Damage Reduction Feasibility Study (2002).*

This last document was prepared in conjunction with the Skagit River Flood Protection/Salmon Restoration Project.

An inventory of stormwater drainage facilities within the Study Area was conducted. The inventory was not comprehensive but focused mostly on the four major drainage sloughs within the Study Area. These four major drainage sloughs are No Name Slough, Joe Leary Slough, Little Indian Slough, and Big Indian Slough. The drainage facility inventory is presented in **Appendix A** of the Phase 1 Plan.

The Surface Water Management Model (SWMM), developed by the U.S. Environmental Protection Agency, incorporated the drainage facility inventory information and was used to assess hydrologic and hydraulic characteristics of the four major drainage sloughs within the Study Area. However, only additional rural development outside of the Bayview Ridge UGA was considered in this Phase 2 Plan. Therefore, this additional rural development only impacts the No Name Slough and the Joe Leary Slough basins. The Big Indian Slough and Little Indian Slough basins primarily drain areas within the Bayview Ridge UGA.

The model results indicated that there are areas of potential flooding along each of the No Name Slough and Joe Leary Slough as a result of additional rural development outside of the Bayview

Ridge UGA. Conceptual stormwater drainage improvements were developed and evaluated that could improve capacity in the limiting facilities. Potential drainage facility improvements that were evaluated included the following:

- Enlarging existing slough channels,
- Regional detention,
- Stormwater pump stations,
- Bypass channels,
- Increasing levee heights, and
- Upsizing culverts or replacing culverts with bridges.

The Capital Improvement Plan of the proposed drainage facilities improvements for the Phase 2 Plan is presented in **Chapter 7** for each drainage basin. A summary of the proposed improvements and their associated project cost estimates are presented in **Table 1-1**.

Table 1-1: Summary of Proposed Capital Improvements in Each Drainage Basin

Drainage Basin	Proposed Stormwater Capital Improvement	Project Cost Estimate
No Name Slough Basin	Marihugh Road Regional Detention Pond	$ 1,450,000
	Marihugh Road Bypass Pipeline	$ 1,675,000
Joe Leary Slough Basin	Joe Leary Slough Channel Widening	$ 223,000
	Outfall Pump Station	$ 6,700,000
	South Spur Pump Station	$ 1,900,000
Total Capital Improvement Cost Estimate		$ 11,948,000

In addition to capital improvements, stormwater management strategies were also recommended to help ensure that the existing and proposed facilities would be adequately maintained to provide maximum efficiency during a storm event.

Although stormwater runoff is the primary focus of this Plan, stormwater quality and treatment strategies are briefly discussed. Big Indian Slough, Joe Leary Slough, and No Name Slough are listed as impaired waters on the Washington State Department of Ecology's 303(d) list. The primary contamination sources include pavement runoff, septic tanks, and agricultural activities. Stormwater treatment techniques have been developed and tested primarily for urban settings and their application and effectiveness in rural settings is not fully known. Typical treatment techniques for rural stormwater runoff include wet ponds, bio-retention swales, constructed wetlands, and other low impact development guidelines.

Chapter 2

Planning Basis

This Chapter presents the planning basis for the *Bay View Watershed Stormwater Management Plan*. This planning basis was developed during the preparation of the Phase 1 Plan. The information presented is a summary of the planning basis. A more detailed discussion of the planning basis is presented in the Phase 1 Plan.

A. Stakeholders

There are several entities that have a stake in stormwater drainage planning in the Bay View watershed. These entities are listed below. The stormwater planning objectives of each stakeholder is also discussed in detail in Chapter 2 of the Phase 1 Plan.

- Skagit County
- Drainage and Irrigation District No. 14
- Drainage District No. 19
- Dike, Drainage and Irrigation District No. 12
- Port of Skagit
- City of Burlington
- Large Tract Land Owners
- Washington State Department of Ecology
- Washington State Department of Fish and Wildlife
- Washington State Department of Transportation
- Federal Aviation Administration

B. Related Planning Documents

There are several existing reports and documents that provide information relative to the stormwater planning and facility design in the Bay View watershed. The following are abstracts and summaries from these related documents.

- **Padilla Bay/Bay View Watershed Non-Point Action Plan,** prepared by the Padilla Bay/Bay View Watershed Management Committee and Skagit County Department of Planning and Community Development (1995).
- **Stormwater Management Master Plan for the Bay View Business and Industrial Park and Skagit Regional Airport,** prepared by David Evans and Associates, Inc., for the Port of Skagit (October 1998).

- **Padilla Bay Hydrology – Hydrologic and Hydraulic Model of the No Name Slough Drainage**, prepared by Northwest Hydraulic Consultants for the Padilla Bay National Estuarine Research Reserve (November 2000).
- **Bayview Ridge Subarea Plan,** prepared by Reid-Middleton for Skagit County Planning and Development Services (December 2006).
- **Joe Leary Slough, Maiben Road Ditch and South Spur Ditch Drainage Analysis and Findings**, letter report prepared by Semrau Engineering & Surveying for Drainage and Irrigation District No. 14 (January 29, 2002).
- **Inventory and Evaluation of Tide Gates and Pump Stations related to Alternatives #5 and #7 of the Skagit River Flood Damage Reduction Feasibility Study,** prepared by Skagit County Public Works Surface Water Management (November 2002 Draft).
- **No Name Slough Watershed Characterization,** prepared by the Skagit Conservation District and the Padilla Bay National Estuarine Research Reserve (May 2004).

C. Study Area

The Bay View watershed is located in the westerly portion of Skagit County, west of the City of Burlington. This area has four drainage sloughs that convey stormwater runoff from the Bay View watershed to Padilla Bay. A Vicinity Map of the Bay View Watershed Study Area (herein referred to as the Study Area) is shown on **Figure 2-1.** The Vicinity Map shows the outline of the Study Area that was used for this Phase 2 Plan.

The Study Area is primarily bounded on the west by Padilla Bay, on the south and southeast by Big Indian Slough, and on the north and northeast by Joe Leary Slough and its tributaries. The Study Area is approximately 11,277 acres. **Figure 2-2** is an aerial photograph of the Bay View ridge and surrounding farmland.

D. Land Use and Development

Existing development varies within the Study Area. **Figure 2-3** provides an indication where development has occurred. Prominent developments in the Bay View ridge area include the rural village Bay View, Bay View State Park, Padilla Bay's Breazeale Interpretive Center, Skagit Regional Airport, numerous industrial and commercial developments, and residential plat and cluster developments.

Chapter 3 of the Phase 1 Plan provides a detailed discussion of the historical development within and around the Bay View ridge area and how that development has addressed or contributed to drainage problems.

The Skagit County Comprehensive Plan describes the general development patterns that are proposed within all areas of the county. Future land use within the UGA will be governed by the development patterns outlined in the *Bayview Ridge Subarea Plan*, which builds on the existing land use pattern including residential, commercial, business/industrial, and park/open space related uses. A map showing the land use designations in the Study Area is presented in **Figure 2-4. Table 2-1** summarizes the land use designations within the Study Area.

Table 2-1: Land Use Designation Summary within the Study Area			
Land Use Designation	**Total Area**	**Percentage**	**Average Densities**
Agriculture	2,556 Acres	22.7 %	1 dwelling unit per 40 acres
Commercial / Industrial	0 Acres	0 %	N/A
Public / Open Space	99 Acres	0.9 %	N/A
Rural Intermediate	888 Acres	7.9 %	1 dwelling unit per 2.5 acres
Rural Reserve	4,440 Acres	39.4 %	1 dwelling unit per 5 acres
Rural Resource	257 Acres	2.3 %	1 dwelling unit per 10 acres
Rural Village	171 Acres	1.5 %	1 dwelling unit per 1 acres
Proposed Bayview UGA	2,829 Acres	25.1 %	N/A
Water Bodies	37 Acres	0.3 %	N/A
Totals	**11,277 Acres**	**100 %**	
Source: Skagit County Mapping Services. Acreage figures are derived based on best information and technology available. Accuracy may vary depending on the source of the information, changes in political boundaries or hydrological features, or the methodology used to map and calculate a particular land use.			

E. Environment

The environment within and around the Bay View ridge area is described in detail in Chapter 3 of the Phase 1 Plan. The following is a summary of the Bay View ridge environment.

1. Natural Features

Prominent natural features include Padilla Bay, No Name, Joe Leary and Big Indian Sloughs, Bay View ridge area, and the surrounding alluvial farmland. Padilla Bay is an estuary at the saltwater edge of the large delta of the Skagit River and it is the receiving water for all of the stormwater drainage from the Bay View ridge area.

2. Soil and Topography

The soils in the Bay View ridge area have limited suitability for building site development and septic tank drain fields. The hydrologic group is a D classification for the soils due to the generally low permeability and the presence of a perched water table between November and May. The elevation of the Bay View ridge extends to approximately 220 feet above the surrounding floodplain. The terrain is generally characterized as gently sloping with isolated areas with sleep slopes.

3. Climate

The average rainfall in the Bay View watershed is approximately 30 inches per year. This estimate was determined after review of rainfall data records from gauging stations located in Anacortes and Mount Vernon. Typically there is slightly more rainfall in Mount Vernon than in Anacortes.

Most of the annual rainfall occurs during the fall and winter months. On average, between 65 and 70 percent of the annual rainfall occurs between October and March.

The average high temperature typically occurs in August at approximately 73ºF (23ºC). The average lower temperature typically occurs in January at approximately 34ºF (1ºC).

4. Aquifer Recharging Areas

The Bay View watershed does not contain any identified critical aquifer recharged areas. Some areas in the north portion of the Bay View watershed are currently not served by a public water system and, therefore, homeowners rely on groundwater wells for their water supply. There are other properties throughout the Bay View watershed that may also rely on groundwater wells for their source of water. Development within the Bay View ridge areas may reduce groundwater infiltration of stormwater.

5. Flood Hazard Areas

The Bay View ridge outside of the surrounding floodplain, is not prone to flooding, however, some soil designations within the Study Area are prone to perched water tables. In the past, undersized or poorly designed stormwater conveyance facilities have resulted in localized flooding during severe storm events. These flooding incidences are typically short-lived and many times result in corrections to the stormwater conveyance facilities.

6. Wetlands

Understanding the relationship of wetlands is critical in developing the stormwater management plan for this area. There are numerous wetlands scattered throughout the Bay View ridge. The *Bayview Ridge Subarea Plan* provides a detailed discussion regarding wetlands on the Bay View ridge. A map showing the wetlands on the Bay View ridge is presented in **Figure 2-5.**

Wetlands are considered critical areas that are legally protected under the Federal Clean Water Act, the State Growth Management Act, and Skagit County codes and regulations. Wetlands are defined by the presence of water during the growing season, hydric soils, and the presence of a plant community that is able to tolerate prolonged soil saturation. These areas provide important environmental functions, including habitat for wildlife, aquifer recharge, water for fish and other aquatic species and wildlife, a visual buffer in the built landscape, and reducing the impact or frequency of flooding.

Within most of the Bay View ridge, wetlands have been identified based on the National Wetlands Inventory and interpretations of aerial photography. Approximately 349 acres of wetlands and buffers have been identified in the Bay View ridge area outside of the Port of Skagit ownership. The precise boundaries of these wetlands are not known and would be delineated by project proponents as specific development projects are proposed.

The Port of Skagit has identified and delineated 694 acres of wetlands, buffers, and open space within their 1830-acre ownership as part of the Wetlands and Industry Negotiation [WIN] Management Plan. Of the 694 acres, 250 acres have been delineated as high functioning wetlands along with 200 acres identified as buffers.

The WIN Program is a planning process that began in 1994 to identify and protect high functioning wetlands, along with identifying and improving low functioning wetlands. This process was completed in 2001 for the Port of Skagit property.

7. Priority Habitat

The Priority Habitats and Species (PHS) Program, administered by the Washington State Department of Fish and Wildlife, provides comprehensive information on important fish, wildlife, and habitat resources in Washington State. PHS is the principal means by which this information is transferred from their resource experts to those who can protect habitat.

Figure 2-6 shows the priority habitat within the Bay View ridge area that has been established by the Washington State Department of Fish and Wildlife.

Chapter 3

Stormwater Quality Analysis

The purpose of the *Bay View Watershed Stormwater Management Plan* is to quantify stormwater runoff within the Study Area in order to analyze drainage and flood control options. Typically, flood control generally relies on controlling large and infrequent stormwater runoff, while stormwater quality management is aimed at smaller storm events. As such, stormwater quantity control is directly addressed in this Phase 2 Plan.

Stormwater quality in the Study Area is regulated under Skagit County's Drainage Ordinance. The Drainage Ordinance incorporates the requirements of the 1992 *Stormwater Management Manual for the Puget Sound*[1] (Stormwater Manual) as Skagit County has not yet adopted the Department of Ecology's 2005 update to the Stormwater Manual.

The *Padilla Bay/Bay View Watershed Nonpoint Action Plan*[2] (Nonpoint Action Plan) is the most significant work to date regarding stormwater pollution in the Bay View watershed. The Skagit County Department of Planning and Community Development, with the assistance of the Padilla Bay/Bay View Watershed Management Committee, prepared the Nonpoint Action Plan to provide a program of actions to reduce or prevent nonpoint source pollution and protect beneficial water uses. The Nonpoint Action Plan contains extensive background information on watershed characteristics, outlines goals and objectives for reducing nonpoint pollution, identifies and sometimes quantifies sources of nonpoint pollution, and outlines an implementation strategy. The Nonpoint Action Plan was reviewed and approved by the Washington State Department of Ecology on May 30, 1995. This plan is currently undergoing an implementation status review by the Skagit Conservation Education Alliance (SCEA), a non-profit foundation administered by the Skagit Conservation District to protect natural resources.

Big Indian Slough, Joe Leary Slough, and No Name Slough are listed as impaired waters on the Washington State Department of Ecology's 303(d) list. Big Indian Slough and Joe Leary Slough are listed for dissolved oxygen, fecal coliform, and temperature. No Name Slough is listed for dissolved oxygen and fecal coliform. Some water quality data for No Name Slough is on file with both the Breazeale-Padilla Bay Interpretive Center and the Skagit Conservation District.

Waters placed on the 303(d) list can trigger the preparation of Total Maximum Daily Load [TMDLs] for those water bodies, a key tool in the work to clean up polluted waters. TMDLs identify the maximum amount of a pollutant allowed to be released into a water body so as not to impair users of the water, and allocate that amount among various sources. Prior to completion of a TMDL, the inclusion of a water body on the 303(d) list can reduce the amount of pollutants allowed to be released under National Pollution Discharge Elimination System (NPDES) permits

[1] *Stormwater Management Manual for the Puget Sound*, prepared by the Washington State Department of Ecology (February 1992).

[2] *Padilla Bay/Bay View Watershed Nonpoint Action Plan*, Prepared by the Skagit County Department of Planning and Community Development (May 30, 1995).

issued by Ecology. Ecology began implementing the Phase 2 NPDES General Permit for Municipal Storm Sewers in January 2007. This permit will increase the rules and regulations local governments must follow concerning the water quality of the stormwater in their drainage systems. The stormwater systems (existing and projected) within the Bay View ridge area will be subject to these augmented regulations.

There are several potential sources of contamination for stormwater runoff that are discussed in detail in Chapter 6 of the Phase 1 Plan. Below is a list of some of the obvious and abundant sources of stormwater contamination within the Study Area.

- Pavement runoff and roadside ditches.
- Septic tanks.
- Agricultural activities.
- Future development.

Chapter 6 of the Phase 1 Plan provides a general discussion of stormwater management strategies for each potential contamination source based on recommendations presented in the Nonpoint Action Plan, the recommended best management practices [BMPs] presented in the 2005 Stormwater Manual[3], and the low impact development guidelines[4]. Specific stormwater treatment techniques that are discussed include stormwater ponds, bioswales, and wetlands.

[3] *Stormwater Management Manual for Western Washington*, prepared by the Washington State Department of Ecology (August 2001).

[4] *Low Impact Development – Technical Guidance Manual for Puget Sound*, prepared by the Puget Sound Action Team and the Washington State University Pierce County Extension (January 2005).

Chapter 4

Storm Drainage Facilities

For the purposes of this Stormwater Management Plan, the Study Area was divided into three basins; the No Name Slough Basin, the Joe Leary Slough Basin, and the Indian Slough Basin. The No Name Slough Basin covers the west portion of the Study Area. The Joe Leary Slough Basin covers the north and northeast portion of the Study Area. The Indian Slough Basin covers the south and southeast portion of the Study Area. The characteristics of each of these basins, with emphasis on its storm drainage facilities, are discussed below.

A. No Name Slough Basin

The No Name Slough Basin covers the west portion of the Study Area. It is also referred to as Basin A in the hydraulic modeling. Several smaller subbasins located north of the No Name Slough Basin drain directly to Padilla Bay through numerous culverts that cross the Bay View Edison Road.

The basin is characterized by rural type development with the exception of the community of Bay View, which has a couple of commercial industries and a concentration of residential houses.

The pump station facilities at the outlet of No Name Slough have two vertical turbine pumps. Both pumps operate at 1200 rpm. The larger pump, manufactured by Prime Pump Corporation, has a 50-hp motor and has an estimated discharge flow rate of 9,000 gpm (20 cfs). This pump discharges through a 24-inch fiberglass pipe with a flap gate on the end. The smaller pump has a 25-hp motor and has an estimated discharge flow rate of 6,750 gpm (15 cfs) based on the pump nameplate information. This smaller pump discharges through an 18-inch fiberglass pipe with a flap gate on the end.

The pump station only operates during peak storm events that coincide with high tides. The pump station is controlled by floats, which stage the starting of the two pumps. The smaller pump typically starts first. The Drainage District personnel occasionally adjust the floats. The report entitled *Padilla Bay Hydrology – Hydrologic and Hydraulic Model of the No Name Slough Drainage* provides some estimates for pump control elevations. According to Drainage District personnel, it takes approximately 36 to 40 hours to drain No Name Slough with the pump station after a typical storm event.

The stormwater drainage facilities inventory is presented in **Appendix A.** The following is a summary of the drainage facilities and management responsibilities within the No Name Slough Basin.

Drainage District: Dike, Drainage and Irrigation District No. 12.

Primary Drainage Facility: No Name Slough.

Gravity Outfall Structures: Four outfall structures total; one 5’x3’ box culvert with tide gate, one 48” HDPE with tide gate, and two 36” culverts with a common tide gate.

Pump Station: There are stormwater pump station facilities with two pumps at the outlet of No Name Slough (previously described).

Ponds and Detention Facilities: There are a few ponds and/or stormwater detention facilities within the No Name Slough Basin. One primary detention facility is located on the Paccar property. A new detention facility has been recently constructed on Port property northeast of the intersection of Ovenell Road and Farm-to-Market Road.

Ditches: Roadside ditches have been extensively used within this basin to convey stormwater drainage to Padilla Bay outfalls.

Culverts and Pipes. In addition to the roadside ditches, there are some roadside culverts and storm drainage piping systems within the basin and a few catch basin structures. There are also a few bridge structures within the basin that cross storm drainage ditches and sloughs.

No Name Slough Outfall and Pump Stations

1. Proposed Phase 1 Improvements for No Name Slough

Dike, Drainage and Irrigation District No. 12 has been continuously making improvements to No Name Slough. Before any project is carried forward, the hydraulic model should be updated to account for any projects that have been completed at that time or approved for development and for changes in existing or expected land use. In addition, it is recommended that additional modeling be performed to better define design criteria for these conceptual projects. The following alternatives were recommended in the Phase 1 Plan for the No Name Slough Basin:

a. Channel Widening

Widen the existing slough from nodes NN-70 to NN-180, a length of approximately 4,000 LF. The new bottom width would be approximately 6 feet wide. This bottom width would more closely match the existing channel width downstream of node NN-70.

b. Culvert Replacement

Replacement of two undersized culverts was recommended; culverts NN-C-3 and NN-C-5. Dike, Drainage and Irrigation District No. 12 replaced culvert NN-C-3 with a bridge in 2007. Culvert NN-C-5 should be replaced with a 4-ft culvert pipe. Local topographic survey information is needed as part of the final design to verify that the specified culvert shape and material are appropriate for that location.

c. Bypass Channel

The bypass channel has already been constructed by Dike, Drainage and Irrigation District No. 12 in 2006. The length of the bypass channel is approximately 3,000 LF. An additional 4-ft culvert may need to be installed under the Bay View-Edison Road to optimize the efficiency of the bypass channel. Since this project is completed, the hydraulic models should be updated with the most recent drainage configuration.

d. Increased Pumping Capacity

Pumping remains the best option for reducing the flooding in the slough's lowland areas near the outfall. Two pumps with a combined capacity of 36 cfs already exist at the outfall. An additional pump with a capacity of 54 cfs is recommended to reduce the flooding potential at the outfall.

B. Joe Leary Slough Basin

The Joe Leary Slough Basin is the largest of the three basins within the Study Area. It is also referred to as Basin B in the hydraulic modeling. Storm drainage from this basin discharges directly to Joe Leary Slough and its Maiben Ditch and South Spur tributaries. Most of the Joe Leary Slough drainage area lies outside of the Study Area.

Compared to the other two basins, the Joe Leary Slough Basin is the least developed and is expected to remain rural in nature for the near future. Development within this basin typically consists of small-scale agriculture and livestock operations with some large-tract residential development. A portion of the Bayview UGA will contribute drainage to the South Spur Ditch. Existing development includes the Bay View Elementary School and manufacturing facilities along Josh Wilson Road. Future development is expected to be urban density residential housing.

The stormwater drainage facilities inventory is presented in **Appendix A.** The following is a summary of the drainage facilities and management responsibilities within the Joe Leary Slough Basin.

Drainage District: Drainage and Irrigation District No. 14.

Primary Drainage Facility: Joe Leary Slough along with the Maiben Road and South Spur tributaries.

Gravity Outfall Structures: Twelve 48" culvert pipes with tide gates.

Pump Station: There are no stormwater pump stations within the Joe Leary Slough Basin.

Ponds and Detention Facilities: There are very few ponds and/or stormwater detention facilities within the Joe Leary Slough Basin. This has contributed to uncontrolled runoff from the Bay View ridge area to Joe Leary Slough and its tributaries.

Ditches: Roadside ditches have been extensively used within this basin to convey storm water drainage to Joe Leary Slough and the Maiben Ditch and South Spur tributaries.

Culverts and Pipes: Culverts and storm drainage pipes have been used primarily for roadway and driveway crossings of drainage ditches. There are four bridge structures that also span Joe Leary Slough.

Joe Leary Slough Outfall

1. Proposed Phase 1 Improvements for Joe Leary Slough

As with the other drainage basins in the Bay View watershed, flooding in Joe Leary Slough is largely driven by the tidal cycle. Since ground elevations of adjacent agricultural fields are often in the range of 5 to 10 feet, stormwater drainage alternatives that can reduce flooding are limited. The following drainage improvements were recommended in the Phase 1 Plan for the Joe Leary Slough Basin:

a. Peth Property Slough Bypass

A slough bypass along the toe of the ridge would provide a more direct route to the outlet of the slough. The slough bypass channel would be located primarily on low lying properties owned by J. Peth, W. Paulus and others. The bypass would circumvent the culvert and channel restrictions along D'Arcy Road where the channel is confined by the road. This bypass channel would lower water surface elevations in the lower section of the slough up to Farm-to-Market Road.

b. Joe Leary Slough Widening

The existing slough from Allen West Road to the confluence of Maiben Ditch and South Spur Ditch has channel restrictions. Widening this section of the slough would provide increased conveyance that is equivalent to the existing capacity of the 15-ft wide arch culvert at Allen West Road. The length of this section of Joe Leary Slough is approximately 9,000 LF.

c. Bridge Replacement

In order to widen Joe Leary Slough, four existing wood bridges would need to be replaced with new wood bridges. These existing bridges provide access to property on the west side of the slough.

d. South Spur Ditch Widening

The existing South Spur Ditch from the confluence with Joe Leary Slough to Josh Wilson Road has channel restrictions. Widening this section of the slough would provide increased conveyance. The length of this section of South Spur Ditch is approximately 9,000 LF.

C. Indian Slough Basin

The Indian Slough Basin is the most developed of the three drainage basins. It is also referred to as Basin C in the hydraulic modeling. The Indian Slough Basin is divided into the Little Indian Slough Basin and the Big Indian Slough Basin. This drainage basin also encompasses most of the designated Urban Growth Area. Because of its trend toward urbanization, many stormwater treatment and conveyance systems already exist within this drainage basin.

Historically, the Big Indian Slough Basin was considerably smaller. Higgins Slough, located south of Big Indian Slough, drained most of the south Bay View ridge area. At some point (the specific date is not known) a manmade channel was constructed between State Route 20 and the BNSF railroad track from near the outlet of Big Indian Slough to the intersection with Higgins Slough near the west end of State Route 536 (Memorial Highway). The manmade channel is approximately 6,700 LF long. The new drainage route was considerably shorter since Big Indian Slough discharged directly to Padilla Bay. The outfall structure for Big Indian Slough was constructed around 1922 according District records.

Higgins Slough discharges into the Swinomish Channel. Under the current configuration, normal stormwater drainage discharge through the Big Indian Slough Channel and only large peak storm events overflow into Higgins Slough. For the sake of this Study, we are considering the diverted portion of Higgins Slough to be called Big Indian Slough.

In the early 1980s, the Port of Skagit began developing the Bay View Business and Industrial Park. This development included the construction of stormwater drainage and conveyance improvements. In 1988, the Port of Skagit hired LeGro and Associates to develop a more comprehensive drainage plan for the Bay View Business and Industrial Park. An attempt was made to use two ponds at the corner of Watertank Road and Higgins Airport Way as stormwater detention facilities. However, these two ponds did not function well as detention facilities considering the size of the Bay View Business and Industrial Park and the amount of impervious surfaces.

In 1995, the Port of Skagit committed to reducing erosion impacts and detaining its stormwater on-site prior to release into the Big Indian Slough conveyance system. In 1998 the Port of Skagit hired David Evans and Associates to develop a Stormwater Management Master Plan and to design drainage improvements for the developed properties. The most noticeable stormwater drainage facility that resulted from this effort are the several detention cells along Higgins Airport Way north of Ovenell Road.

The pump station at the outlet of Big Indian Slough has two vertical turbine pumps. The larger pump has a 50-hp motor and has an estimated discharge flow rate of 15,000 gpm (33.4 cfs). The smaller pump has a 30-hp motor and has an estimated flow rate of 10,000 gpm (22.3 cfs). Each pump discharges through a 24-inch corrugated metal pipe with a flap gate on the end.

The pump station only operates during peak storm events that coincide with high tides. A series of floats control the pump station but there is no information available regarding the pump control parameters or operating conditions.

The stormwater drainage facilities inventory is presented in **Appendix A.** The following is summary of the drainage facilities and management responsibilities within the Indian Slough Basin.

Drainage District: Drainage District No. 19.

Primary Drainage Facilities: Little Indian Slough and Big Indian Slough, with potential overflows to Higgins Slough from Big Indian Slough.

Gravity Outfall Structures: Big Indian Slough has seven 48” culvert pipes with aluminum tide gates in a concrete dam. Little Indian Slough has two 48” culvert pipes with tide gates under Bay View-Edison Road.

Pump Station: There is one stormwater pump station with two pumps at the outlet of Big Indian Slough.

Ponds and Detention Facilities: The primary capital improvement project recommended by David Evans and Associates in its 1998 Report was to reconstruct existing detention facilities, conveyance system, and outlet to Big Indian Slough, and to construct seven detention cells along Higgins Airport Way. This project also created fish spawning habitat below the outfall of the detention cells. This project was completed in 1999. Other smaller capital improvement projects that improve stormwater conveyance and reduce erosion have also been recently completed.

Ditches: Like the other two basins, the Indian Slough Basin has numerous roadside ditches for the conveyance of stormwater.

Culverts and Pipes: There are several storm drainage piping systems within this basin, primarily in the east portion within the newer residential developments. Some of the more recent improvements at the Port of Skagit also have utilized more drainage piping systems to improve storm water conveyance. In the older developments, roadside ditches and culverts are still extensively used. There are also several bridge structures that cross Big and Little Indian Sloughs.

Little Indian Slough Outfall

1. Proposed Phase 1 Improvement for Little Indian Slough

West of Farm-to-Market Road, flooding in Little Indian Slough appears to be limited to the 25-year recurrence interval. Flooding at this recurrence interval may be acceptable in the fields located in the lower portion of the slough. The slough has enough storage at the downstream end, and its outlet structure appears adequate to handle peak flows through the 10-year event.

Upstream of Farm-to-Market Road, flooding can be more frequent as a result of the undersized channels and culverts. Modeled results with upgrades to the channel and culvert capacity in the upper slough did not consider the effects of any existing upstream detention. Therefore the result may be conservative.

Before new projects are implemented, the analysis presented in this document should be updated to account for any improvements in the slough system or changes in projected land use. If possible, additional modeling should be completed at a higher resolution at the specific project locations, using the most recent topographic data available.

The following drainage improvement was recommended in the Phase 1 Plan for the Little Indian Slough Basin:

a. Culvert Replacement and Channel Widening

Culvert replacement and channel widening appears to be the most cost-effective alternative in reducing flooding upstream of Farm-to-Market Road. According to the hydraulic model, downstream impacts from removing the culvert restrictions are likely to be insignificant. Subbasin C-2 would contribute to this channel section.

Detention is not recommended at this time. Detention could eliminate flooding upstream of Farm-to-Market Road. However, the storage volume required is relatively large, and construction and maintenance costs would be significantly higher than the costs of replacing the restrictive culverts and increasing the channel capacity of the slough.

Given the low ground elevations at the outlet of the slough, a pump station would likely be the best alternative for reducing flooding in lower portions of the slough. This option was not examined due to the high costs that would be expected if the pump station were to be operated to reduce flooding at the 25-year event. Flooding in the agricultural fields at this frequency level may be acceptable. The downstream land owner has granted a Drainage and Flood Water Easement to Skagit County for the subject property.

Big Indian Slough Outfall and Pump Station

2. Proposed Phase 1 Improvements for Big Indian Slough

Big Indian Slough has the greatest impact from development within the Bayview UGA. Considerable development, including residential, commercial, and light industrial development, has already occurred, with more development planned within the Bayview UGA. The following drainage improvements were recommended in the Phase 1 Plan for the Big Indian Slough Basin:

a. Outfall Detention Pond

The outfall detention pond will provide additional storage near the outfall that can be discharged quickly during a receding tide. In addition to constructing the detention pond, it is proposed that the existing overflow from Big Indian Slough to Higgins Slough be eliminated or at least controlled with an adjustable weir. This additional storage will help accommodate the additional peak flow that would be prevented from entering Higgins Slough. Elimination of stormwater discharge to Higgins Slough will reduce flooding potential in that basin.

In addition to constructing the detention pond, the existing concrete dam with the outlet pipes should be replaced. The existing concrete dam is almost 80 years old.

b. Big Indian Channel Widening

The existing slough from Farm-to-Market Road through Airport Higgins Way to culvert BI-C-5 is too narrow to convey peak stormwater flows. Widening this section of the slough will provide increased conveyance to accommodate the increase in stormwater runoff from development within the UGA.

c. Replace Culverts with Bridges

In conjunction with the channel widening, three existing culverts (BI-C-2, BI-C-3 & BI-C-5) need to be replaced. These three culverts provide access between local farm fields. Alternative field access may be available. If it is determined that these field access locations are necessary, then bridges are recommended to replace the culverts to prevent obstruction of the flow within the channel.

Additional pumping capacity is not recommended at this time. The ability for Big Indian Slough to discharge some stormwater during most high tide conditions without overtopping the levees is a significant advantage compared to the other drainage basins studied. Stormwater is able to discharge more efficiently through the outlet pipes at most high tides.

Before new projects are implemented, the analysis presented in this document should be updated to account for any new projects or changes in the slough system. If possible, additional modeling should be completed at a higher resolution at the specific project locations, using the most recent topographic data available.

Chapter 5
Stormwater Quantity Analysis

As part of this Stormwater Management Plan, stormwater hydraulic models of the drainage sloughs were developed to identify existing and potential drainage problems. These hydraulic models were also used to analyze the benefits of potential drainage improvements to address identified problems. This chapter summarizes the methods and results of the hydraulic modeling. **Chapter 6** outlines conceptual alternatives for eliminating flooding problems. These conceptual alternatives that demonstrate viability for mitigating flooding problems are incorporated into the Capital Improvement Plan presented in **Chapter 7**.

A. Hydraulic Model Development

The XPSWMM-v10 modeling program marketed by XP Software was used to assess hydrologic and hydraulic characteristics of the four primary slough-based drainage systems. This program is a commercially available pre- and post-processor for the Surface Water Management Model (SWMM) developed by the U.S. Environmental Protection Agency. The following describe the hydraulic model inputs and assumptions.

Hydrologic and hydraulic modeling were conducted for two previous studies in the study area: *Bay View Business and Industrial Park and Skagit Regional Airport Stormwater Master Plan (1998)* and *Hydrologic and Hydraulic Model of the No Name Slough Drainage (November 2000)*. Hydrographs for the 10- and 100-year storm events from the 1998 Master Plan were input to the model to represent airport runoff into Big Indian Slough. The hydrographs were routed through recent drainage improvements implemented by the Port of Skagit that would have otherwise been difficult to reproduce in SWMM. Also, since no calibration data for the study area is available, the modeling results in these reports were used as a check of the SWMM results where applicable.

1. Basin Development Scenarios

Three different development scenarios were conceptualized to simulate different development conditions. The three development scenarios are described as follows.

Predevelopment Scenario: This hydraulic model simulates stormwater drainage conditions prior to any development on the Bay View ridge along with current farming operations within the floodplain. The Bay View ridge area was modeled as a forest condition. This is consistent with analysis of predevelopment conditions outlined in the current Ecology Stormwater Management Manual[5].

[5] *Stormwater Management Manual for Western Washington,* Washington State Department of Ecology (February 2005)

Existing Development Scenario: This hydraulic model simulates stormwater drainage conditions from existing development on the Bay View ridge and surrounding farmland. This model, when compared to the Predevelopment Scenario, will provide the impact directly contributable to existing development on the Bay View ridge.

Future Development Scenario: This hydraulic model simulates stormwater drainage conditions resulting from development within the Study Area. This scenario expands beyond the development scenario used in the Phase 1 Plan, which only considered development within the proposed UGA. This model, when compared to the Existing Development Scenario, will provide the impact directly contributable to development within the Study Area.

2. Impervious Conditions

Existing effective impervious area [EIA] for each basin was determined using current aerial photographs; future EIA was estimated assuming full build-out conditions under Skagit County's current zoning coverage as of January 2003. The EIA for each zoning classification is shown in **Table 5-1.**

Table 5-1: Effective Impervious Area [EIA] Estimates For Zoning Classifications

Zoning Classification	Estimated EIA
Agriculture	5%
Rural Resource	5%
Public/Open Space	5%
Rural Villages	20%
Rural Intermediate	8%
Rural Reserve	6%
Commercial / Industrial	75%
Urban Growth Area	35%

3. Tide Conditions

The Bay View watershed modeling used a tidal cycle with high and low tide elevations of 3.85 feet and -4.55 feet was set at the downstream boundary condition, based on the mean higher high water and mean lower low water for this area of Padilla Bay. The timing of high tide was set to the approximate time of peak flow in the sloughs to give a conservative estimate of capacity.

A sensitivity analysis of different tidal cycles indicated that stay tides (tidal cycles with only one low tide during a 24-hour period) had little impact on flooding near the slough outfall. Peak discharge through the outfall tide gates always corresponded to the lowest tide elevation in the hydraulic model. In addition to the peak discharge at the lowest low tide, additional stormwater discharge seems to occur during the higher low tide of the stay tide, reducing flooding potential.

The tidal cycle has no influence on the middle and upper channel sections.

4. Rainfall Events

The Bay View watershed modeling used a 24-hour, single-event rainfall hyetograph to model the 10, 25, and 100-year rainfall events. The U.S. Soil Conservation Service's Type 1A rainfall distribution was used. Rainfall amounts were 2.3, 2.8, and 3.5 inches in 24 hours for the 10-, 25-, and 100-year events, respectively. Several small subbasins in the Study Area drain directly into Padilla Bay and were not included in the modeling of the four primary slough-based drainage systems.

5. Model Basin Descriptions

The following sections describe each basin and the elements included in the models for each. **Figure 5-1** shows the main drainages and the subbasin boundaries in the Study Area.

a. No Name Slough Modeling Basin

Figure 5-2 shows the modeled elements in the No Name Slough Basin and subbasin boundaries. The No Name Slough Basin is located on the west side of Bay View ridge.

No Name Slough basin drains approximately 2,700 acres. This basin was subdivided into 9 subbasins for the hydrologic modeling. The basin topography consists of steep uplands that drain into flat agricultural areas.

No Name Slough was modeled from its outlet into Padilla Bay to north of Marihugh Road. A small tributary from the southeast was also modeled. Key culverts at Bay View-Edison Road, Bay View Road, Marihugh Road, and Farm-to-Market Road were included in the SWMM modeling. Two other culverts were modeled; these culverts are not located on primary roads and appear to be located on access roads for the agricultural fields. There are two existing stormwater pump stations at the outfall. When the gravity outfall culverts cannot discharge stormwater due to tidal influence, these stormwater pump stations are considered to have a combined pumping capacity of 36 cfs in the hydraulic model.

Effective impervious areas for each subbasin were estimated for each development scenario based on past, existing or future land use. These were used in the hydraulic model to simulate stormwater runoff rates. **Table 5-2** lists the EIA for the modeled subbasin for each of the development scenarios.

Table 5-2: No Name Slough Effective Impervious Areas				
Basin Name	Total Area	Effective Impervious Areas for Each Scenario		
		Predevelopment	Existing	Future Development
A-4	489 acres	4.0%	4.0%	6.1%
A-5	306 acres	5.1%	5.1%	5.2%
A-6	100 acres	5.1%	5.1%	5.1%
A-7	325 acres	0.0%	10.2%	28.1%
A-8	127 acres	0.0%	4.0%	7.0%
A-11a	417 acres	0.0%	6.0%	21.3%
A-11b	672 acres	0.0%	6.0%	6.2%
A-11c	126 acres	0.0%	4.0%	5.5%
A-12	139 acres	0.0%	6.0%	7.3%
Totals	2,701 acres			

b. Joe Leary Slough Modeling Basin

Figure 5-3 shows the modeled elements in the Joe Leary Slough Basin and subbasin boundaries. The Joe Leary Slough Basin is located on the north and northeast side of Bay View ridge.

The Joe Leary Slough basin drains about 10,300 acres. This basin was subdivided into 20 subbasins for the hydrologic modeling. The upper portion of the basin drains primarily agricultural land. The topography in the upper basin is very flat and drainage is facilitated by the use of agricultural drainage tiles. The lower portion of the basin, which gets most of its runoff from the Bay View ridge area, is smaller than the upper portion of the basin. However, the topography along the north slope of Bay View ridge is much steeper and the resulting shorter time of concentration causes runoff from this area to produce sharper peak flows than runoff from the upper part of the basin.

The main stem of Joe Leary Slough forks into two tributaries, Maiben Road Ditch and South Spur Ditch, about 4 miles upstream from its outlet into Padilla Bay and downstream of the intersection of Benson Road and Thomas Road. Joe Leary Slough was modeled from its outlet to Avon-Allen Road along South Spur Ditch and Maiben Road Ditch. The SWMM program was used to establish the relationship of the tidal fluctuations in Padilla Bay with the capacity of the slough. Key culverts along the slough are also included in the hydraulic model.

Effective impervious areas for each subbasin were estimated for each development scenario based on past, existing or future land use. These were used in the hydraulic model to simulate stormwater runoff rates. **Table 5-3** lists the EIA for the modeled subbasin for each of the development scenarios.

Table 5-3: Joe Leary Slough Effective Impervious Areas				
Basin Name	Total Area	Effective Impervious Areas for Each Scenario		
		Predevelopment	Existing	Future Development
B-1a	116 acres	0.0%	5.0%	6.0%
B-1b	100 acres	4.0%	4.0%	4.0%
B-1c	189 acres	0.0%	6.0%	6.0%
B-1d	112 acres	5.0%	5.0%	5.0%
B-1e	108 acres	5.0%	5.0%	5.0%
B-2	244 acres	0.0%	4.0%	6.0%
B-3	495 acres	0.0%	5.0%	6.0%
B-4	148 acres	0.0%	5.0%	6.0%
B-5	86 acres	0.0%	5.0%	6.0%
B-6a	308 acres	5.0%	5.0%	5.0%
B-6b	233 acres	5.0%	5.0%	6.0%
B-6c	215 acres	0.0%	4.0%	5.0%
B-6d	112 acres	0.0%	5.0%	5.0%
B-7	933 acres	4.0%	4.0%	5.0%
B-8a	346 acres	0.0%	6.0%	11.8%
B-8b	537 acres	0.0%	6.0%	30.0%
B-9	1,867 acres	5.0%	5.0%	9.0%
B-10	589 acres	4.0%	4.0%	5.0%
B-11	910 acres	5.0%	5.0%	5.0%
B-12	2,634 acres	6.0%	6.0%	7.0%
Totals	10,282 acres			

c. Indian Slough Modeling Basin

Future development within the Indian Slough basin, both the Little Indian and Big Indian sloughs, is all within the Bayview Ridge UGA. Therefore, the Phase 1 Plan has already address anticipated drainage problems and proposed drainage facility improvements. This information is not repeated in this Phase 2 Plan. For information regarding the hydraulic modeling, drainage problems and proposed drainage facility improvements, please refer to the Phase 1 Plan.

B. Hydraulic Model Results

The hydraulic models were updated for the No Name Slough Basin and the Joe Leary Slough Basin. To better understand the new drainage problems from future development, the proposed drainage facilities from the Phase 1 Plan are considered to be constructed in the hydraulic model for the future development scenario. This will help identify additional problems resulting from development outside of the Bayview Ridge UGA. The results of the hydraulic modeling are presented below.

1. No Name Slough

Predicted peak flows in No Name Slough for the 10-, 25-, and 100-year storm events at various locations are listed in **Table 5-4**. **Table 5-5** compares the peak runoff rates for each development scenario in each subbasin.

The increase in stormwater flow rates between the Predevelopment Scenario and the Existing Development scenario are in the order of 5% at the outfall at Padilla Bay and 7% at the confluence with No Name Creek (node NN-83). The increase in stormwater flow rates between the Existing Development Scenario and the Future Development Scenario are in the order of 34% at the outfall to Padilla Bay and 35% at the confluence with No Name Creek.

The SWMM hydraulic model indicated flooding at locations throughout the basin. The flooding is indicated at the 10-, 25-, and 100-year recurrence interval for all three development scenarios. **Table 5-6** shows predicted flooding locations with no drainage improvements implemented.

Table 5-4: Peak Flows for No Name Slough

SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			Future Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-10	Outlet of Slough (before pumps)	104 cfs	143 cfs	185 cfs	110 cfs	150 cfs	185 cfs	139 cfs	180 cfs	228 cfs
NN-83	Confluence of Tributaries	89 cfs	121 cfs	168 cfs	95 cfs	128 cfs	177 cfs	120 cfs	154 cfs	200 cfs
NN-110	Marihugh Road	29 cfs	40 cfs	57 cfs	32 cfs	43 cfs	57 cfs	32 cfs	43 cfs	57 cfs

See **Figure 5-2** for node locations.

Table 5-5: Subbasin Peak Runoff for No Name Slough									
	Predevelopment Scenario			Existing Development Scenario			Future Development		
Subbasin	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
A-4	10 cfs	12 cfs	16 cfs	10 cfs	12 cfs	16 cfs	12 cfs	16 cfs	22 cfs
A-5	10 cfs	12 cfs	16 cfs	10 cfs	12 cfs	16 cfs	10 cfs	13 cfs	17 cfs
A-6	5 cfs	7 cfs	10 cfs	5 cfs	7 cfs	10 cfs	5 cfs	7 cfs	10 cfs
A-7	27 cfs	35 cfs	47 cfs	27 cfs	35 cfs	47 cfs	37 cfs	48 cfs	64 cfs
A-8	10 cfs	13 cfs	18 cfs	10 cfs	13 cfs	18 cfs	11 cfs	14 cfs	20 cfs
A-11a	23 cfs	31 cfs	43 cfs	23 cfs	31 cfs	43 cfs	29 cfs	38 cfs	52 cfs
A-11b	33 cfs	44 cfs	60 cfs	33 cfs	44 cfs	60 cfs	50 cfs	67 cfs	90 cfs
A-11c	15 cfs	20 cfs	28 cfs	15 cfs	20 cfs	28 cfs	16 cfs	21 cfs	29 cfs
A-12	7 cfs	10 cfs	13 cfs	7 cfs	10 cfs	13 cfs	7 cfs	10 cfs	13 cfs
See **Figure 5-2** for subbasin locations.									

Table 5-6: No Name Slough Flooding Locations with No Improvements										
SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			Future Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Upstream of							●	●	●
NN-60	Upstream Culv. NN-C2			●			●	●	●	●
NN-65	Lower Slough			●			●	●	●	●
NN-67	Middle Slough		●	●		●	●	●	●	●
NN-80	Upstream Culv. NN-C3		●	●		●	●	●	●	●
NN-83	Confluence	●	●	●	●	●	●	●	●	●
NN-170	S, Stem Near Dahlstadt Farm	●	●	●	●	●	●	●	●	●
See **Figure 5-2** for node locations.						● denotes predicted flooding for the storm event				

Ground elevations in the adjacent farm fields range between 2.0 to 3.5 feet. This elevation is lower than the high tide elevation used in the hydraulic model. This combination results in flooding of farm fields during most storm events regardless of the development scenario. Most of the impact is in the upper reaches of the slough furthest from the outfall.

2. Joe Leary Slough

Predicted peak flows in Joe Leary Slough for the 10-, 25-, and 100-year storm events at various locations are listed in **Table 5-7**. **Table 5-8** compares the peak runoff rates for each development scenario in each subbasin. Predicted flooding locations are illustrated in **Table 5-9.**

The increase in stormwater flow rates between the Predevelopment Scenario and the Existing Development scenario are in the order of 1% at the confluence of Maiben Ditch and South Spur Ditch (node JL-126) and 4% in the South Spur Ditch (node JL-190). There is no measurable difference at the outfall to Padilla Bay. The increase in stormwater flow rates between the Existing Development Scenario and the Future Development Scenario are in the order of 4% at the outfall (node JL-20) to Padilla Bay, 8% at the confluence, and 19% in the South Spur Ditch.

The culvert at Josh Wilson Road appears to have enough capacity for 100-year peak flows. The limiting factor for conveyance along South Spur Ditch appears to be the shallow slope and backwater effects from Maiben Ditch.

There is no indication of flooding along Maiben Ditch because adjacent ground elevations are high (typically above 13 feet) and stormwater is contained within the channel. This can not be said for the lower reaches of Joe Leary Slough and the South Spur Ditch. The hydraulic model indicates that ground elevations below 6 feet in the lower reach of Joe Leary Slough flood at all storm events modeled. The hydraulic model also indicates that ground elevations below 8 feet in the lower reach of South Spur Ditch will flood at all storm events modeled.

Table 5-7: Peak Flows for Joe Leary Slough

SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			Future Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
JL-20	Before Outfall Pipes	335 cfs	408 cfs	512 cfs	335 cfs	408 cfs	512 cfs	343 cfs	418 cfs	525 cfs
JL-60	Farm-to-Market Road	190 cfs	248 cfs	336 cfs	192 cfs	251 cfs	339 cfs	198 cfs	259 cfs	350 cfs
JL-80	Allen West Road	171 cfs	225 cfs	308 cfs	172 cfs	227 cfs	311 cfs	183 cfs	238 cfs	319 cfs
JL-126	Confluence	143 cfs	191 cfs	265 cfs	145 cfs	194 cfs	267 cfs	153 cfs	203 cfs	277 cfs
JL-190	Josh Wilson Road	48 cfs	64 cfs	86 cfs	50 cfs	66 cfs	89 cfs	59 cfs	78 cfs	105 cfs
JL-160	Maiben Ditch	44 cfs	58 cfs	82 cfs	44 cfs	58 cfs	82 cfs	44 cfs	58 cfs	82 cfs

See **Figure 5-3** for node locations.

Table 5-8: Subbasin Peak Runoff for Joe Leary Slough

	Predevelopment Scenario			Existing Development Scenario			Future Development		
Subbasin	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
B-1a	21 cfs	28 cfs	37 cfs	22 cfs	28 cfs	38 cfs	22 cfs	29 cfs	39 cfs
B-1b	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs
B-1c	18 cfs	24 cfs	32 cfs	18 cfs	24 cfs	34 cfs	18 cfs	24 cfs	34 cfs
B-1d	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs
B-1e	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs
B-2	18 cfs	24 cfs	34 cfs	19 cfs	25 cfs	35 cfs	19 cfs	25 cfs	35 cfs
B-3	33 cfs	44 cfs	60 cfs	34 cfs	45 cfs	62 cfs	34 cfs	46 cfs	63 cfs
B-4	17 cfs	23 cfs	31 cfs	18 cfs	24 cfs	33 cfs	18 cfs	24 cfs	33 cfs
B-5	10 cfs	13 cfs	18 cfs	10 cfs	13 cfs	18 cfs	10 cfs	14 cfs	19 cfs
B-6a	13 cfs	17 cfs	24 cfs	13 cfs	17 cfs	24 cfs	13 cfs	17 cfs	24 cfs
B-6b	31 cfs	41 cfs	56 cfs	31 cfs	41 cfs	56 cfs	31 cfs	41 cfs	56 cfs
B-6c	12 cfs	16 cfs	22 cfs	12 cfs	16 cfs	22 cfs	12 cfs	16 cfs	22 cfs
B-6d	26 cfs	33 cfs	44 cfs	27 cfs	35 cfs	46 cfs	27 cfs	35 cfs	47 cfs
B-7	22 cfs	30 cfs	42 cfs	22 cfs	30 cfs	42 cfs	25 cfs	33 cfs	44 cfs
B-8	37 cfs	48 cfs	66 cfs	38 cfs	50 cfs	68 cfs	50 cfs	64 cfs	86 cfs
B-9	33 cfs	43 cfs	57 cfs	33 cfs	43 cfs	57 cfs	50 cfs	63 cfs	83 cfs
B-10	19 cfs	25 cfs	35 cfs	19 cfs	25 cfs	35 cfs	19 cfs	25 cfs	35 cfs
B-11	21 cfs	27 cfs	36 cfs	21 cfs	27 cfs	36 cfs	21 cfs	27 cfs	36 cfs
B-12	64 cfs	82 cfs	110 cfs	64 cfs	82 cfs	110 cfs	64 cfs	82 cfs	110 cfs

See **Figure 5-3** for subbasin locations.

Table 5-9: Joe Leary Slough Flooding Locations with No Improvements

SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			Future Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Joe Leary Slough										
JL-20	Outfall	●	●	●	●	●	●	●	●	●
JL-30	Joe Leary	●	●	●	●	●	●	●	●	●
JL-40	Joe Leary	●	●	●	●	●	●	●	●	●
JL-50	Joe Leary		●	●		●	●		●	●
JL-60	Farm-to-Market		●	●		●	●		●	●
JL-70	Joe Leary			●			●			●
JL-80	Allen West Rd		●	●		●	●		●	●
JL-90	Joe Leary		●	●		●	●		●	●
JL-100	Joe Leary	●	●	●	●	●	●	●	●	●
JL-110	Joe Leary		●	●		●	●		●	●
JL-120	Joe Leary									
JL-126	Confluence									
South Spur Ditch										
JL-170	South Spur			●			●			●
JL-181	South Spur		●	●		●	●	●	●	●
JL-190	Josh Wilson Rd	●	●	●	●	●	●	●	●	●
JL-210	Michael Pl		●	●		●	●		●	●
JL-230	South Spur		●	●	●	●	●	●	●	●
JL-250	Avon-Allen Rd		●	●		●	●		●	●

See **Figure 5-3** for node locations.

● denotes predicted flooding for the storm event

Chapter 6

Storm Drainage Alternatives Analysis

Several conceptual alternatives for reducing flooding potential in each basin are proposed and evaluated below. The conceptual alternatives were selected for evaluation based on their probability of correcting flooding problems due to proposed development within the Bay View ridge area. Other alternatives or variations on these alternatives may become viable solutions as future alternative analysis proceeds. The optimal solution will most likely require a combination of the alternatives.

A. Conceptual Alternatives

Various combinations of drainage facility improvements were considered for relieving flooding in this basin. These improvements have not been optimized at this time, but are presented as a basis for further investigation. Other drainage improvements, or variations on these improvements, may become apparent as the drainage facility analysis proceeds. The optimal drainage solution will most likely require a combination of different drainage improvements.

The following conceptual drainage facility improvements were considered for reducing flooding potential due to development within the Bay View ridge area:

- **Replace Undersized Culverts.** Consider replacing existing culverts where hydraulic restrictions occur. Increasing the size of the culvert will reduce backwater affects and flooding potential upstream of the culvert.

  The downstream impacts of the upsized culvert will also need to be evaluated. Undersized culverts may be preventing downstream flooding. Increasing the culvert size may increase downstream flooding potential.

- **Widening Existing Channel.** Consider widening existing channel where hydraulic restrictions occur. Increasing the width of the channel section will reduce backwater affects and flooding potential upstream.

  The downstream impacts from channel widening will also need to be evaluated. A constricting channel may be preventing downstream flooding. Increasing the channel capacity may increase downstream flooding potential.

- **Bypass Channel.** Consider a bypass channel in areas where the existing channel travels further than necessary. A bypass channel can provide a shorter route for the stormwater runoff, resulting in increase overall channel capacity. A bypass channel usually only affects the flooding potential within the area of the existing parallel channel.

  A bypass channel can also increase downstream flooding potential by increasing the stormwater runoff rate.

- **Increase Outfall Capacity.** The tidal condition affects the capacity of the existing outfall pipes to Padilla Bay. If the capacity of the existing outfall pipes in not sufficient to discharge the required stormwater runoff during a given tidal cycle, then increasing the number and/or size of outfall culverts may reduce upstream flooding potential.
- **Construct Levees.** Consider constructing levees along drainage channels where the adjacent ground elevations are too low to prevent flooding. A drawback to levee construction is that natural drainage patterns from the farm fields are disrupted, potentially resulting in poor drainage during even small storm events.
- **Regional Detention.** A regional detention pond could reduce or delay the amount of runoff entering the slough and perhaps eliminate flooding throughout the area. Detention could also add the benefit of water quality treatment to remove sediment or other pollutants from reaching Padilla Bay. Further analysis would be needed to determine the optimal size and location of the pond.
- **Pump Station.** A stormwater pump station can decrease the upstream hydraulic grade elevation, resulting in an increase in flow in the upstream channel and a decrease in flooding. A pump station can also increase downstream hydraulic grades which will increase downstream channel flow rates but also increase flooding potential. The impacts of a proposed pump station on downstream flooding will need to be evaluated.

B. No Name Slough

Flooding in the No Name Slough drainage basin has been widespread in the lower reaches. Dike, Drainage and Irrigation District No. 12 has made several drainage improvements since taking over the district in 2004. Some recent improvements include: 1) construction of a bypass channel through the middle reach of the slough, 2) removal of culvert NN-C-3 and replacement with a bridge, and 3) upsizing of culvert NN-C-5. Other drainage facility improvements to be constructed include additional outfall pumping capacity and channel widening upstream of the confluence of tributaries.

Stormwater runoff in the steep portions of the drainage basin causes considerable erosion of the stream channel. The following conceptual alternatives are proposed to reduce this erosion potential and relieve flooding in the lower reaches of the drainage basin.

1. Marihugh Road Regional Detention Facility

Future development within the No Name Slough basin is anticipated to occur north of Marihugh Road. A regional detention pond near where No Name Stream crosses Marihugh Road would provide two functions that are priorities within this basin, reducing the erosion potential within the No Name Stream and the flooding potential in the low lying farmland. There are two other detention ponds already existing within the No Name Slough Basin; one on the Paccar property and one on the Port property near the intersection of Farm-to-Market Road and Ovenell Road. Both of these existing detention ponds are within the Bayview Ridge UGA.

A regional detention pond near Marihugh Road would primarily serve future development outside of the Bayview Ridge UGA. The 25-year peak flow at this location is estimated to be 42 cfs under future development conditions. The target peak flow rate out of a detention pond would be approximately 18 cfs. This would require the detention pond to be approximately 32 acre-feet in sized.

If land use conditions change to allow more urban-type development in the area north of Marihugh Road, the size of a regional detention pond at this location will need to be increased to accommodate the increased stormwater runoff generated.

2. Marihugh Road Bypass Pipeline

A regional detention pond at Marihugh Road would help control flows downstream in the No Name Stream, but all of the stormwater would still need to be conveyed through the No Name Slough and through the outfall structures. A solution that would reduce the volume of stormwater from entering the No Name Stream and No Name Slough is to bypass peak flow through a pipeline along Marihugh Road. The pipeline outfall could be through a new outfall into Padilla bay or into the WDFW property along Bay View-Edison Road near the intersection with Marihugh Road.

Discharge through a bypass pipeline will need to be controlled and/or dissipated by some means to prevent downstream erosion. This could be accomplished by either hydraulically operated control valves and/or flow control structures.

A bypass pipeline along Marihugh Road may also reduce the required size of the Marihugh Road regional detention pond if a new outfall or downstream retention is developed at the end of the bypass pipeline.

3. No Name Slough Drainage Improvement Recommendations

The hydraulic model indicates that there is wide spread flooding in the low-lying farmland during all three development scenarios and there is obvious evidence of erosion in the No Name Stream channel. Additional drainage improvement recommendations for No Name Slough beyond the recommendation presented in the Phase 1 plan include the following:

- Marihugh Road Regional Detention Pond.
- Marihugh Road Bypass Pipeline and Outfall.

Table 6-1 illustrates the reduction in flooding potential with the proposed improvements listed above for the No Name Slough Basin. Only in the upper reaches of the slough is flooding still predicted at the 25-year storm event.

Table 6-1: No Name Slough Flooding Locations with and without Proposed Improvements										
SWMM Model Node	Approximate Location	Existing Development Scenario with No Improvement			Future Development Scenario with No Improvement			Future Development Scenario with Proposed Improvements		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Outlet				●	●	●			●
NN-30	Lower Slough				●	●	●			●
NN-40	Lower Slough				●	●	●			●
NN-60	Middle Slough			●	●	●	●			●
NN-65	Middle Slough			●	●	●	●			●
NN-70	Culvert NN-C-3		●	●	●	●	●			●
NN-83	Confluence	●	●	●	●	●	●	●	●	●
NN-140	Upper Slough	●	●	●	●	●	●	●	●	●
NN-160	Upper Slough	●	●	●	●	●	●	●	●	●
NN-170	Culvert NN-C-5	●	●	●	●	●	●	●	●	●
See **Figure 5-2** for node locations.						● denotes predicted flooding for the storm event				

4. No Name Slough Development Regulation Recommendations

All new development and redevelopment activities in the No Name Slough Basin should be required as a minimum to include:

- Best management practices (BMPs),
- Operational and structural source control BMPs,
- Treatment BMPs included in the 2005 Stormwater Manual for mitigation of the water quality impacts.

Channel erosion and flooding are still predicted at the 25-year storm event. The flow control within this basin should require that stormwater discharges match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 50% of the 2-year peak flow up to the full 100-year peak flow.

The pre-developed condition to be matched shall be a forested land cover in all cases for land above the flood plain. Pasture conditions may be considered for agriculture-related development activities within the flood plain. Downstream analysis for many projects may need to extend down to the flood plain level of the No Name Slough system.

C. Joe Leary Slough

The flooding problems in Joe Leary Slough appear to be concentrated along the low lying areas between the outfall and Farm-to-Market Road and the along the South Spur Ditch. The low hydraulic gradient of the slough and the large impact of tidal influence restrict the capacity of Joe Leary Slough to convey stormwater runoff.

The Joe Leary Slough drainage area is the largest of the four drainage basins evaluated as part of this Plan. Unlike many other large drainage basins in the Skagit Valley, Joe Leary Slough does not have a pump station at its outfall to assist in stormwater drainage during periods of high tide. Because of the large size of its drainage basin, a pump station would need to be large to provide the sufficient benefit. Therefore, drainage improvements within the Joe Leary Slough first focused on improving and optimizing the channel efficiency; allowing the maximum amount of stormwater runoff to be discharge to Padilla Bay during the low tide cycles. Drainage alternatives that were evaluated include a bypass channel along the lower reaches of Joe Leary Slough, a pump station at the South Spur Ditch, and widening the existing channel in areas that demonstrate channel restrictions.

1. Bypass Channel Alternative

Drainage and Irrigation District No. 14 requested the study include an analysis of an alternative to the Peth Bypass through widening of the existing channel. The existing channel is 6500 LF as compared to the Peth Bypass of 2300 LF. The amount of fall in the channel is the same for both. The widened main stem channel will be trapezoidal in shape with the following minimum characteristics:

- 11-foot widening for 25 foot minimum bottom width
- 2:1 side slopes
- 10 feet of total depth
- Average slope of 0.0018 percent
- Manning's 'n' roughness coefficient of 0.045

The modeling shows that the alternate will result in similar lowering of the water surface but responds slower or later during the storm event than the Peth Bypass. The Peth Bypass is steeper and a more efficient channel section. The alternative is a longer channel improvement and results in a 20% increase in excavation and more land acquisition. This increase in excavation increases the channel storage which offsets the slow movement of the water to the outfall. The Peth Bypass can pass a larger volume of water in a shorter time when a tide is low or receding. The alternative will increase the channel bottom to 25 feet which will result in more difficult maintenance especially along D'Arcy Road. The alternative was also modeled after removal of the 15 foot arch culvert near node JL-40 which will need to be replaced by a bridge.

This analysis shows that an alternative is feasible to receive similar results if this is the best alternative for the stakeholders. Because the alternative is less efficient, requires more excavation, requires more land and may make maintenance of the large channel width difficult and more expensive, the Peth Bypass is still the preferred alternative.

2. Saltwater Side of Joe Leary Slough Outfall

Drainage and Irrigation District No. 14 has questioned the capacity of the saltwater side, downstream of the 12-48” culvert outfall to Joe Leary Slough. Survey work of the saltwater channel cross-sections has been performed downstream of the outfall in June of 2009. In addition, the channel conditions were observed in detail in 2001 and in part in 2003 in conjunction with the inventory portion of the Phase 1 Plan. In addition to these field observations, there are two aerial surveys from August 1998 and April 2002 that provide additional topographic detail of the saltwater channel.

The 12-48” culverts with tide gates at the outfall provide a flow area of 75 SF. The slope of the channel bottom west of Farm to Market Road is approximately 0.0018 percent. From the April 2002 aerial survey, the slope of the outfall water surface was measured and three cross sections were analyzed. Assuming that the water surface profile was similar to the channel slope, the saltwater side was found to have an average slope of 0.006 percent and a minimum flow area approximately 400% larger than the outfall pipes.

It appears from this analysis that the saltwater channel performance is dictated by the tidal cycle and that the saltwater channel has more flow capacity at a receding tide than Joe Leary Slough and outfall structure. A more detailed analysis of this outfall system can and should occur with a more detailed analysis of an outfall pump station.

3. Joe Leary Slough Channel Widening

Phase 1 Plan indicated that widening of the channel to 13 feet from Allen West Road to the Maiben Ditch confluence would mitigate for the development in the Bayview Ridge UGA. This Phase 1 Plan also indicated that a 15 foot bottom would optimize the channel section to the capacity of the 15-foot wide arch culvert at Allen West Road and the 15-foot wide arch culvert at Benson Heights Place. These two culverts have more capacity than the channel in this vicinity. Widening the channel will make the channel capacity match the culvert capacity. The current width of the channel in this section is between 8.7 feet and 14.3 feet. The length of the proposed channel widening is approximately 9,000 LF. The widened channel will be trapezoidal in shape with the following minimum characteristics:

- 15-foot bottom width
- 2:1 side slopes
- Manning’s ‘n’ roughness coefficient of 0.045

4. Detention at Outlet

Detention at the outlet was examined conceptually. It is not known whether land is available for detention, but a sensitivity analysis was completed to estimate what effect detention could have on water surface elevations at the outlet. Detention volumes of 20 acre-feet and 70 acre-feet were examined. Because ground elevations near the outlet are low, the storage area would likely require a very large area. For example, near the outlet, where only about 3 feet of storage depth is available, the required pond area would be approximately 20 acres. Because of the large land areas required, larger pond volumes were not examined.

Given the large volume of water generated during a peak event, considerable storage appears to be required to have an appreciable effect. The analysis indicates that a 20-acre-foot pond would have no appreciable effect on water surface elevations. In addition, a 70-acre-foot pond would decrease water levels by a maximum of 0.3 feet for the 100-year future conditions storm event. Because of the large area of land that would be required to provide the required storage, this option was not examined in any further detail.

5. Flood Easements

A flooding easement is a management tool that can be examined for application in the Joe Leary Slough basin. A flooding easement is a negotiated agreement between a drainage control party, such as the County or the Drainage District, and a property owner. The flooding easement would describe the potential area that may be flooded for a given storm event. The agreement would stipulate financial compensation to the property owner for damages incurred as a result of a flooding event. The advantage of a flooding easement is that, in many cases, it can be negotiated quicker than the design and construction of drainage improvements. Flooding easements may also be used as temporary measures to provide financial protection to property owner now while storm drainage improvements are studied, designed and constructed.

6. Pump Station at the Outfall

A pump station at the outfall was examined as a way to reduce water surface elevations in the slough during high tide. A range of peak pump capacities were examined to estimate the effectiveness of a pump station at the outfall. The results indicate that in order for a pump station to be effective, its capacity must approach the peak flow in the slough. Because of the high volume of water produced by the design storm, even a small decrease in the capacity of the outfall culverts can exceed the storage capacity of the slough, raising water levels in the slough and cause flooding of adjacent fields that have low ground elevations. Therefore, if a pump station’s capacity does not approach the expected peak flow in the slough, the storage volume of the slough will be exceeded and the slough will flood. For the 10-year design storm in conjunction with the design tidal conditions, the capacity of the pump station needed to mitigate flooding is approximately 300 cubic feet per second (cfs).

According to the model results, a pump station at the outfall would provide the most benefit from the outfall of the slough to approximately Farm-to-Market Road. Larger pump stations, which might deter flooding for larger storms were not examined in detail because they were deemed impractical to construct and operate. A smaller pump station might be effective at reducing flooding for smaller design storms and/or different tidal conditions, but these storms were not examined as part of the study. Before any pump station is designed or constructed, additional detailed modeling should be completed to determine specific benefits that could be expected.

Alternatives to this pump station would be increasing the channel storage or acquiring flood easements west of Farm to Market Road. As development density increases above the density in this study or as the rural portions in this study reach full build out, the alternatives may no longer provide sufficient results without a pump station. A more detailed study needs to be performed to determine what the cost benefit/cost of possible alternatives has in reducing or eliminating the pump station. Determining an acceptable depth of flooding or in other words raising the flood assumptions or increasing the depth of ponding may be the only way to eliminate a pump station. Raising the flood depths will require some berms or levees in addition to the flood easements.

7. Pump Station at the South Spur Ditch

Drainage and Irrigation District No. 14 believes that existing flooding impacts on farmland along the South Spur Ditch are unacceptable. Phase 1 Plan only mitigated for the impacts from development in the Bayview Ridge UGA but did not mitigate for the current condition that the District thinks in unacceptable. A pump station on the South Spur Ditch near node JL-161 was examined as a way to reduce water surface elevations in that portion of the slough.

A pump station with a capacity of 60 cfs could reduce water surface elevations by 3.5 feet at the pump station and by 2 feet at Michael Place. This is more than can be accomplished with any combination of channel widening and/or regional detention.

A South Spur Pump Station will require development upstream to provide detention in order to buffer the pump station during storm event for the 10-year and larger storm events. Detention should be required for all development in the Bayview Ridge UGA until a detailed study is completed that would indicate a reduced flow control standard is appropriate.

There are only a few large-tract land owners on Bay View ridge that discharge into the South Spur Ditch. Drainage and Irrigation District No. 14 may be able to negotiate directly with potential developers to help fund the study and construction of the South Spur Pump Station.

8. Joe Leary Slough Drainage Improvement Recommendations

Additional drainage improvement recommendations for Joe Leary Slough beyond the recommendation presented in the Phase 1 Plan include the following:

- Widening Joe Leary Slough south of Allen West Road up to the confluence with Maiben Ditch and South Spur Ditch an additional 2 feet from the Phase 1 Plan recommendation to a minimum width of 15 feet. To accommodate this channel widening, four existing wooden bridges also need to be replaced.
- Pump Station at the Outfall with study of the Flood Easement and additional channel storage.
- South Spur Pump Station and detention for developments upstream. Design of the pump station should include a determination if a reduction in the flow control requirements for upstream development would be acceptable.

Table 6-2 illustrates the reduction in flooding potential with the proposed improvements listed above for the Joe Leary Slough Basin.

9. Joe Leary Slough Development Regulation Recommendations

All new development and redevelopment activities in the Joe Leary Slough Basin should be required as a minimum to include:

- Best management practices (BMPs),
- Operational and structural source control BMPs, and
- Treatment BMPs included in the 2005 Stormwater Manual for mitigation of the water quality impacts.

Channel erosion is not an issue for the main stem of Joe Leary Slough and the South Spur Ditch portion of the drainage system.

Until such time that more detailed study occurs on both the South Spur Pump Station and the Outfall Pump Station that indicates an appropriate reduced flow control standard, all new development and redevelopment in this basin should require that stormwater discharges match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 50% of the 2-year peak flow up to the full 100-year peak flow.

The pre-developed condition to be matched shall be a forested land cover in all cases for land above the flood plain. Pasture conditions may be considered for agriculture-related development activities within the flood plain. Downstream analysis for projects within the UGA may need to extend down to main stem of South Spur Ditch (e.g., Node JL-200). Downstream analysis for projects outside the Bayview Ridge UGA may need to extend down to main stem of Joe Leary South.

Table 6-2: Joe Leary Slough Flooding Locations with and without Proposed Improvements

SWMM Model Node	Approximate Location	Existing Development Scenario with No Improvement			Future Development Scenario with No Improvement			Future Development Scenario with Proposed Improvements◊		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year†	100-Year†
Joe Leary Slough										
JL-20	Slough Outlet	●	●	●	●	●	●	●	●	●
JL-30	Lower Slough	●	●	●	●	●	●	●	●	●
JL-40	Lower Slough	●	●	●	●	●	●	●	●	●
JL-50	Lower Slough		●	●		●	●			●
JL-60	Farm-To-Market Rd		●	●		●	●			●
JL-70	Middle Slough			●			●			
JL-80	Allen West Road		●	●		●	●		●	●
JL-90	Middle Slough		●	●		●	●			●
JL-100	Middle Slough	●	●	●	●	●	●		●	●
JL-110	Middle Slough		●	●		●	●			●
JL-120	Middle Slough									
JL-126	Confluence									
South Spur Ditch										
JL-170	South Spur			●			●			
JL-181	South Spur		●	●	●	●	●		●	●
JL-190	Josh Wilson Road	●	●	●	●	●	●		●	●
JL-210	Michael Place		●	●		●	●		●	●
JL-230	South Spur		●	●	●	●	●		●	●
JL-250	South Spur		●	●		●	●		●	●

See **Figure 5-3** for node locations. ● denotes predicted flooding for the storm event

◊ Future development scenario does not include the outfall pump station.

† The South Spur pump station is only included in the 10-year future development scenario..

Chapter 7
Capital Improvement Plan

The stormwater drainage projects presented here are proposed for consideration to reduce or eliminate existing and/or future flooding conditions within the Bay View watershed as a result of potential future development within the Bay View ridge area. Some projects are simple, consisting of replacing or upsizing existing culverts. Other projects are more complex, such as new or expanded pump stations, channels and detention ponds, which will require additional hydraulic modeling, evaluation and optimization in order to determine the appropriate and cost effective design criteria.

Operation, maintenance and replacement costs for existing and proposed stormwater facilities are also an essential part of a fully-functioning stormwater drainage system. Skagit County Drainage Utility should work closely with the Drainage Districts to ensure these ongoing costs are adequately funded.

Taxation and revenue generation to finance regional drainage system improvements will come from three primary sources, the Drainage District's property assessments, the Skagit County Drainage Utility, and special assessments of properties within the Bay View ridge area. A breakdown of estimated financial contributions by these three entities is not part of this Plan.

A. Cost Estimating Methodology

Cost estimates presented within this Capital Improvement Plan are considered "Concept Budgetary Estimates". Construction cost estimates are made without design plans. These project cost estimates should be considered funding "goals". Detailed project cost estimates will need to be developed during the project planning and design phases.

All project costs are adjusted to March 2010 pricing levels. Project costs proposed to begin much beyond this time frame should be adjusted for potential price escalation.

1. Construction Cost Index

The *Civil Works Construction Cost Index System [CWCCIS]*[6] prepared by the US Army Corp of Engineers was used to adjust historical construction cost to March 2010 cost. The purpose of this manual is to provide historical and forecasted cost indexes for use in escalating civil works project costs. Cost data used to develop the cost indexes were derived from several published sources.

[6] *Civil Works Construction Cost Index System (CWCCIS),* US Army Corps of Engineers, Manual No. 1110-2-1304, March 31, 2000 (Tables Revised as of March 31, 2010)

The Composite Index has 19 Civil Work Breakdown Structure [CWBS] feature codes. The CWCCIS also provides State correction factors, which allows the user to adjust construction costs from one State to another.

2. Pump Station Construction Costs

Stormwater pump station costs were estimated using parametric estimating, which is a technique using a statistical relationship between historical data and other variables such as pump station capacity. Data from nine existing stormwater pump station project costs were used to estimate the pump station project costs presented in this Plan. These nine pump station capacities ranged from 18 cfs to 1,250 cfs. The historical costs were adjusted to March 2010 cost using the CWCCIS Pumping Plant Cost Index.

The capacity and project cost data for the nine pump stations were analyzed using bivariate power regression to calculate an estimating equation. Regression analysis is a statistical tool employed to compute and evaluate a proposed mathematical relationship between two variables. In this analysis, the pump station capacity is the independent variable and project cost is the dependent variable. The resulting correlation is expressed in the following equation.

$$\text{Pump Station Project Cost (in \$)} = 0.0723 \times (\text{Capacity in cfs})^{0.8054}$$

The square of the correlation coefficient is the portion of the total variability in the dependent variable that is explained by the independent variable. The square of the correlation coefficient for this analysis is expressed below.

$$R^2 = 0.9684$$

3. Culverts

Proposed culverts construction costs are estimated based on a schematic layout. Construction costs include a gross estimate of excavation, hauling and disposal of earth material, pipe material, and imported fill material, along with an appropriate estimate of restoration. Indirect costs, which include planning, surveying, geotechnical investigations, design, permitting, project management, construction management, financing costs and construction cost contingencies, were estimated to be 50 percent of the construction cost estimate. No additional land costs are expected for culvert installations. The historical costs were adjusted to March 2010 cost using the CWCCIS Cost Index.

4. Channel and Detention Pond Excavation

Proposed channels and detention ponds are estimated based on a schematic layout. Construction costs are estimated based on a gross estimate of excavation, hauling and disposal of earth material, along with an appropriate estimate of restoration. Indirect costs, which include planning, surveying, geotechnical investigations design, permitting, project management, construction management, financing costs and construction cost contingencies, were estimated to be 50 percent of the construction cost estimate. Land costs, in the form of easements or simple fee purchases, are expected for new channels and detention ponds, but are not estimated or included in these project cost estimates.

B. Capital Improvements

Table 7-1 provides a proposed Capital Improvement Plan for planning, design and construction of the stormwater drainage facilities in the Bay View watershed to accommodate future growth within the Bay View ridge area. A description of proposed capital improvements in each basin is described below. The costs allocation in future years has been escalated to account for inflation based on the *Civil Works Construction Cost Index,* which is derived from projection published by the Office of Management and Budget. The average inflation rate for the past 4 years has been approximately 3 percent for the *Civil Works Construction Cost Index*.

1. No Name Slough Recommendations

Dike, Drainage and Irrigation District No. 12 has been continuously making improvements to No Name Slough. Before any project is carried forward, the hydraulic model should be updated to account for any projects that have been completed at that time and for changes in existing or expected land use. In addition, it is recommended that additional modeling be performed to better define design criteria for these conceptual project. The following are additional alternatives beyond the Phase 1 Plan that are recommended for the No Name Slough basin:

a. Marihugh Regional Detention Pond

The Marihugh regional detention pond is a 23 acre-foot regional detention pond located within or adjacent to the No Name Stream near Marihugh Road. The project will involve additional study to optimize the detention pond size and location, acquisition of property, and construction of the detention pond and outfall control structure.

b. Marihugh Road Bypass Pipeline

The Marihugh Road bypass pipeline is a gravity drainage interceptor pipe constructed along Marihugh Road from the regional detention pond to a new outfall near or within Padilla Bay. The project will involve a study to identify the best interaction with the Marihugh regional detention pond, coordination and involvement of the Washington State Department of Fish & Wildlife, permitting for a possible new outfall to Padilla Bay, construction of the gravity pipeline, and reconstruction of Marihugh Road.

Table 7-1: Recommended Capital Improvements for the Bay View Watershed						
Drainage Basin and Proposed Stormwater Capital Improvement	Project Cost Estimate (FY 2010)	Projected Capital Improvement Costs with Escalation[1]				
		FY 2011	FY 2012	FY 2013	FY 2014	FY 2015
No Name Slough Basin						
23 ac-ft Marihugh Regional Detantion Pond	$ 1,450,000			$ 505,000	$ 1,027,000	
Marihugh Road Bypass Channel	$ 1,675,000			$ 583,000	$ 1,186,000	
Joe Leary Slough Basin						
Joe Leary Slough Channel Widening	$ 223,000			$ 78,000	$ 158,000	
300 cfs Outfall Pump Station	$ 6,700,000					$ 7,238,000
60 cfs South Spur Pump Station	$ 1,900,000					$ 2,053,000
Totals	$ 11,948,000	$ -	$ -	$ 1,166,000	$ 2,371,000	$ 9,291,000
Note 1: Escalation is per EM 1110-2-1304, Civil Works Construction Cost Index System (31 March 2010)						

2. Joe Leary Slough Recommendations

As with the other drainage basins in the Bay View watershed, flooding in Joe Leary Slough is largely driven by the tidal cycle. Since ground elevations of adjacent agricultural fields are often in the range of 5 to 10 feet, stormwater drainage alternatives that can reduce flooding are limited. In addition to the recommended drainage improvements presented in the Phase 1 Plan, the following drainage improvements are recommended:

a. Joe Leary Slough Widening

The existing slough from Allen West Road to the confluence of Maiben Ditch and South Spur Ditch has channel restrictions. The Phase 1 Plan proposed widening this section of the slough to a bottom width of 13 feet to increase conveyance that is equivalent to the existing capacity of the arch culvert at Allen West Road. The length of this section of Joe Leary Slough is approximately 9,000 LF. Additional widening of 2 feet for this section of the slough to a bottom width of 15 feet will also increase the conveyance along this reach of the slough.

b. Outfall Pump Station

A pump station at the outlet is the most effective way of reducing flooding in the lower portion of the slough. For the events analyzed, the capacity of the pump station must be nearly equal to that of the expected peak flow in the slough. For the 10-year event, the pump station capacity must be approximately 300 cfs. This size of a pump station would be expensive to construct and to operate. Before a pump station of this size is considered, further study should be done on the acceptable flood stage downstream of Allen West Road or increasing the available channel and flood storage. Smaller pump stations that would reduce more frequent flooding (the 5-year or 2-year event, for example) may be more cost-effective but have not been analyzed. Before a smaller pump station is proposed, additional modeling is required to determine the potential benefits and necessary operating conditions. All subbasins would contribute to the outfall pump station.

c. South Spur Pump Station

A pump station on the South Spur Ditch would lower water surface elevations and reduce flooding on the South Spur Ditch. To reduce flooding for the 10-year event, the pump station capacity must be approximately 60 cfs. The pump station would likely cause a small increase in water surface elevations downstream and might increase flooding, depending on the downstream hydraulic conditions in the slough. For the pump station to have the maximum benefit the channel should be widened to match Cross Section BX-17 (see **Appendix A**). Subbasins B-8 and B-9 would contribute to a pump station on the South Spur Ditch.

Before new projects are implemented, the hydraulic analysis should be updated to account for any improvements or changes in the slough system. If possible, additional modeling should be completed at a higher resolution at the specific project locations using the most recent topographic data available.

C. Stormwater Management Strategies

There are several stormwater management strategies recommended in the Phase 1 Plan to be instituted in the Bay View watershed. These stormwater management strategies are not repeated in the Plan.