Bay View Watershed
Stormwater Management Plan

March 2006 Draft

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Chapter 1
Executive Summary

The Bay View area has developed with an inadequate understanding of the impacts of stormwater runoff. As a result, several stakeholders within the watershed have expressed concerns regarding the quality and quantity of stormwater being discharged to the adjacent sloughs and Padilla Bay. The purpose of the *Bay View Watershed Stormwater Management Plan* is to: 1) inventory stormwater drainage facilities within the watershed; 2) develop stormwater hydraulic models in order to understand current and future drainage impacts; 3) propose stormwater facility improvements; and 4) propose stormwater management strategies to manage drainage within the Bay View area and to reduce farmland flooding. Skagit County funded the preparation of this Plan from its Drainage Utility fund.

The Bay View Watershed Stormwater Management Planning 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 and around the Skagit Golf and County Club. 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 the Skagit County, dike and drainage districts, Port of Skagit County, 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 area. These documents include the *Padilla Bay/Bay View Watershed Non-Point Action Plan*, the *Port of Skagit County Stormwater Management Master Plan*, the report entitled *Hydrologic and Hydraulic Model of the No Name Slough Drainage,* the *Bayview Ridge Subarea Plan*, the *Joe Leary Slough Drainage Study*, and the *Inventory and Evaluation of Tide Gates and Pump Stations related to Alternatives #5 and #7 of the Skagit River Flood Damage Reduction Feasibility Study.* 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 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. The model results indicated that there are areas of potential flooding along each of the four major drainage sloughs. Conceptual stormwater drainage improvements were developed and evaluated that could correct capacity limiting facilities. Potential drainage facility improvements that were evaluated included the following:

- Enlarging and regrading slough channels,
- Regional detention,
- Stormwater pump stations,
- Bypass channels,
- Increasing levee heights, and
- Upsizing culverts.

The Capital Improvement Plan of the proposed drainage facilities improvements is presented in **Chapter 8** for each drainage basin. A summary of the proposed improvements are presented below.

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	Increase Pump Station Capacity	$ 2,600,000
	32 ac-ft Marihugh Road Detention Pond	$ 1,257,000
	Various Culvert and Outfall Additions and/or Replacement	$ 135,000
Joe Leary Slough Basin	Peth Property Bypass Channel	$ 820,000
	300-cfs Outfall Pump Station	$ 6,300,000
	60-cfs South Spur Pump Station	$ 1,700,000
Little Indian Slough Basin	Culvert Replacement and Increase Channel Capacity	$ 124,000
Big Indian Slough Basin	Ovenell Road Bypass Channel and Higgins Slough Overflow Modifications	$ 4,850,000
Total Capital Improvement Cost Estimate		$ 17,786,000

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 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, bioswales, and constructed wetlands.

Chapter 2
Introduction

The Bay View Watershed is located in the westerly portion of Skagit County, west of the City of Burlington. A Vicinity Map of the Bay View Watershed Management Planning Area is shown on **Figure 2-1.** The Vicinity Map shows the outline of the Study Area that was used for this Plan.

A. Purpose and Scope

The Bay View Watershed has developed with an inadequate understanding of the impacts of stormwater runoff. As a result, several stakeholders within the area have expressed concerns regarding the quality and quantity of stormwater being discharged to the adjacent sloughs and Padilla Bay. The purpose of the *Bay View Watershed Stormwater Management Plan* is to: 1) inventory stormwater drainage facilities within the watershed; 2) develop stormwater hydraulic models in order to understand current and future drainage impacts; 3) propose stormwater facility improvements; and 4) propose stormwater management strategies to manage drainage within the Bay View area and to reduce farmland flooding.

B. Stakeholders Purpose and Objectives

There are several entities that have a stake in stormwater drainage planning in the Bay View area. These entities are discussed below. The stormwater planning objectives of each stakeholder is also discussed.

1. Skagit County

Skagit County Surface Water Management (herein called the County) is the lead agency for the *Bay View Watershed Stormwater Management Plan* (herein called the Plan); providing project management, and project funding. The County provides representation for the residents and property owners within the Study Area. County government is developing the Plan to provide a means to minimize present and future stormwater impacts to the citizens of the County and their properties located within the Study Area. The County has many interests in the Study Area including development of a programmatic Environmental Impact Statement [EIS] for the proposed non-municipal Urban Growth Area (UGA). In addition, a large portion of the project area is located in the County's Drainage Utility service area and much of the stormwater from the Drainage Utility service area is discharged to adjacent Drainage District facilities. Therefore, the County has a vested interest in working with the Drainage Districts to mitigate potential impacts from stormwater runoff.

2. Drainage District No. 14

Drainage District No. 14 owns and maintains drainage ditches and outfalls in the farmland areas north and northeast of the Study Area. A portion of the service area for Drainage District No. 14 is shown on **Figure 2-2.** The District's primary drainage channel is Joe Leary Slough, which forms the north and northeast boundary of the Study Area. Joe Leary Slough discharges by gravity to Padilla Bay through tide gates located downstream from Bay View-Edison Road. Drainage from portions of the Study Area directly impact stormwater conveyance within Joe Leary Slough and its outfall to Padilla Bay. Many of the District's stormwater management objectives are presented in the *Joe Leary Slough Drainage Study*, which is discussed later in this chapter.

3. Drainage District No. 19

Drainage District No. 19 owns and maintains drainage ditches and outfalls in the farmland areas south and southeast of the Study Area. A portion of the service area for Drainage District No. 19 is shown on **Figure 2-2.** The District's primary drainage channels are Little Indian Slough and Big Indian Slough, which forms the south and southeast boundary of the Study Area, and Higgins Slough, which is located south and southwest of the Study Area. Little Indian Slough discharges by gravity to Padilla Bay through tide gates. Big Indian Slough discharges to Padilla Bay through tide gates and/or a pump station. Drainage from portions of the Study Area directly impact storage or conveyance within the sloughs and their outfall into Padilla Bay.

The District's objectives for stormwater management within the Study Area are twofold. First, the District supports measures that reduce erosion and sedimentation to and within its stormwater conveyance systems, which reduces its maintenance requirements. Second, the District supports measures that reduce peak stormwater runoff, which has the potential to overload its existing conveyance capacities resulting in localized lowland flooding.

4. Dike and Drainage District No. 8

On February 3, 2004, the property owners within Dike and Drainage District No. 8 voted to be incorporated in Dike District No. 12. The incorporation process was completed in August, 2004. Up until the vote, Dike and Drainage District No. 8 maintained a levee along Padilla Bay and several field ditches.

A notable feature within this District is the Padilla Bay Trail. This trail extends a distance of 2.2 miles along the top of the levee system adjacent to Padilla Bay. Skagit County Parks and Recreation Department and the Department of Ecology, as part of the Padilla Bay National Estuarine Research Reserve, maintain the trail.

5. Dike and Drainage District No. 12

As a result of incorporating Dike and Drainage District No. 8 in 2004, Dike District No. 12 renamed itself to Dike and Drainage District No. 12. This District historically owned and operated dikes, levees and outfalls along portions of the Skagit River and Padilla Bay. The drainage service area for Dike and Drainage District No. 12 is shown on **Figure 2-2.** The dike service area for Dike and Drainage District No. 12 is shown on **Figure 2-3.** The District's primary drainage channel is No Name Slough, which discharges to Padilla Bay through gravity tide gates and/or pump stations.

Specific responsibilities of Dike and Drainage District No. 12 are to maintain 1) the dike system along the north side of the Skagit River from the east end of the City of Burlington to the community of Avon and 2) levees along a portion of Padilla Bay and connecting sloughs. The purpose of these dike and levee systems is to protect properties from flood and seawater damage. Now, an additional responsibility is maintaining the stormwater drainage facilities within the No Name Slough basin.

6. Port of Skagit County

The Port of Skagit County (herein called the Port) owns approximately 1830 acres of property in the Bay View area. The boundaries for the Port are shown in **Figure 2-2.** The Port's primary purpose is to create jobs. In the Bay View area this is accomplished through two means: 1) to operate the Skagit Regional Airport; and 2) to develop light industry at the Bay View Business and Industrial Park. In the past, the Port has taken measures to reduce the impact of stormwater runoff from its property. Many of the Port's stormwater management objectives are presented in the *Port of Skagit County Stormwater Management Master Plan*, which is discussed later in this chapter.

7. City of Burlington

Most of the city limits of Burlington lie outside of the Study Area. There are currently ten homes along Peterson Road that contribute storm water drainage to Drainage District 19. A portion of Burlington's commercial area within the northern portion of the city does drain into the Maiben Ditch. The City and Drainage District No. 14 have a contractual arrangement where the City collect drainage utility fees from its commercial property owners in this area and transfers them on to Drainage District No. 14.

The City of Burlington currently provides sanitary sewer service to a portion of the Study Area. All commercial and light industrial developments within the Urban Growth Area will have sanitary sewer service. In addition, transportation impacts from development within the Study Area will effect transportation planning within the City; therefore, the City has an interest in development within its sewer service area. None of the existing or proposed stormwater drainage facilities within the Study Area impact stormwater facilities within the City of Burlington.

8. Large Tract Land Owners

There are several large tract landowners within the Study Area. It is anticipated that some of these large tract landowners will desire to someday develop their property. The designation of the Urban Growth Area will limit the development opportunities for those property owners outside of the UGA. Landowners inside the UGA will have more opportunities to develop their property and, therefore, will require increased attention to stormwater planning to accommodate anticipated growth.

9. Washington State Department of Ecology

The Washington State Department of Ecology is actively involved in the research and preservation of Padilla Bay through the Padilla Bay National Estuarine Research Reserve. The Reserve owns and manages the majority (11,000 acres) of Padilla Bay, including approximately 8,000 acres of eelgrass meadow. Padilla Bay is the receiving water for all stormwater drainage from the Study Area. The Reserve has been involved in drainage and stormwater quality issues of No Name Slough and Joe Leary Slough.

The Reserve also owns approximately 200 acres of land within the Study Area, primarily in and around the vicinity of the Padilla Demonstration Farm at the mouth of No Name Slough and the Breazeale-Padilla Bay Interpretive Center.

The Reserve's goals regarding stormwater management are to protect the natural resources of Padilla Bay and sustain agriculture on the adjacent flood plain by encouraging development and utility infrastructure that will facilitate proper stormwater controls.

10. Washington State Department of Fish and Wildlife

The Washington State Department of Fish and Wildlife is responsible for protecting and enhancing fish and wildlife habitats. Some storm drainage facilities, such as detention ponds and sloughs, can provide habitat for waterfowl. Some of the larger sloughs support various fish species. The Department of Fish and Wildlife is interested in the type and location of any new storm drainage facilities proposed by the *Bay View Watershed Stormwater Management Plan.* Specific objectives for the Washington State Department of Fish and Wildlife include maximizing and enhancing anadromous fish spawning and rearing habitat, reducing erosion and sedimentation, and minimizing impacts to wetlands.

11. Washington State Department of Transportation

The Washington State Department of Transportation [WSDOT] has jurisdiction over the design, operation and maintenance of State Route 20 [SR 20], which extends along the southern boundary of the Study Area. WSDOT is in the planning stages for widening SR 20 from the intersection with Memorial Highway [SR 536] to Interstate 5 in Burlington. The proposed widening will expand the highway from its current 2 lanes to four lanes. Stormwater mitigation measures for the proposed widening will need to be coordinated with proposed stormwater drainage improvements in this area.

12. Federal Aviation Administration

The Airport Planning Division of the Federal Aviation Administration [FAA] is responsible for providing guidance to airport operators regarding design and operation standards. With regards to stormwater management, the FAA has two concerns, both having to do with the location of detention ponds. First, the FAA has restrictions on what can be placed in the various flight path zones that are established around an airport. Second, the FAA also does not want any type of stormwater facility, such as a detention pond, to be located in an area that might attract waterfowl into the normal fight path of aircraft. The FAA is interested in the proposed location of future detention ponds around the Skagit Regional Airport. The FAA is opposed to the development or enhancement of wildlife habitat within 10,000 feet of runways. In addition, the FAA desires to minimize open water conditions within the Runway Protection Zone. To minimize open water conditions, the FAA requests that detention ponds be designed to drain completely within 48 hours after a storm event.

C. 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 area. The following are abstracts and summaries from these related documents.

1. Padilla Bay/Bay View Watershed Non-Point Action Plan[1]

In 1995 the Padilla Bay/Bay View Watershed Management Committee and Skagit County Department of Planning and Community Development prepared the *Padilla Bay/Bay View Watershed Non-Point Action Plan*. The committee included representatives of local residents, government agencies, environmental groups, members of the agricultural community, timber industry, Native American tribes, and other affected or interested parties. The mission of the Watershed Management Committee was to develop a Watershed Action Plan for the management of non-point source pollution in the Padilla Bay watershed as defined by the Washington State Administrative Code 400-12.

The Committee looked at several sources of potential contamination within the study area and recommended measures to control non-point pollution. The goal was to develop and implement a source control strategy for various non-point pollution sources. The Plan provides thirteen source control recommendations for stormwater drainage and erosion control. Some of these recommendations included modifications to existing county ordinances, the implementation of Best Management Practices [BMPs], and the restoration of existing drainage facilities that were contributing to pollution of Padilla Bay.

2. Port of Skagit County Stormwater Management Master Plan[2]

The *Stormwater Management Master Plan* was prepared for the Port of Skagit County by David Evans and Associates, Inc. and was completed in 1998. The Stormwater Management Master Plan is a comprehensive plan document that covers the entire area served by the Port of Skagit County, including the Bay View Business and Industrial Park and the Skagit Regional Airport.

The *Stormwater Management Master Plan* presents a review of existing stormwater facilities, including pipes, culverts, ditches, ponds, and channels. Capacities of existing and anticipated future stormwater conveyance facilities were evaluated using a hydraulic computer model. In addition to stormwater quantity calculations, stormwater quality characteristics are also addressed.

Based on the hydraulic analysis of existing stormwater facilities, water quality characteristics, and future developed conditions, a capital improvement plan was prepared which recommended specific stormwater capital improvements over the next few years. The primary emphasis of the capital improvement plan was to construct a series of regional detention facilities along Higgins Road and associated stormwater conveyance infrastructure.

[1] *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).

[2] *Stormwater Management Master Plan for the Bay View Business and Industrial Park and Skagit Regional Airport*, prepared by David Evens and Associates, Inc. (October 1998).

3. Hydrologic and Hydraulic Model of the No Name Slough Drainage[3]

The *Hydrologic and Hydraulic Model of the No Name Slough Drainage* was prepared for the Padilla Bay National Estuarine Research Reserve by Northwest Hydraulic Consultants in November 2000. The purpose of the study was to develop a hydraulic model to characterize the existing hydrology of the watershed and to allow future analysis of various land use scenarios and operational alternatives. A computer model was developed for the hydraulic modeling effort using the Hydrologic Simulation Program-Fortran (HSPF) model developed by the U.S. Environmental Protection Agency. The results of the modeling task provided some indication of the amount of runoff generated in the basin, discharge volumes to Padilla Bay, and frequency of flooding in the lower reaches near the levee and tide gates.

4. Bayview Ridge Subarea Plan[4]

Under the Growth Management Act, government entities are required to establish Urban Growth Areas [UGAs] and to set aside other areas as rural. Skagit County and the Port of Skagit County desire to establish an urban growth area around the Skagit Regional Airport. The UGA boundary and proposed land use is shown on **Figure 2-4.**

The purpose of this urban growth area is to develop a self-sufficient urban community to insure the continued viability of the Skagit Regional Airport. The *Bayview Ridge Subarea Plan*, currently being prepared by Reid-Middleton along with input from Skagit County, City of Burlington, and the Port of Skagit County, is intended to specifically address detailed community planning issues in the Bayview Ridge Urban Growth Area.

The *1997 Skagit County Comprehensive Plan* provides general guidelines for community development within Skagit County. The *Bayview Ridge Subarea Plan* coordinates and provides consistency with the *1997 Skagit County Comprehensive Plan* while providing detailed guidelines to facilitate future growth within the UGA.

The *Bayview Ridge Subarea Plan* documents in detail the critical planning characteristics within the area. These characteristics include the following:

- Land use
- Business and industrial development
- Commercial and community centers
- Housing
- Transportation elements
- Capital facilities elements
- Utilities elements

[3] *Padilla Bay Hydrology – Hydrologic and Hydraulic Model of the No Name Slough Drainage*, prepared by Northwest Hydraulic Consultants (November 2000).

[4] *Bayview Ridge Subarea Plan,* prepared by Reid-Middleton (March 2005 Draft).

- Parks, recreation, and open space
- Natural environment
- Essential public facilities

Two of the characteristics, specifically land use and the natural environment characteristics, will have a direct impact on stormwater drainage planning in this area.

5. Joe Leary Slough Drainage Study[5]

A letter report dated January 29, 2002, entitled *Joe Leary Slough, Maiben Road Ditch and South Spur Ditch Drainage Analysis and Findings* was prepared for Drainage District No. 14 by Semrau Engineering & Surveying. The letter report presents the findings from a study that 1) inventoried and surveyed drainage structures within the District's boundaries, 2) delineated and characterized the drainage subbasins, and 3) presented the results of a preliminary hydraulic model for the drainage basin.

The hydraulic model identified several deficiencies in the stormwater conveyance systems. The capacity of the Joe Leary Slough outfall is approximately 900 cfs at mean tide, but several of the upstream culverts are limited to approximately 330 cfs. Capacity restrictions are also present on the South Spur Ditch. Several recommendations were presented in the letter report and are summarized as follows:

- Berms at the Joe Leary Slough outfall to raise the stored water surface elevation and increase outfall capacity,
- Increase the conveyance capacity through upstream culverts in Joe Leary Slough,
- Provide additional storage at the Joe Leary Slough outfall,
- Provide additional storage along the South Spur ditch between Josh Wilson Road and Joe Leary Slough,
- Investigate if Bay View Ridge properties should mitigate for runoff volumes instead of peak discharge flow rates.

6. Tide Gate and Pump Station Study[6]

This Study, entitled *Inventory and Evaluation of Tide Gates and Pump Stations related to Alternatives #5 and #7 of the Skagit River Flood Damage Reduction Feasibility Study,* was performed in conjunction with the Skagit River Flood Protection/Salmon Restoration Project. The first draft was completed in November 2002. Skagit County and the U.S. Army Corps of Engineers selected two preferred alternatives for conveying the 100-year flood event in the Skagit River. One alternative, known as Alternative 5, is a proposal to set back the existing levees along the Skagit

[5] *Joe Leary Slough, Maiben Road Ditch and South Spur Ditch Drainage Analysis and Findings*, letter report prepared by Semrau Engineering & Surveying, PLLC (January 29, 2002).

[6] *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).

River. The concept behind Alternative 5 is that the 100-year flood event would then be contained within the river channel. The project would involve the setting back of levees from Burlington through Mount Vernon and downstream to the mouth of the North Fork and South Fork of the Skagit River.

Another alternative, known as Alternative 7, involves the construction of a 1600 to 2000-foot wide bypass channel that would be used to convey peak stormwater flows from the main river channel. This new channel would have a capacity to divert up to 80,000 cfs and would discharge into the Swinomish Channel instead of Skagit Bay.

Both proposed alternatives would greatly impact several storm drainage facilities within the Skagit Valley. The purpose of this study was fourfold:

- Provide an inventory, including location and condition, of existing tide gates, culverts, and pump stations within the project “footprint" of the two alternatives.
- Identify new and additional storm drainage facilities that may be required by either alternative.
- Identify those storm drainage facilities that may require modification and/or relocation.
- Identify the nature and condition of any potential habitat landward of the existing storm drainage facilities.

7. No Name Slough Watershed Characterization Report[7]

This report, completed in May 2004, was prepared by the Skagit Conservation District and the Padilla Bay National Estuarine Research Reserve. The objectives of the study were to 1) prepare a detailed characterization of existing hydrology and water quality, 2) provide public education and outreach, and 3) propose a comprehensive collection of projects to improve water quality, provide more consistent stream flows, and support fish and wildlife habitat. Proposed projects include wetland enhancement with stormwater storage components, conversion of ditches to bioswales, tree buffer installations, septic tank replacement, detention pond modifications, culvert replacement, and slough channel dredging.

[7] *No Name Slough Watershed Characterization,* prepared by the Skagit Conservation District and the Padilla Bay National Estuarine Research Reserve (May 2004).

Chapter 3
Study Area

The *Bay View Watershed Stormwater Management Plan* area (herein referred to as the Study Area) is primarily bound to the west by Padilla Bay, to the south and southeast by Big Indian Slough, and to the north and northeast by Joe Leary Slough and its tributaries. The Study Area is approximately 11,277 acres. **Figure 3-1** is an aerial photograph of the Bay View area.

Table 3-1 summarizes the land use designations within the Study Area.

Table 3-1: 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 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.

A. Land Use and Development

Existing development varies within the Study Area. **Figure 3-2** provides an indication where development has occurred. Prominent developments in the Bay View 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 cluster developments.

1. Historical Development

Some history regarding past development is presented below[8].

[8] *Bay View: Pioneer City on the Sound – An Oral History,* Don Eklund, Occasional Paper #22, Center for Pacific Northwest Studies, Western Washington University, 1987.

- The Bay View area was solely inhabited by native tribes until about the middle of the 19th century. The native inhabitants did not significantly modify the existing drainage systems, and likely had little impact on stormwater runoff and discharge.

- Some of Skagit County's earliest pioneers established homesteads in the Bay View area in the late 1840s. The rural village of Bay View was named by William Mckenna, who platted the original town site in 1884. The rural village of Fredonia was platted in 1890.

- Samuel Calhoun and Michael J. Sullivan are widely accepted to be the first whites to establish permanent settlements in (what is now) Skagit County in 1867. They are also thought to be the visionaries for constructing dikes around the salt flats, a process which allowed reclamation of the tidelands for growing crops. Dike construction changed the natural stormwater drainage. For example, Joe Leary Slough, prior to the construction of dikes, was a fish bearing stream large enough to raft logs.

- By 1871, reclaimed tidelands were producing barley, oats, hops and potatoes. The biggest challenge faced by the settlers that were farming the reclaimed tidelands was to keep the dikes from breaking, which was an ongoing problem. At the end of World War II, modern machinery allowed for the revamping and extending of dikes, broadening them at their base and building some to a height of eight to nine feet, as they are today.

- Bay View State Park overlooks Padilla Bay and offers picnic tables, a playground, and camping. The Skagit County Agricultural Association, with the understanding that it would become a State Park, donated the original portion of the Bay View State Park to Washington State in 1925. Additional parcels were acquired up through 1968. The park site was formerly a baseball field and racetrack.

- Development at the Skagit Regional Airport site began in 1933 with a small airport that was constructed by the Public Works Administration and the Works Progress Administration. The present runway and taxiway system was constructed in 1943 by the United States Navy as an alternative airfield for the Whidbey Islands Naval Air Station. The airfield was transferred to the Skagit Board of County Commissioners in 1958, and later transferred to the Port Districts of Anacortes and Skagit County. In 1975 the sole ownership of the airport property was transferred to the Port of Skagit County.

- Suburban type residential development occurred in the eastern portion of the Bay View Ridge area with the extension of sanitary sewer service from the City of Burlington, which started in the 1970s. Sanitary sewer service has steadily expanded since that time. The City of Burlington has recently completed a new sanitary sewage lift station near the intersection of Peterson Road and Avon-Allen Road with a new forcemain extending to its wastewater treatment facility.

- Padilla Bay's Breazeale Interpretive Center overlooks Padilla Bay. The property was obtained from the Breazeale family in 1973 and the Interpretive Center opened in 1982. The recent expansion was completed in 2005. The Interpretive Center overlooks Padilla Bay and provides interpretive exhibits, a lecture hall and research facilities. The old Breazeale family barn and house are now used as a laboratory with overnight quarters for visiting researchers and offices for staff.

- In 1989, the Skagit County Parks and Recreation Department and the Department of Ecology (Padilla Bay National Estuarine Research Reserve) began discussions with Dike and Drainage District No. 8 regarding developing a 2¼-mile dike trail along the southeastern shore of Padilla Bay. Planning and construction grants were obtained from the Aquatic Lands Enhancement Account (Department of Natural Resources), Skagit County Pathway Funds, and Ecology/NOAA Section 315 Funds. The Padilla Bay Trail was opened in 1990.

2. Skagit County Planning Efforts

The Skagit County Comprehensive Plan describes the general development patterns that are proposed within all areas of the county. A map showing the land use designations in the Bay View area is presented in **Figure 3-3.**

The *Bayview Ridge Subarea Plan* provides a detailed discussion regarding development within the proposed Urban Growth Area. The concept of the *Bayview Ridge Subarea Plan* supports the existing urban land use patterns. The overall intent of the *Bayview Ridge Subarea Plan* is to create a rural cohesive community which functions as a small city, providing for an urban level of development along with an urban level of services.

Future land use within the Bay View Ridge Subarea will build on the existing land use pattern and will include residential, commercial, business/industrial, and park/open space related uses. Land use prohibitions in and around the Skagit Regional Airport will limit some use options.

The highest concentration of residential development has occurred along the east side of the Study Area within the UGA, most of which occurred through large tract plats. There is still some potential for higher density residential plats within the UGA along Peterson Road east of the Skagit Regional Airport; however, approximately 70 percent of the residential zone areas are already developed. Due to constraints of the airport safety zones, future densities are limited to four dwelling units per acre. Lot sizes are to be between 8,400 and 10,890 square feet.

Outside of the UGA, residential development will be limited due to the rural designation. Proposed residential developments outside of the UGA will be required to be clustered so as not to preclude future urban development.

B. Natural Features

Prominent natural features include Padilla Bay, No Name, Joe Leary and Big Indian Sloughs, Bay View Ridge area, and the alluvial surrounding farmland.

Padilla Bay is an estuary at the saltwater edge of the large delta of the Skagit River. It is about eight miles long and three miles across.

Most of Padilla Bay's watershed (23,000 acres) is low flat delta that is now farmland. In the late 1800's, the marshes of the Skagit River delta were drained and levees were constructed. Portions of the Skagit River were diverted and are now confined to channels that empty into Skagit Bay leaving Padilla Bay "orphaned" from the river that formed its mud flats. Today, Padilla Bay's freshwater comes from a number of agricultural sloughs. The Swinomish Channel connects Padilla Bay to Skagit Bay located to the south. Padilla Bay is bordered on the east and south by levees that protect adjacent farmland from flooding. To the north and west are the rocky San Juan Islands in northern Puget Sound.

The surrounding alluvial farmland is within the floodplain of the Skagit Valley. Much of this area was reclaimed tidelands through the construction of dikes and drainage sloughs. For this reason, this area will be more susceptible to flooding. Development within the floodplain has been limited through development restrictions, zoning, and other farmland protection measures. Farming activities are expected to continue to dominate land use activities within the floodplains surrounding the Bay View Ridge area.

1. Topography

The Bay View Ridge area is situated east of Padilla Bay. This glacial terrace is elevated 220 feet above the surrounding floodplain. The physical features within the Bay View Ridge area range between gentle sloping terrain and steep hillsides. Undeveloped areas tend to have a mix of trees stands and opened fields or meadows.

2. Soil

The general classification for soils in the Bay View Ridge area are described as Bow-Coveland-Swinomish and are characterized by "moderately deep and very deep, somewhat poorly drained and moderately well drained, level to steep soils; on terraces, plains, and hills"[9]. The predominated soil classification in the area is Bow Gravelly Loam. The soils have a high percentage of fine-grained material, are typically saturated with poor percolation yields, and 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 presence of a perched water table between November and May.

3. Climate

Climate data for the Bay View area was derived from data published by the National Climate Data Center, which collects climate data from National Oceanic and Atmospheric Administration.

The average rainfall in the Bay View area 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 and less rainfall 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).

[9] Soil Survey of Skagit County Area, Washington, prepared by the United States Department of Agriculture – Soil Conservation Service in cooperation with the Washington State Department of Natural Resources and the Washington State University, Agriculture Research Center (September 1989).

C. Critical Areas

Critical areas include aquifer recharging areas, flood hazard areas, geologically hazardous areas, and fish and wildlife habitat conservation areas. Some of these critical areas, such as the wetlands within the Bay View Business and Industrial Park, have been delineated. However, most critical areas within the Bay View area have not been precisely identified and their exact locations are not accurately mapped. Skagit County, like many other jurisdictions, relies on critical area site assessments performed by development project proponents.

1. Aquifer Recharging Areas

The Bay View area does not contain any identified critical aquifer recharged areas. Development within aquifer recharge areas may reduce groundwater infiltration of stormwater. Some areas in the north portion of the Bay View area 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 area that may also rely on groundwater wells for their source of water.

2. Flood Hazard Areas

The Bay View area 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.

3. 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 area. The *Bayview Ridge Subarea Plan* provides a detailed discussion regarding wetlands in the Bay View area. Much of the discussion regarding wetlands presented in this Plan is derived directly from the *Bayview Ridge Subarea Plan*. The map showing the wetlands in the Bay View area is presented in **Figure 3-4.**

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.

Many wetlands and associated buffers have been identified in the Bay View area. Some wetlands in the area have been fragmented or isolated by existing development, while others have been hydrologically modified by uncontrolled or poorly controlled stormwater runoff. In some cases this has led to the support of primarily invasive and undesirable plants and animal species.

Within most of the Bay View area, 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 area outside of the Port ownership. The precise boundaries of these wetlands are not known and would be delineated by project proponents as specific development projects are proposed.

4. Port of Skagit County Wetlands Management Plan

The Port of Skagit County 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 property.

5. Priority Habitat

The Priority Habitats and Species (PHS) Program, administered by the Washington State Department of Fish and Wildlife, provide 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 3-5 shows the priority habitat within the Bay View Ridge area for the bald eagle and fish that has been established by the Washington State Department of Fish and Wildlife.

Chapter 4
Storm Drainage Facilities

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 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. For modeling purposes, the Indian Slough Basin was also divided into two separate basins, the Little Indian Basin and the Big Indian Basin. 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 units.

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** under Basin A**.**

Drainage District: Dike and Drainage District No. 12.

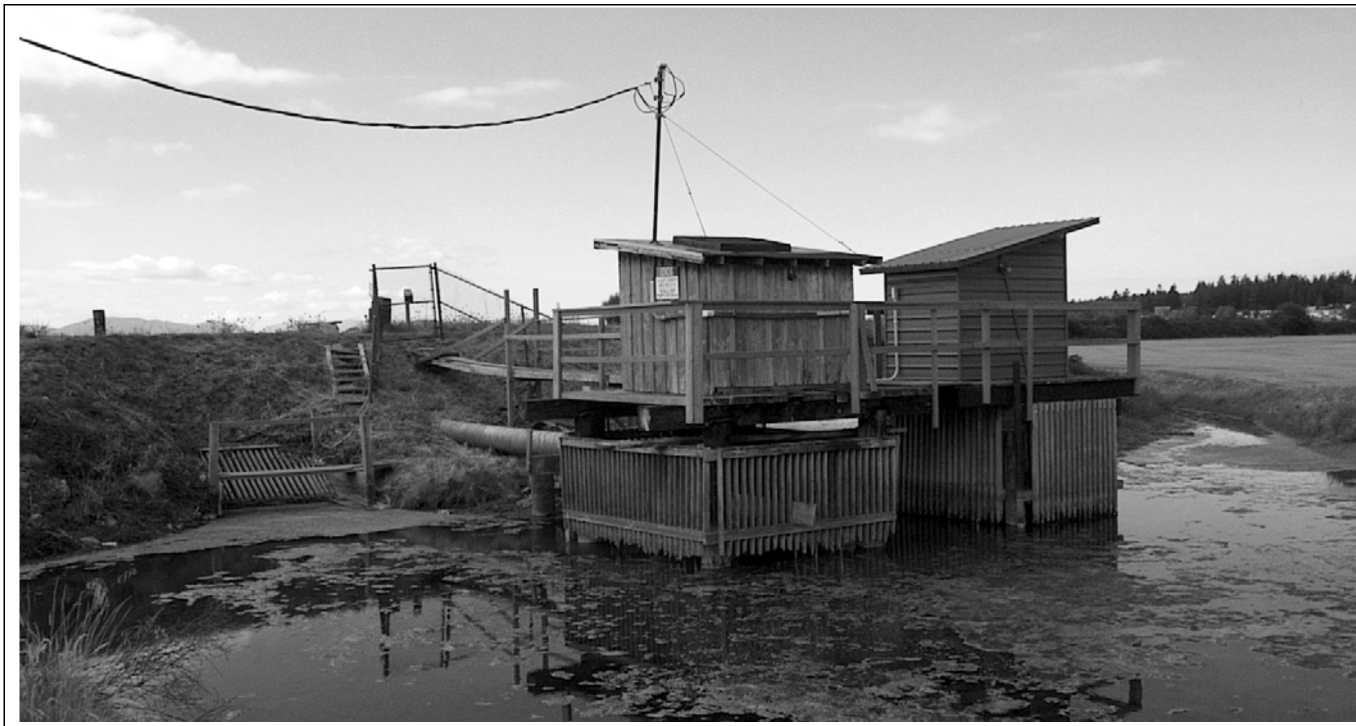
Primary Drainage Facility: No Name Slough.

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

Ponds and Detention Facilities: There are 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 is being 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.

Culverts and Pipes. Storm water conveyance is primarily through roadside ditches and culverts. There are some 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

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. One notable exception is a portion of the Bay View Ridge proposed UGA that 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** under Basin B**.**

Drainage District: Drainage District No. 14.

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

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

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 County 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 County 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 County 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 County 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 result from this effort are 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** under Basin C**.**

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.

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 County 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

Big Indian Slough Outfall and Pump Station

Chapter 5
Stormwater Quantity Analysis

As part of this Stormwater Management Plan, a hydraulic modeling study was completed to identify existing and potential drainage problems and to develop a list of capital improvement projects to address identified problems. This chapter summarizes the methods and results of the hydraulic modeling. **Chapter 7** 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 8**.

A. Modeling Methodology

1. Computer Hydraulic Model

The XPSWMM-2000 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 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 basins in the study area drain directly into Padilla Bay and were not included in the modeling of the four primary slough-based drainage systems.

2. Model Input

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 high water and mean 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.

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%

Existing effective impervious area [EIA] for each basin was determined using current aerial photographs; future EIA was estimated assuming full buildout conditions under Skagit County's current zoning coverage as of January 2003. The EIA for each zoning classification is shown in **Table 5-1.** **Figure 5-1** shows the main drainages and the subbasin boundaries in the Study Area. **Table 5-2** lists existing and future EIA for each modeled subbasin.

Table 5-2: Existing and Future Effective Impervious Area

Subbasin	Total Area	Existing EIA	Future EIA	Subbasin	Total Area	Existing EIA	Future EIA
Basin A: No Name Slough							
A-4	490 acres	4.0%	6.1%	A-11a	417 acres	6.0%	21.3%
A-5	306 acres	5.0%	5.2%	A-11b	636 acres	6.0%	6.2%
A-6	100 acres	5.0%	5.1%	A-11c	126 acres	4.0%	5.5%
A-7	325 acres	7.0%	28.1%	A-12	138 acres	6.0%	7.3%
A-8	127 acres	4.0%	6.0%				
Basin B: Joe Leary Slough							
B-1a	90 acres	5.0%	6.0%	B-6b	266 acres	5.0%	6.0%
B-1b	100 acres	4.0%	6.0%	B-6c	213 acres	4.0%	4.0%
B-1c	189 acres	6.0%	6.0%	B-6d	112 acres	5.0%	6.0%
B-1d	112 acres	5.0%	6.0%	B-7	930 acres	4.0%	5.0%
B-1e	108 acres	5.0%	5.0%	B-8a	768 acres	6.0%	11.8%
B-2	245 acres	4.0%	6.0%	B-8b	100 acres	6.0%	30.0%
B-3	500 acres	5.0%	6.0%	B-9	1,870 acres	5.0%	9.0%
B-4	150 acres	5.0%	6.0%	B-10	590 acres	4.0%	5.0%
B-5	90 acres	5.0%	6.0%	B-11	910 acres	5.0%	5.0%
B-6a	305 acres	5.0%	5.0%	B-12	2,630 acres	6.0%	7.0%
Basin C: Little Indian Slough							
C-1a	54 acres	5.0%	5.0%	C-1c	156 acres	5.0%	5.0%
C-1b	218 acres	5.0%	5.0%	C-2	166 acres	15.0%	35.0%
Basin C: Big Indian Slough							
C-2a	135 acres	8.0%	35.0%	C-5	133 acres	4.0%	35.0%
C-3a	363 acres	5.0%	19.0%	C-6	116 acres	8.0%	27.0%
C-3b	220 acres	5.0%	5.0%	C-7	1,647 acres	18.0%	35.0%
C-4	422 acres	4.0%	24.1%	C-8	2,018 acres	8.0%	12.0%

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.

3. Model Basin Descriptions

The following sections describe each basin and the elements included in the models for each.

a. No Name Slough Modeling Basin

Located on the west side of the Study Area, No Name Slough modeling basin drains approximately 2,600 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. **Figure 5-2** shows the modeled elements in the No Name Slough Basin.

b. Joe Leary Slough Modeling Basin

The Joe Leary Slough modeling basin is the most northern basin in the study area and covers about 10,400 acres. This basin was subdivided into 19 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, just 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.

Figure 5-3 shows the modeled elements in the Joe Leary Slough Basin.

c. Little Indian Slough Modeling Basin

The Little Indian Slough modeling basin lies between No Name Slough and Big Indian Slough. This is the smallest of the modeled drainages with a basin area of approximately 600 acres. This basin was subdivided into 4 subbasins for the hydrologic modeling. The topography in Little Indian Slough is mostly flat, although there is some elevation gain in the upper portion of the basin.

Little Indian Slough was modeled from its outlet at Padilla Bay to beyond Farm-to-Market Road. Key culverts at Bay View-Edison Road and Farm-to-Market Road were included in the SWMM model, as well as a culvert crossing on a minor road to the east of Farm-to-Market Road. **Figure 5-4** shows the modeled elements in the Little Indian Slough Basin.

d. Big Indian Slough Modeling Basin

The Big Indian Slough modeling basin is in the southernmost part of the study site and has a drainage area of about 5,000 acres. The topography in most of the basin is flat; the northern part of the basin is part of Bay View Ridge and has steeper slopes. This basin was subdivided into 8 subbasins for the hydrologic modeling.

Big Indian Slough was modeled from its outlet at Padilla Bay to the crossing of SR 20 upstream of Higgins Airport Way. The model includes the key bridges and culverts in this portion for the drainage system. At higher water surface elevations, flow can escape from Big Indian Slough near SR 536 and flow into Higgins Slough. This overflow was included in the model in order to quantify the effects on Big Indian Slough and the possible impacts on Higgins Slough. **Figure 5-5** shows the modeled elements in the Big Indian Slough basin.

B. Modeling Results

Hydraulic modeling was completed for each of the four main drainages in the Bay View watershed: No Name Slough, Joe Leary Slough, Little Indian Slough, and Big Indian Slough.

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 5-3**. **Table 5-4** compares the existing and future peak runoff rates for the 24-hour storm event in the subbasins.

The existing condition peak flows calculated by the SWMM model were compared to the existing conditions peak flows reported in Northwest Hydraulic Consultant's (NHC) *Hydrologic and Hydraulic Model of the No Name Slough Drainage* (November 2000). **Table 5-5** shows a comparison of the flows at three locations for existing conditions. In general, the estimated peak flows calculated by the SWMM model were higher than the peak flows reported in the NHC study. The NHC study did not compute peak flows for future conditions.

The modeling indicated flooding at locations throughout the basin. The flooding is indicated at the 10-, 25-, and 100-year recurrence interval for both existing and future land use conditions. **Table 5-6** shows predicted flooding locations with no drainage improvements implemented.

Table 5-3: Existing and Future Conditions Peak Flows for No Name Slough

SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-10	Outlet of Slough (before pumps)	111 cfs	152 cfs	201 cfs	139 cfs	180 cfs	228 cfs
NN-85	Confluence of Tributaries	95 cfs	130 cfs	178 cfs	120 cfs	154 cfs	200 cfs
NN-130	Marihugh Road	32 cfs	42 cfs	52 cfs	32 cfs	42 cfs	52 cfs

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

Table 5-4: Existing and Future Conditions Peak Runoff for No Name Slough						
	Existing Conditions Peak Runoff			Future Conditions Peak Runoff		
Subbasin	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
A-4	10 cfs	12 cfs	16 cfs	12 cfs	16 cfs	22 cfs
A-5	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
A-7	27 cfs	35 cfs	47 cfs	37 cfs	48 cfs	64 cfs
A-8	10 cfs	13 cfs	18 cfs	11 cfs	14 cfs	20 cfs
A-11a	23 cfs	31 cfs	43 cfs	29 cfs	38 cfs	52 cfs
A-11b	33 cfs	44 cfs	60 cfs	50 cfs	67 cfs	90 cfs
A-11c	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
See **Figure 5-1** for subbasin locations.						

Table 5-5: Comparison of Existing Condition Peak Flow from SWMM and NHC Study						
	SWMM Existing Conditions Peak Flows for No Name Slough			NHC Exiting Conditions Peak Flows for No Name Slough		
Approximate Location	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Outlet of Slough	111 cfs	152 cfs	201 cfs	91 cfs	115 cfs	154 cfs
Confluence of Tributaries	95 cfs	130 cfs	178 cfs	81 cfs	101 cfs	132 cfs
Marihugh Road	32 cfs	42 cfs	52 cfs	12 cfs	16 cfs	23 cfs

Table 5-6: No Name Slough Flooding Locations with No Improvements							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows [cfs]			Future Conditions Peak Flows [cfs]		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Upstream of Outlet	●	●	●	●	●	●
NN-30	Upstream of Slough Outlet	●	●	●	●	●	●
NN-40	Downstream of Bay View-Edison Road	●	●	●	●	●	●
NN-50	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-60	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-65	Slough Upstream of Bay View-Edison Rd	●	●	●	●	●	●
NN-70	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-80	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-140	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-150	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-160	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-170	S, Stem Near Dahlstadt Farm	●	●	●	●	●	●
See **Figure 5-2** for node locations.				● denotes predicted flooding for the storm event			

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 existing and future peak runoff rates for the 24-hour storm event in the subbasins. Flooding locations are listed in **Table 5-9.**

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 the confluence with Maiben Ditch.

Table 5-7: Existing and Future Conditions Peak Flows for Joe Leary Slough						
	Existing Conditions Peak Flows			Future Conditions Peak Flows		
Approximate Locations	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Bay View-Edison Road	326 cfs	395 cfs	495 cfs	344 cfs	418 cfs	525 cfs
Farm-to-Market Road	185 cfs	240 cfs	325 cfs	202 cfs	262 cfs	352 cfs
Allen West Road	165 cfs	215 cfs	295 cfs	183 cfs	238 cfs	322 cfs
Downstream of Confluence	145 cfs	195 cfs	270 cfs	154 cfs	205 cfs	281 cfs
South Spur Ditch at Josh Wilson Road	52 cfs	68 cfs	92 cfs	57 cfs	74 cfs	98 cfs
Maiben Ditch at Thomas Road	76 cfs	104 cfs	146 cfs	78 cfs	106 cfs	149 cfs

Table 5-8: Existing and Future Conditions Peak Runoff for Joe Leary Slough						
	Existing Conditions Peak Runoff			Future Conditions Peak Runoff		
Subbasin	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
B-1a	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
B-1c	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
B-1e	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs
B-2	19 cfs	25 cfs	35 cfs	19 cfs	25 cfs	35 cfs
B-3	34 cfs	45 cfs	62 cfs	35 cfs	46 cfs	63 cfs
B-4	18 cfs	24 cfs	33 cfs	18 cfs	24 cfs	33 cfs
B-5	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
B-6b	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
B-6d	27 cfs	35 cfs	46 cfs	27 cfs	35 cfs	46 cfs
B-7	22 cfs	30 cfs	42 cfs	25 cfs	33 cfs	44 cfs
B-8	38 cfs	50 cfs	68 cfs	68 cfs	89 cfs	118 cfs
B-9	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
B-11	21 cfs	27 cfs	36 cfs	21 cfs	27 cfs	36 cfs
B-12	64 cfs	82 cfs	110 cfs	70 cfs	91 cfs	121 cfs
See **Figure 5-1** for subbasin locations.						

Table 5-9: Joe Leary Slough Flooding Locations with No Improvements						
SWMM Model Node	Existing Conditions Peak Flows			Future Conditions Peak Flows		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
JL-20	●	●	●	●	●	●
JL-24	●	●	●	●	●	●
JL-25	●	●	●	●	●	●
JL-26	●	●	●	●	●	●
JL-30	●	●	●	●	●	●
JL-32	●	●	●	●	●	●
JL-34	●	●	●	●	●	●
JL-36	●	●	●	●	●	●
JL-40	●	●	●	●	●	●
JL-44	●	●	●	●	●	●
JL-46	●	●	●	●	●	●
JL-48	●	●	●	●	●	●
JL-50		●	●	●	●	●
JL-52	●	●	●	●	●	●
JL-54	●	●	●	●	●	●
JL-55		●	●		●	●
JL-60		●	●		●	●
JL-64		●	●	●	●	●
JL-71		●	●	●	●	●
JL-72	●	●	●	●	●	●
JL-84		●	●		●	●
JL-90		●	●		●	●
JL-91		●	●		●	●
JL-92		●	●		●	●
JL-100	●	●	●	●	●	●
JL-105	●	●	●	●	●	●
JL-110		●	●		●	●
JL-111			●			●

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

3. Little Indian Slough

Predicted peak flows in Little Indian Slough for the 10-, 25-, and 100-year storm events are shown in **Table 5-10.** **Table 5-11** compares the existing and future peak runoff rates for the 24-hour storm event in the subbasins. Flooding in Little Indian Slough is not expected to increase much under future conditions since it is not likely that there will be a large increase in impervious area at full buildout. The upstream subbasin (C-2) that drains into the slough is part of the Urban Growth Area (UGA), and some development there will increase the EIA in the basin from 15 to 35 percent.

Some flooding problems indicated by the modeling in the Little Indian Slough Basin are listed in **Table 5-12.** The flooding sites are in the ditch downstream of Farm-to-Market Road (modeling Node LI-60) and in a culvert under a private drive east of Farm-to-Market Road (modeling Node LI-80). According to the model, both locations experience flooding at the 10-, 25-, and 100-year recurrence intervals for future conditions, and the culvert at McFarland Road also experiences flooding for the 100-year recurrence interval for existing conditions.

The model does not predict roadway overtopping at the culvert at Farm-to-Market Road, due to the height of the road embankment; however, the culvert there appears to be undersized due to the relatively high water surface elevation and the high predicted flow velocity (12.3 feet per second) in the culvert for the 100-year future conditions rainfall event. The high water surface elevation at Farm to Market Road also may be contributing to flooding upstream.

Table 5-10: Existing and Future Conditions Peak Flows for Little Indian Slough

SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-10	Outlet of Slough	83 cfs	100 cfs	121 cfs	86 cfs	103 cfs	127 cfs
LI-32	Between Bay View-Edison Road & Farm to Market Road	16 cfs	23 cfs	35 cfs	25 cfs	35 cfs	50 cfs
LI-60	Farm to Market Road	13 cfs	18 cfs	26 cfs	23 cfs	30 cfs	41 cfs

See **Figure 5-4** for node locations.

Table 5-11: Existing and Future Conditions Peak Runoff for Little Indian Slough

Subbasin	Existing Conditions Runoff			Future Conditions Runoff		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
C-1a	4 cfs	7 cfs	11 cfs	4 cfs	7 cfs	11 cfs
C-1b	8 cfs	12 cfs	20 cfs	8 cfs	12 cfs	20 cfs
C-1c	6 cfs	11 cfs	17 cfs	6 cfs	11 cfs	17 cfs
C-2	14 cfs	19 cfs	27 cfs	23 cfs	31 cfs	43 cfs

See **Figure 5-1** for subbasin locations.

Table 5-12: Little Indian Slough Flooding Locations with No Improvements							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-20	Upstream of Bay View-Edison Road		●	●		●	●
LI-22	Lower Slough		●	●		●	●
LI-24	Lower Slough		●	●		●	●
LI-26	Lower Slough		●	●		●	●
LI-32	Middle Slough			●			●
LI-60	Upstream End of Culvert LI-C-1	●	●	●	●	●	●
LI-80	Downstream End of Culvert LI-C-2	●	●	●	●	●	●
LI-90	Upstream End of Culvert LI-C-2	●	●	●	●	●	●

See **Figure 5-4** for node locations. ● denotes predicted flooding for the storm event

4. Big Indian Slough

Predicted peak flows in Big Indian Slough for the 10-, 25-, and 100-year storm events are listed in **Table 5-13.** **Table 5-14** compares the existing and future peak runoff rates for the 24-hour storm event in the subbasins. Six flooding areas were identified in the Big Indian Slough Basin, concentrated around Higgins Airport Way, where runoff from the UGA (including the Skagit Regional Airport) enters the slough. According to the model, the slough is unable to handle the high flows at the confluence; the backwater from this constriction propagates upstream, causing additional flooding. The flooding locations and frequency are listed in **Table 5-15**.

The model indicates the most severe flooding at the culverts upstream and downstream of Higgins Airport Way (model Nodes BI-210, BI-240, and BI-250). Flooding at these locations is predicted at the 10-, 25-, and 100-year recurrence interval for both existing and future conditions. Flooding at the 100-year recurrence interval also is predicted at the Higgins Airport Way culvert (Node BI-230) and at the culvert upstream of SR 20 (Nodes BI-180 and BI-190) for future and existing conditions.

The hydraulic model did not predict flooding downstream of the SR 20 culvert. However, it is well documented that flooding of SR 20 west of Farm-to-Market Road occurred during the November 1990 storm event. The channel appears to have sufficient capacity in this part of the slough for both existing and future conditions. There is relatively little additional tributary area to the slough downstream of the SR 20 culvert, but the channel capacity increases significantly. Reasons for the SR 20 flooding may be from one or a combination of: 1) channel blockage, 2) tidal influence, 3) localized poor drainage, and 4) model calibration.

An overflow in the vicinity of SR 536 provides some relief during peak storm events, as water is diverted from Big Indian Slough south into Higgins Slough. The amount of overflow is presented in **Table 5-16** for each storm event. The model indicates that if the overflow were kept within Big Indian Slough, the downstream portion of the channel would still have adequate capacity for existing and future flow conditions based on the modeling assumptions. However, observed flooding of SR 20 indicates that some hydrologic condition exist that can overwhelm the outfall and lower reach of Big Indian Slough.

Table 5-13: Existing and Future Conditions Peak Flows for Big Indian Slough

SWMM Model Node	Approximate Location	Existing Conditions Peak Flows 10-Year	Existing Conditions Peak Flows 25-Year	Existing Conditions Peak Flows 100-Year	Future Conditions Peak Flows 10-Year	Future Conditions Peak Flows 25-Year	Future Conditions Peak Flows 100-Year
BI-10	Outlet to Padilla Bay	311 cfs	352 cfs	403 cfs	404 cfs	564 cfs	631 cfs
BI-160	Downstream of SR 20	151 cfs	205 cfs	286 cfs	170 cfs	265 cfs	337 cfs
BI-230	Higgins Airport Way	53 cfs	72 cfs	104 cfs	84 cfs	87 cfs	131 cfs
BI-270	Above SR 20	49 cfs	63 cfs	85 cfs	54 cfs	69 cfs	93 cfs
See **Figure 5-5** for node locations.							

Table 5-14: Existing and Future Conditions Peak Runoff for Big Indian Slough

Subbasin	Existing Conditions Peak Runoff 10-Year	Existing Conditions Peak Runoff 25-Year	Existing Conditions Peak Runoff 100-Year	Future Conditions Peak Runoff 10-Year	Future Conditions Peak Runoff 25-Year	Future Conditions Peak Runoff 100-Year
C-2a	9 cfs	15 cfs	22 cfs	28 cfs	32 cfs	50 cfs
C-3a	16 cfs	23 cfs	35 cfs	40 cfs	53 cfs	72 cfs
C-3b	8 cfs	11 cfs	20 cfs	8 cfs	12 cfs	16 cfs
C-4	17 cfs	26 cfs	41 cfs	55 cfs	72 cfs	98 cfs
C-5	6 cfs	13 cfs	17 cfs	25 cfs	33 cfs	45 cfs
C-6	15 cfs	22 cfs	34 cfs	25 cfs	36 cfs	50 cfs
C-7	110 cfs	175 cfs	220 cfs	110 cfs	175 cfs	220 cfs
C-8	47 cfs	60 cfs	82 cfs	60 cfs	76 cfs	106 cfs
See **Figure 5-1** for subbasin locations.						

Table 5-15: Big Indian Slough Flooding Locations with No Improvements							
SWMM		Existing Conditions Peak Flows			Future Conditions Peak Flows		
Model Node	Approximate Location	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-180	Culvert Upstream of SR 20			●			●
BI-190	Culvert Upstream of SR 20			●			●
BI-210	Culvert Downstream of Higgins Airport Way	●	●	●	●	●	●
BI-230	Culvert at Higgins Airport Way			●			●
BI-240	Culvert Upstream of Higgins Airport Way	●	●	●	●	●	●
BI-250	Culvert Upstream of Higgins Airport Way	●	●	●	●	●	●

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

Table 5-16: Peak Overflow Rates from Big Indian Slough to Higgins Slough			
	Peak Flow to Higgins Slough		
Condition	10-Year	25-Year	100-Year
Existing Conditions	1 cfs	4 cfs	14 cfs
Future Conditions	7 cfs	16 cfs	35 cfs

Chapter 6
Stormwater Quality and Treatment

The purpose of the Bay View Watershed Stormwater Management Plan is to quantify the quantity of stormwater runoff within the Study Area in order to analyze 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 quality control is only cursorily addressed in this report. 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*[10] (Stormwater Manual) as Skagit County has not yet adopted the Department of Ecology's 2001 update to the Stormwater Manual.

The *Padilla Bay/Bay View Watershed Nonpoint Action Plan*[11] (Nonpoint Action Plan) is the most significant work to date regarding stormwater pollution in the Bay View area. 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.

A. Bay View Area Stormwater Quality

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 issued by Ecology. Ecology is expected to issue a NPDES General Permit for Municipal Storm Sewers (Phase II) in late 2006 or early 2007. This

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

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

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 Subarea will be subject to these augmented regulations.

B. Contamination Sources and Management Strategies

There are several potential sources of contamination for stormwater runoff. Below is a brief discussion of some of the obvious and abundant sources of stormwater contamination within the Study Area, followed by a brief discussion of stormwater management strategies for each potential contamination source. The stormwater treatment strategy for the Bay View area is based on recommendations presented in the Nonpoint Action Plan and recommended best management practices [BMPs] presented in the 2001 Stormwater Manual[12].

1. Pavement Runoff and Roadside Ditches

Roadside ditches serve a majority of the roadway system within the Study Area. Only recent residential plats have curbs, gutters, and catch basins. Common stormwater pollutants associated with direct stormwater input into roadside ditches include sediment, hydrocarbons, organic and inorganic particulates, and heavy metals. To minimize pollutant impacts, roadside ditches should be maintained to preserve their condition and design capacity while minimizing bare or thin vegetated surfaces.

Volume IV, Chapter 2 of the 2001 Stormwater Manual provides the BMPs for maintenance of roadside ditches. The Nonpoint Action Plan also has several recommendations for mitigating stormwater runoff quality from pavement and roadside ditches.

2. Septic Tanks

Sanitary sewers currently serve only the southeastern portion of the Study Area. The areas served by sanitary sewers are the commercial areas within and adjacent to the Port of Skagit County and medium density residential developments in the southeast quadrant of the Study Area. The remaining development within the Study Area is served by individual septic tanks.

Septic tanks are a principal means of wastewater treatment and disposal for rural and suburban areas. Septic tanks can be an effective means of wastewater treatment and disposal when properly designed, installed and maintained. However, improperly design, installed and/or maintained septic tanks and cesspools, for both human and animal wastes, can be a major source of ground water and surface water pollution. Individual pollution potential from septic tanks and/or cesspools may be of little significance, but the aggregate impact can be detrimental in specific areas. The principal contaminants from septic tanks are nutrients, fecal coliform, and other biological contaminants, but small quantities of household chemicals can also be a problem. The 303(d) listing of Big Indian Slough, Joe Leary Slough, and No Name Slough for fecal coliform provides supporting evidence of this problem within the Study Area.

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

The *Soil Survey of Skagit County Area, Washington* rates the soils in the Study Area as "severe" for septic tank absorption fields due to wetness and slow percolation characteristics. Soils of this type could be a factor in potential degradation of ground and surface water quality in the Study Area.

The Nonpoint Action Plan recommends several steps that can be taken to help reduce water quality degradation form septic tanks, including:

- Institute public education programs to encourage property owners to actively maintain their septic systems.
- Ensure regular septic system maintenance.
- Promote water conservation measures to improve performance and extend septic system life.
- Provide access to septage disposal facilities.
- Consider using recent advances in septic system technology is areas where conventional systems are inappropriate.
- Provide strong enforcement of septic system maintenance and prompt response to known problems.
- Require sanitary sewer service be provided in newly developed areas within the UGA.

3. Agricultural Activities

Agriculture is a predominant industry in the Bay View area. Agricultural activities include both crop production and livestock operations. Agricultural chemicals and contaminants can contribute sediment, fecal coliform, nutrients, pesticides, fungicides and herbicides to stormwater. *Washington's Nonpoint Source Management Plan*[13], Chapter 5, offers BMPs for agricultural activities. Skagit County's recently adopted Agricultural Critical Areas Ordance requires that agricultural operators 'do not harm' critical areas. Do not harm is defined as:

- Meeting the water quality standards required by RCW 90.48 (Water Pollution Control Act) and WAC 173-201A,
- Meeting the requirements of any Total Maximum daily load (TMDL) requirements established by the Department of Ecology,
- Meeting all applicable requirements of RCW 77.55 (Hydraulics Code) and WAC 220-110, and
- Meeting specific agricultural practice standards as defined in the ordinance.

The Padilla Bay Demonstration Farm, located in the Study Area, is a full-scale crop farming operation that is used for investigating and demonstrating the application of agricultural BMPs. It is

[13] *Washington's Nonpoint Source Management Plan,* prepared by the Washington State Department of Ecology (April 2000).

a cooperative effort by the Skagit Valley farmers, Skagit Conservation District, Washington State University, and Padilla Bay Staff. Agricultural BMP's implemented on the demonstration farm that are shown to have significant water quality benefits could be implemented throughout the Study Area to help in reducing non-point source pollution.

The Nonpoint Action Plan recommends several BMPs for mitigating the water quality impacts of agricultural activities. These BMPs include erosion and sedimentation controls, management of runoff from confined animal facilities, nutrient and pesticide management measures, and grazing management practices.

C. Stormwater Treatment Techniques

1. Stormwater Ponds and Bioswales

There are two general types of stormwater ponds, dry ponds and wet ponds. There are also several types of hybrid ponds that utilize a combination of dry and wet pond characteristics.

A dry pond primarily provides temporary stormwater detention by holding stormwater and releasing it at a controlled rate over a period of time. Most of the time the pond is dry and there is very little stormwater treatment. The primary purpose is to reduce the peak stormwater runoff rate and reduce downstream erosion impacts. Water quality benefits, if any, result from settling of suspended solids and attached pollutants and absorption onto soils. Dissolved pollutants are most likely not removed.

A wet pond contains a permanent pool of water and provides both stormwater detention and treatment. Water within a wet pond may dry up during the dry season. Water quality benefits result from settling of suspended solids and attached pollutants, absorption onto soils, and transformation and uptake by bacteria and algae.

Bioswales are shallow grass-lined channels that stormwater runoff passes through. Water quality benefits result from settling of suspended solids and attached pollutants, absorption onto soils, and uptake by grass roots. Bioswales are often used in conjunction with dry ponds to provide stormwater treatment. Bioswales are most often used in small-scale developments, typically sloped to drain, and do not hold water.

From a maintenance standpoint, fewer, larger ponds are more advantageous than numerous smaller ponds.

2. Wetlands

Wetlands, both natural and constructed, have been demonstrated to provide good stormwater treatment. Water quality benefits from wetlands result from settling of suspended solids and attached pollutants, absorption into soils, and transformation and uptake by bacteria, algae and vegetation roots. Wetlands also provide wildlife habitat and are typically more aesthetically pleasing when compared to ponds.

Recognizing the valuable contribution of wetlands, both natural and constructed, their protection is extremely important. In addition, it is important to preserve the natural balance in a wetland. Any disruption of a wetland, both directly to the wetland and/or indirectly to the contributing drainage area, could alter its biological balance. When the biological balance it altered, the wetland's effectiveness for stormwater treatment could diminish.

D. West Nile Virus

Within the past few years, wetlands and detention ponds have been scrutinized for their possible contribution as a breeding ground for mosquitoes and the spread of the West Nile Virus. Though not yet prevalent in the Pacific Northwest, the West Nile Virus has been spreading at an alarming rate. Currently, it is thought the mosquitoes that are responsible for transmitting the West Nile Virus, such as the *Culex* species, are not common in wetlands. Research into this disease is in its infancy, however, some agencies, such as the US Environmental Protection Agency[14] and others[15] have publics some initial findings.

Wetlands, both natural and constructed, and detention ponds have a potential to provide a breeding ground for mosquitoes. Common characteristics include shallow water depths (less than 1 meter), dense aquatic vegetation, and stagnant water during summer conditions. It is thought that a healthy wetland can reduce the potential for mosquito breeding, but not eliminate it. It is sometimes difficult to maintain a healthy wetland in an urban environment.

Some design and maintenance measures to achieve a healthy wetland or detention pond include the following:

- For wet ponds, maintain a minimum depth of 1 meter and construct steep side slopes. This will limit the amount of area that can be used as mosquito breeding habitat.
- Design dry pond to drain completely within 72 hours.
- Maintain a constant supply of fresh water to the wetlands and wet pond to diminish stagnation.
- Aerate the wet pond to increase the concentration of dissolved oxygen and diminish stagnation.
- Drain or pump out flow control structures during the spring and summer mosquito breading period.
- Submerge inlet and outlet pipe to reduce surface area from mosquitoes to lay eggs.
- Control the growth and density of pond-edge vegetation that would inhibit mosquito predators. Also, adult mosquitoes are attracted to the dense vegetation near the waters edge to lay their eggs. Impermeable liners may be used to control pond-edge vegetation.

[14] *Do Stormwater Retention Ponds Contribute to Mosquito Problems?,* Nonpoint Source News-Notes, US Environmental Protection Agency, Issue No. 71, May 2003.

[15] *Stormwater Management Could Combat West Nile Virus,* R. Dale Downey, PE, Cumming Cockburn Limited, September 2003.

In order to achieve these goals, a responsible entity, such as Skagit County or drainage districts, needs to understand the importance of routine maintenance to maximize the stormwater treatment potential of detention ponds and to minimize the potential for developing mosquito breeding habitat.

Chapter 7
Storm Drainage Alternatives Analysis

Several conceptual alternatives for relieving flooding in each basin are proposed and evaluated below. This is not considered to be a comprehensive list; conceptual alternatives were selected for evaluation based on their probability of correcting flooding problems. Input from stakeholders provided the design basis for some of the evaluated alternatives. Other alternatives were evaluated based on specific requests by stakeholders. Additional alternatives or variations on these alternatives may become viable solutions in solving flooding problems. The optimal solution will most likely require a combination of the alternatives.

A. Storm Drainage Structures

There are several types of hydraulic structures that are capable of conveying or controlling stormwater. These hydraulic structures include, but are not limited to, open channels, conduits, and pump stations.

1. Open Channels

Open channels are conduits with a free water surface that is exposed to the atmosphere. They are the most common type of stormwater drainage structures used and include sloughs and ditches, free flowing culverts and drainage pipes, and detention ponds. Culverts, when not completely submerged, act as open channels.

a. Sloughs and Ditches

Ditches are used extensively in rural areas to convey stormwater. Ditches can be constructed below the existing ground level or elevated above the existing ground through the construction of levees. Sloughs are typically large drainage channels found in sub-tidal areas. Sloughs sometimes follow historic river channels, while ditches are often manmade. In some cases, stormwater can be temporarily stored and/or treated within sloughs and ditches. Also in some cases, sloughs and ditches can be enhanced and managed for wildlife habitat.

Sloughs and ditches are an effective means of conveying large quantities of stormwater. Design consideration must be given to the location and size in order for them to be effective. There can be no hydraulic barrier that restricts the free flow of water. Over topping of sloughs and ditches during large storm events can have damaging results. Other hydraulic structures, such and culverts and tide gates, are often incorporated into a slough or ditch system.

b. Culverts and Pipes

Free flowing culverts and pipes are often used when it is desirable to place structures, such as roadways, over the stormwater conveyance system. Culverts and pipes can also be used in slough and ditch systems when a hydraulic barrier is encountered, such as tide-influenced outfalls. Culverts and pipes are not as effective and efficient as sloughs and ditches in conveying large quantities of stormwater runoff.

c. Detention Ponds

Detention ponds are used in stormwater conveyance to reduce the peak runoff rate during a storm event. This is accomplished by temporarily detaining stormwater runoff at or near the source and releasing is to the downstream conveyance system at a reduced rate. Detention ponds can also be designed to incorporate stormwater treatment measures such as sedimentation and nutrient absorption.

Detention ponds allow a new downstream conveyance system to be smaller or reduce existing capacity problems on existing downstream conveyance systems. The tradeoff is that detention ponds require available land for construction of the facility and ongoing maintenance to ensure their effective operation.

2. Conduits

Conduits are pressurized pipelines where the free water surface is almost never within the conduit itself. The most common conduit is stormwater conveyance is the discharge forcemain of a pump station. During storm events, culvert and gravity flow pipelines can experience surcharging, resulting in a temporary pressurized condition. During high tides, tide gates also act as pressurized conduits.

3. Pump Stations

Pump stations are used to overcome a hydraulic barrier. Typical hydraulic barriers in stormwater drainage include tide-influenced outfalls and terrain barriers. A pump station at a tide-influenced outfall provides the opportunity to discharge stormwater during periods of high tide when tide gates are closed. A pump station could also be employed to effectively drain stormwater from a low lying area.

Two pump options to consider are screw pumps and centrifugal pumps. Either option can effectively convey stormwater but they present different design and operational considerations.

a. Screw Pumps

Screw pumps are an efficient means of lifting large quantities of water at low heads and are ideal as a drainage pump in low-lying areas such as reclaimed land areas. These pumps have a flow range of 0.2 cfs to 200 cfs and can provide lift of 3 to 30 feet.

The operation of a screw pump is like that of a moving bucket conveyor. The volume of the "bucket" is formed between two flights on the screw with a trough acting as the bottom and sides. Since the screw is on an incline there is always a void space at the top of each bucket. Screw pumps offer the following advantages:

- They offer variable pump capacity while operating at a constant speed.

- They have high operating efficiencies over a greater range than other pumps.
- They can handle large objects so the pumps do not clog.
- They are fish friendly.
- They require minimal maintenance and upkeep.
- They do not require a wet well, piping or a pump house.

The main disadvantages of a screw pump are that the lift elevation is limited and that discharge from a screw pump is to atmosphere and cannot be delivered to a pressurized discharge location. These issues are not a concern for the expected application of pumps for the slough outfalls within the Bay View area.

b. Kinetic Pumps

Kinetic pumps use kinetic energy to impart velocity and pressure to a column of fluid as it move through the pump's impeller Current pump stations at the outlets of the No Name Slough and Big Indian Slough utilize kinetic type pumps. Kinetic type pumps offer the following advantages:

- They have a lower capital cost than screw pumps.
- Spare parts are more readily available.
- Operators are more familiar with operation and maintenance of this type of pump than other types of pumps.
- Their required footprint is smaller than that of screw pumps.

B. No Name Slough

According to local property owners, flooding in the No Name Slough drainage basin is widespread in the lower reaches. In addition, stormwater runoff in the steep portions of the drainage basin causes considerable erosion of the stream channel. The following conceptual alternatives were examined to relieve flooding and/or erosion in this drainage basin. Modeling results indicated that flooding in the lower basin of No Name Slough is controlled primarily by tidal elevations at the outlet.

1. Upgrade of Restricted Culverts

The 25-year 24-hour future conditions storm was used to identify capacity-restricted culverts and to determine the necessary culvert size to eliminate the restrictions. **Figure 5-2** identifies the culverts that were included in the hydraulic modeling. **Table 7-1** summarizes the restricted culverts and the size of the required replacement culvert; locations of these culverts are highlighted on **Figure 7-1.**

Table 7-1: No Name Slough Identified Culvert Restrictions					
	25-year	Est. Tailwater	Targeted Head	Recommended Replacement	
Culvert ID	Peak Flow	Elevation	Water Elevation	Culvert Size	Type/Material
NN-C-Out	160 cfs	2 feet	2 feet	4-ft x 12-ft	Box/Concrete
NN-C-5	60 cfs	6 feet	6 feet	54-inch	Circular/CMP
NN-C-3	120 cfs	104 feet	105 feet	84-inch	Circular/CMP
cfs = cubic feet per second; CMP = corrugated metal pipe					

2. Regional Detention

Three regional detention alternatives were evaluated. The location of these ponds are shown in **Figure 7-1.** The first two detention pond locations were east of Farm-to-Market Road to collect runoff from Subbasins A-7 and A-8, which discharge into the south stem of No Name Slough. The third detention pond location was north of Marihugh Road to collect runoff from Subbasin A-11b. The analysis of these two ponds indicated that flooding in the lower basin would not be eliminated by even complete detention of these upland flows. Therefore, the analysis focused on the amount of storage needed to mitigate for future development or, in the case of the Marihugh Road Pond, to potentially reduce erosion in No Name Creek. Three locations were identified as potential pond sites:

- **The Paccar Technical Center**—Since the Port has plans to provide detention for the parcels being developed in the southwest quadrant of its property, this area was not included in a regional detention analysis. A detention pond on the south stem was analyzed, but only to collect runoff from the Paccar Technical Center, assuming full buildout conditions. The pond was sized to detain the 100-year existing conditions peak flow from the Paccar Technical Center.
- **Northwest Corner of Port Property in Subbasin A-11a**—This pond was assumed to collect runoff from the Port's property in Subbasin A-11a. It was assumed that the entire parcel would be developed to full buildout conditions. This is a conservative assumption, since FAA regulations may restrict the amount of the parcel that can be developed. The pond was sized to detain the 100-year existing conditions peak flow.
- **North of Marihugh Road**—Subbasin A-11b is not currently within the UGA and the future-conditions density is assumed to remain close to that of existing conditions. Since erosion in No Name Creek is a documented concern and since there are relatively few sites were detention may be an option, a pond was analyzed at this location as possible mitigation for existing development impacts. This pond was sized to detain the 100-year peak flow from pre-development conditions, as documented in *Padilla Bay Hydrology of No Name Slough Drainage* (November 2000). A linear interpolation, by tributary area, of the peak flow documented at Bay View Road was used to estimate the pre-development flow at Marihugh Road. The pre-developed peak flow at Marihugh Road was estimated to be approximately 23 cubic feet per second (cfs).

Table 7-2 summarizes the approximate storage that would be needed to meet the design goals of the ponds.

Table 7-2: No Name Slough Detention Pond Volumes			
Location	25-year Pond Inflow	Peak Flow Target	Estimated Pond Volume
Paccar	62 cfs	28 cfs	14 ac-ft
Subbasin A-11a	85 cfs	35 cfs	13 ac-ft
Marihugh Road†	42 cfs	18 cfs	32 ac-ft
† Marihugh pond design is based on 25-year peak flow.			

3. High-Flow Bypass

Since flooding in the lowlands is driven primarily by high tides coinciding with peak runoff, there are few feasible options for reducing the flooding. A high-flow bypass was analyzed, and although it was shown to increase the capacity of the slough, lowland flooding during high tide was predicted to be only marginally better than without the bypass. The main benefit of the bypass during a high tide was providing additional storage. However, a high-flow bypass in conjunction with an improved outlet would allow No Name Slough to drain more quickly when the tide recedes, which would reduce flood duration. **Table 7-3** and **Figure 7-2** show flooding locations predicted by the high-flow bypass analysis. Flooding reduction from this alternative is most likely a result of the incremental increase in storage volume along the slough.

4. Increased Outlet Pumping Capacity

One of the few ways to reduce flooding in the slough during a high tide would be to increase the capacity of the pump station at the outlet to allow drainage when the tidal head exceeds flood stage in the slough. A pump station that would eliminate flooding at infrequent recurrence intervals would likely be prohibitively expensive; therefore, a pump station should be designed to reduce more frequent flooding events. This analysis examined the pumping capacity that would be needed to reduce flooding at the 10-year recurrence interval, which was the smallest storm analyzed for this study.

The capacity of the existing pump station at the outlet of No Name slough is approximately 35 cfs. The existing conditions 10-year peak flow at the outlet of the slough is estimated to be 111 cfs, and the future conditions 10-year peak flow is estimated to be 138 cfs.

The analysis of increased pumping capacity was performed at a conceptual level; further study will be required to refine the necessary pumping capacity and the proper operating scheme for an upgraded pump station. It also may be appropriate to examine pumping requirements to reduce flooding for a more frequent flood event, such as the 2-year event. Mitigating for more frequent events would likely result in a smaller pump station that would likely be less expensive to maintain and operate.

Table 7-3: No Name Slough Flooding Locations with High Flow Bypass							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Upstream of Outlet	●	●	●	●	●	●
NN-30	Upstream of Slough Outlet	●	●	●	●	●	●
NN-40	Downstream of Bay View-Edison Road	●	●	●	●	●	●
NN-60	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-65	Slough Upstream of Bay View-Edison Rd	●	●	●	●	●	●
NN-70	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-80	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-140	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-160	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-170	S, Stem Near Dahlstadt Farm	●	●	●	●	●	●
See **Figure 5-2** for node locations. See **Table 5-6** for flooding locations with no improvements.				● denotes predicted flooding for the storm event			

The analysis of pumping improvements indicates that increasing the pumping capacity at the outlet of the slough will reduce water surface elevations in the slough and can reduce flooding in the lowland portion of the basin. **Table 7-4** and **Figure 7-2** show predicted flooding using a pump station with a capacity of 90 cfs that begins pumping when the water surface elevation at the outlet of the slough reaches approximately -2.0 feet NGVD (National Geodetic Vertical Datum).

While the results of the pump station analysis indicate that increased pumping at the slough outlet will reduce water surface elevations in No Name Slough, they do not indicate that flooding would be eliminated from the lowland basins. The simplified nature of the hydraulic model used for the analysis may lead to an overestimation of the effects of increased pumping capacity.

Under previous agreement that no lateral drainages would be modeled, the model assumes that all water draining from the lowland basin enters the slough and that flooding occurs only when the water surface elevations exceed the slough's banks. In reality, as the water surface elevation in the slough increases, the lateral systems will not drain into the slough because they are at lower elevations. Therefore, water will still pond in adjoining fields until the elevation in the slough is lowered.

Table 7-4: No Name Slough Flooding Locations with Increased Pumping Capacity							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Upstream of Outlet		●	●		●	●
NN-30	Upstream of Slough Outlet		●	●		●	●
NN-40	Downstream of Bay View-Edison Road		●	●		●	●
NN-60	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-65	Slough Upstream of Bay View-Edison Rd		●	●		●	●
NN-70	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-80	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-83	Confluence of Tributaries	●	●	●	●	●	●
NN-140	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-160	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-170	S, Stem Near Dahlstadt Farm	●	●	●	●	●	●

See **Figure 5-2** for node locations. ● denotes predicted flooding for the storm event

See **Table 5-6** for flooding locations with no improvements.

The model does not indicate flooding until the slough water surface exceeds the estimated overtopping elevation of 3 feet NGVD; however, in reality water will not drain from the fields into the slough until the water surface elevation in the fields exceeds the water surface elevation in the slough. Since there are many areas in the fields that are at or below this elevation, flooding is likely to continue.

Operation of the pump station also plays a factor in flooding frequency. As the flood wave travels down the slough, it increases water surface elevations in the slough and causes flooding. Reducing flooding in the basin will require either keeping water surface elevations in the slough at a reduced level, or providing a large enough pumping capacity to eliminate flood volumes at a rate fast enough to have an impact on the upstream portion of the slough. Pump station operation will have to maximize the storage available in the slough. In order to operate effectively, the pump station must have a large enough capacity and begin pumping early enough to create enough drawdown at the slough's outlet to facilitate drainage upstream. This may mean that the pump station often will operate under conditions that would not have caused upstream flooding, which is inefficient and may be expensive.

C. Joe Leary Slough

The flooding problems in Joe Leary Slough indicate that the there is not enough capacity in the current system to convey peak flows. The existing hydraulic structures (bridges and culverts) appear to have enough capacity to convey peak flows, however, the low gradient of the slough and the large impact of tidal influence overly restrict the slough capacity.

Alternatives to reducing stormwater drainage impacts that were considered include a bypass channel along the lower reaches of Joe Leary Slough, a pump station at the slough outlet, a pump station at the South Spur Ditch upstream of its confluence with Maiben Ditch, and detention at the slough outlet. The location of these alternatives is shown on **Figure 7-3**.

1. Peth Property Bypass Channel

A bypass channel in the lower portion of the slough along the toe of the hill was examined as a way of reducing flooding. This bypass channel would cross existing farmland owned by John Peth & Sons, LLC. Routing flow more directly to the outlet may reduce drainage times for the low-lying fields and reduce peak flows where the slough is confined along D'Arcy Road. It was assumed for this analysis that a portion of Subbasin B-1 would drain directly into the bypass. The bypass channel would be excavated to the bottom elevation of the existing slough and would not have any levees. The bypass channel was assumed to be trapezoidal in shape with a length of 4,200 feet. The following characteristics were used to define the bypass:

- 6-foot bottom width
- 2:1 side slopes
- 10 feet of total depth
- A constant slope of 0.0028 percent
- Manning's 'n' roughness coefficient of 0.045

Table 7-5 and **Table 7-6** show the modeled effect of the bypass channel on water surface elevations for existing and future conditions, respectively. **Table 7-7** and **Figure 7-4** show the modeled flooding locations for existing and future conditions with the bypass. The modeling shows that the bypass would reduce water surface elevations along nearly the entire length of the slough. In some locations, levels would be reduced by 0.6 feet, 0.9 feet and 1.1 feet for the 10-, 25-, and 100-year future conditions events, respectively. At high tides the bypass would provide incremental storage, reducing the volume of water stored in the main stem of the slough. During low tides, the bypass would facilitate drainage in the fields by providing an additional drainage path to the outlet of the slough, directing some of the peak flow away from the channel restriction along D'Arcy Road.

This analysis assumed no upstream flooding, so that the maximum peak flow and volume reach the slough's outlet. In fact, upstream flooding would likely reduce peak flow in the channel and reduce the magnitude that water levels would be lowered by the bypass channel. However, the conclusion that levels would be reduced by the bypass still appears to be valid.

Table 7-5: Joe Leary Slough Existing Conditions Water Surface Elevations With and Without Slough Bypass									
SWMM Model Node	Water Surface Elevation No Improvements			Water Surface Elevation With Slough Bypass			Difference		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Main Stem									
JL-20	3.9 feet	3.9 feet	3.9 feet	3.8 feet	3.9 feet	3.9 feet	-0.1 feet	0.0 feet	0.0 feet
JL-30	3.9 feet	4.1 feet	4.3 feet	3.8 feet	4.0 feet	4.2 feet	-0.1 feet	-0.1 feet	-0.1 feet
JL-40	4.1 feet	4.4 feet	4.9 feet	3.9 feet	4.1 feet	4.4 feet	-0.2 feet	-0.3 feet	-0.5 feet
JL-50	4.6 feet	5.2 feet	5.9 feet	4.1 feet	4.4 feet	4.9 feet	-0.5 feet	-0.8 feet	-1.0 feet
JL-60	4.8 feet	5.5 feet	6.3 feet	4.4 feet	4.9 feet	5.5 feet	-0.4 feet	-0.6 feet	-0.8 feet
JL-70	5.1 feet	5.9 feet	7.0 feet	4.8 feet	5.4 feet	6.2 feet	-0.3 feet	-0.5 feet	-0.8 feet
JL-80	5.4 feet	6.2 feet	7.3 feet	5.0 feet	5.7 feet	6.7 feet	-0.4 feet	-0.5 feet	-0.6 feet
JL-90	5.8 feet	6.6 feet	7.9 feet	5.5 feet	6.3 feet	7.3 feet	-0.3 feet	-0.3 feet	-0.6 feet
JL-100	6.5 feet	7.5 feet	9.1 feet	6.3 feet	7.3 feet	8.7 feet	-0.2 feet	-0.2 feet	-0.4 feet
JL-110	6.5 feet	7.6 feet	9.2 feet	6.3 feet	7.4 feet	8.8 feet	-0.2 feet	-0.2 feet	-0.4 feet
JL-120	6.7 feet	7.8 feet	9.3 feet	6.5 feet	7.5 feet	9.0 feet	-0.2 feet	-0.3 feet	-0.3 feet
Maiben Ditch									
JL-130	7.2 feet	8.2 feet	9.8 feet	7.0 feet	8.1 feet	9.5 feet	-0.2 feet	-0.1 feet	-0.3 feet
JL-140	8.0 feet	9.0 feet	10.4 feet	7.9 feet	8.9 feet	10.2 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-150	8.6 feet	9.6 feet	10.9 feet	8.6 feet	9.5 feet	10.7 feet	0.0 feet	-0.1 feet	-0.2 feet
JL-160	10.0 feet	10.8 feet	11.9 feet	10.0 feet	10.8 feet	11.9 feet	0.0 feet	0.0 feet	0.0 feet
South Spur Ditch									
JL-161	7.1 feet	8.2 feet	9.7 feet	7.0 feet	8.0 feet	9.4 feet	-0.1 feet	-0.2 feet	-0.3 feet
JL-170	7.1 feet	8.2 feet	9.7 feet	7.0 feet	8.1 feet	9.5 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-180	7.2 feet	8.3 feet	9.8 feet	7.1 feet	8.1 feet	9.5 feet	-0.1 feet	-0.2 feet	-0.3 feet
JL-200	7.4 feet	8.4 feet	9.8 feet	7.3 feet	8.3 feet	9.6 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-220	7.4 feet	8.4 feet	9.8 feet	7.3 feet	8.3 feet	9.6 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-250	7.5 feet	8.5 feet	9.9 feet	7.4 feet	8.3 feet	9.6 feet	-0.1 feet	-0.2 feet	-0.3 feet
See **Figure 5-3** for node locations.									

Table 7-6: Joe Leary Slough Future Conditions Water Surface Elevations With and Without Slough Bypass									
SWMM Model Node	Water Surface Elevation No Improvements			Water Surface Elevation With Slough Bypass			Difference		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Main Stem									
JL-20	3.9 feet	3.9 feet	3.9 feet	3.8 feet	3.9 feet	3.9 feet	-0.1 feet	0.0 feet	0.0 feet
JL-30	4.0 feet	4.1 feet	4.4 feet	3.9 feet	4.0 feet	4.2 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-40	4.1 feet	4.5 feet	5.0 feet	4.0 feet	4.1 feet	4.5 feet	-0.1 feet	-0.4 feet	-0.5 feet
JL-50	4.7 feet	5.3 feet	6.1 feet	4.1 feet	4.4 feet	5.0 feet	-0.6 feet	-0.9 feet	-1.1 feet
JL-60	4.9 feet	5.6 feet	6.4 feet	4.5 feet	4.9 feet	5.6 feet	-0.4 feet	-0.7 feet	-0.8 feet
JL-70	5.2 feet	6.0 feet	7.1 feet	4.9 feet	5.5 feet	6.3 feet	-0.3 feet	-0.5 feet	-0.8 feet
JL-80	5.5 feet	6.3 feet	7.5 feet	5.2 feet	5.8 feet	6.8 feet	-0.3 feet	-0.5 feet	-0.7 feet
JL-90	5.9 feet	6.8 feet	8.0 feet	5.7 feet	6.4 feet	7.5 feet	-0.2 feet	-0.4 feet	-0.5 feet
JL-100	6.6 feet	7.7 feet	9.3 feet	6.4 feet	7.5 feet	8.9 feet	-0.2 feet	-0.2 feet	-0.4 feet
JL-110	6.7 feet	7.8 feet	9.4 feet	6.5 feet	7.6 feet	9.0 feet	-0.2 feet	-0.2 feet	-0.4 feet
JL-120	6.8 feet	7.9 feet	9.6 feet	6.7 feet	7.7 feet	9.2 feet	-0.1 feet	-0.2 feet	-0.4 feet
Maiben Ditch									
JL-130	7.3 feet	8.4 feet	10.0 feet	7.2 feet	8.3 feet	9.7 feet	-0.1 feet	-0.1 feet	-0.3 feet
JL-140	8.1 feet	9.1 feet	10.6 feet	8.0 feet	9.0 feet	10.4 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-150	8.7 feet	9.7 feet	11.0 feet	8.7 feet	9.6 feet	10.9 feet	0.0 feet	-0.1 feet	-0.1 feet
JL-160	10.0 feet	10.9 feet	12.1 feet	10.0 feet	10.9 feet	12.0 feet	0.0 feet	0.0 feet	-0.1 feet
South Spur Ditch									
JL-161	7.3 feet	8.4 feet	9.9 feet	7.1 feet	8.2 feet	9.6 feet	-0.2 feet	-0.2 feet	-0.3 feet
JL-170	7.3 feet	8.4 feet	10.0 feet	7.2 feet	8.3 feet	9.7 feet	-0.1 feet	-0.1 feet	-0.3 feet
JL-180	7.4 feet	8.5 feet	10.0 feet	7.3 feet	8.3 feet	9.7 feet	-0.1 feet	-0.2 feet	-0.3 feet
JL-200	7.6 feet	8.6 feet	10.1 feet	7.5 feet	8.5 feet	9.8 feet	-0.1 feet	-0.1 feet	-0.3 feet
JL-220	7.6 feet	8.6 feet	10.1 feet	7.5 feet	8.5 feet	9.8 feet	-0.1 feet	-0.1 feet	-0.3 feet
JL-250	7.7 feet	8.7 feet	10.1 feet	7.6 feet	8.6 feet	9.9 feet	-0.1 feet	-0.1 feet	-0.2 feet
See **Figure 5-3** for node locations.									

Table 7-7: Joe Leary Slough Main Stem Flooding With and Without Slough Bypass									
SWMM Model Node	Future Conditions Flooding Locations Without Improvements			Flooding Locations With Slough Bypass					
				Existing Conditions			Future Conditions		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
JL-20	●	●	●		●	●		●	●
JL-30	●	●	●	●	●	●	●	●	●
JL-40	●	●	●	●	●	●	●	●	●
JL-50	●	●	●			●			●
JL-60		●	●			●			●
JL-71	●	●	●		●	●		●	●
JL-80		●	●		●	●		●	●
JL-90		●	●			●			●
JL-105		●	●		●	●		●	●
JL-110		●	●		●	●		●	●

See **Figure 7-4** for node locations. ● denotes predicted flooding for the storm event

2. Culvert Analysis

None of the culverts along the main stem of Joe Leary Slough appear to be significant flow restrictions. No culvert replacements are recommended. The outlet structure of twelve (12) 48-inch culverts seems to be adequate to drain the slough during a low tide. No changes are recommended at the outlet.

3. Pump Station at the Outlet

A pump station at the outlet 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 outlet. 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 outlet culvert can exceed the storage capacity of the slough and cause flooding or raise water levels in the slough to prevent drainage from adjacent fields that are at 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 cfs.

Table 7-8 and **Figure 7-4** summarize flooding locations for the 10-year existing and future conditions event with a pump capacity of approximately 300 cfs.

Table 7-8: Joe Leary Slough 10-Year Event Flooding Locations With and Without 300-cfs Outlet Pump Station			
SWMM Model Node	**10-Year Flooding Locations Without Improvements**	**10-Year Flooding Locations With 300-cfs Pump Station at Outlet**	
		Existing Conditions	**Future Conditions**
JL-20	●		
JL-24	●		
JL-25	●		
JL-26	●		
JL-30	●		
JL-32	●		
JL-34	●		
JL-36	●		
JL-40	●		
JL-44	●		●
JL-46	●		
JL-48	●		
JL-50	●	●	●
JL-52	●	●	●
JL-54	●	●	●
See **Figure 7-4** for node locations.		● denotes predicted flooding for the storm event	

Table 7-9 shows the effect of a pump station on slough elevations. According to the model results, a pump station at the outlet would provide the most benefit from the outlet 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 should be expected.

Table 7-9: Joe Leary Slough Future Conditions Water Surface Elevations With and Without Outlet Pump Station - 10-Year Storm Event						
	Existing Conditions Water Surface Elevations			Future Conditions Water Surface Elevations		
SWMM Model Node	No Improvements	With Pump Station	Difference	No Improvements	With Pump Station	Difference
Main Stem						
JL-20	3.9 feet	0.5 feet	-3.4 feet	3.9 feet	0.5 feet	-3.4 feet
JL-30	3.9 feet	1.2 feet	-2.7 feet	4.0 feet	1.2 feet	-2.8 feet
JL-40	4.1 feet	2.4 feet	-1.7 feet	4.1 feet	2.4 feet	-1.7 feet
JL-50	4.6 feet	3.8 feet	-0.8 feet	4.7 feet	3.9 feet	-0.8 feet
JL-60	4.8 feet	4.3 feet	-0.5 feet	4.9 feet	4.3 feet	-0.6 feet
JL-70	5.1 feet	4.7 feet	-0.4 feet	5.2 feet	4.8 feet	-0.4 feet
JL-80	5.4 feet	5.1 feet	-0.3 feet	5.5 feet	5.1 feet	-0.4 feet
JL-90	5.8 feet	5.6 feet	-0.2 feet	5.9 feet	5.7 feet	-0.2 feet
JL-100	6.5 feet	6.4 feet	-0.1 feet	6.6 feet	6.5 feet	-0.1 feet
JL-110	6.5 feet	6.5 feet	0.0 feet	6.7 feet	6.6 feet	-0.1 feet
JL-120	6.7 feet	6.7 feet	0.0 feet	6.8 feet	6.8 feet	0.0 feet
Maiben Ditch						
JL-130	7.2 feet	7.2 feet	0.0 feet	7.3 feet	7.3 feet	0.0 feet
JL-140	8.0 feet	8.0 feet	0.0 feet	8.1 feet	8.1 feet	0.0 feet
JL-150	8.6 feet	8.6 feet	0.0 feet	8.7 feet	8.7 feet	0.0 feet
JL-160	10.0 feet	10.0 feet	0.0 feet	10.0 feet	10.0 feet	0.0 feet
South Spur Ditch						
JL-161	7.1 feet	7.1 feet	0.0 feet	7.3 feet	7.2 feet	-0.1 feet
JL-170	7.1 feet	7.1 feet	0.0 feet	7.3 feet	7.3 feet	0.0 feet
JL-180	7.2 feet	7.2 feet	0.0 feet	7.4 feet	7.4 feet	0.0 feet
JL-200	7.4 feet	7.4 feet	0.0 feet	7.6 feet	7.6 feet	0.0 feet
JL-220	7.4 feet	7.4 feet	0.0 feet	7.6 feet	7.6 feet	0.0 feet
JL-250	7.5 feet	7.5 feet	0.0 feet	7.7 feet	7.7 feet	0.0 feet
See **Figure 5-3** for node locations.						

4. South Spur Pump Station

A pump station on the South Spur Ditch was examined as a way to reduce water surface elevations in that portion of the slough. It was assumed that a culvert with a flap gate would be installed on the South Spur Ditch along with the pump station. The flap gate would prevent back flow from the downstream portion of the slough, and the pump station would pass the peak flow in the slough when the flow in the culvert is reduced by downstream hydraulic conditions.

The modeling showed that a pump station on the South Spur ditch would be most effective at reducing flooding for storms at or below the 10-year recurrence interval. A pump station with a maximum capacity of 48 cfs appears to be effective at reducing flooding for the 10-year existing conditions storm event, while a pump station with a maximum capacity of 73 cfs appears to be effective at reducing flooding for the 10-year future conditions storm event. The model results indicate a restriction in the South Spur Ditch that reduces the pump station's effectiveness in reducing flooding. The restriction appears to be a narrowing of the channel approximately 800 feet downstream of the Josh Wilson Road culvert crossing (see **Figure 7-3**). The channel cross-section at the restriction is represented by Cross Section BX-18, which is smaller than Cross Section BX-17; the smallest cross-sectional required to convey the peak flow from the 10-year storm. The existing ditch cross-section geometry is shown in **Appendix A.** If the South Spur Ditch is enlarged to match the dimensions of Cross Section BX-17, flooding could be reduced along the entire length of the South Spur Ditch. If the restriction is not removed, flooding would still occur in the low area between the Josh Wilson Road culvert and the Michael Road culvert. **Table 7-10** shows the modeled effect of the pump station on water surface elevations for existing and future conditions in the South Spur Ditch at the 10-year event.

Table 7-10: Joe Leary Slough Water Surface Elevation With and Without South Spur Pump Station - 10-Year Storm Event

SWMM Model Node	Water Surface Elevation No Improvements		Water Surface Elevation With Pump Station		Difference	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
JL-161	7.1 feet	7.3 feet	7.1 feet	7.4 feet	0.0 feet	0.1 feet
JL-170	7.1 feet	7.3 feet	7.1 feet	7.4 feet	0.0 feet	0.1 feet
JL-180	7.2 feet	7.4 feet	3.7 feet	3.7 feet	-3.5 feet	-3.7 feet
JL-200	7.4 feet	7.6 feet	4.9 feet	5.1 feet	-2.5 feet	-2.5 feet
JL-220	7.4 feet	7.6 feet	5.0 feet	5.7 feet	-2.4 feet	-1.9 feet
JL-250	7.5 feet	7.7 feet	5.4 feet	6.4 feet	-2.1 feet	-1.3 feet

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

Downstream impacts from a pump station on the South Spur Ditch appear to be relatively small. Downstream water levels at some locations would increase by a maximum of 0.1 feet for the 10-year storm event. The increase in flow and water surface elevation downstream could increase flooding and drainage times in the downstream portion of the slough. These effects could be mitigated by implementing alternatives to address downstream flooding; however, additional modeling should be done to quantify downstream impacts for a variety of tidal conditions.

For the 25-year and 100-year storm events, additional pumping capacity would likely help to reduce the duration of flooding; however, flooding would still likely occur for these storm events due to the low elevations in this area and the limited capacity of the channel. A pump station to control the larger storm events was not analyzed in detail for this summary because of its lower expected cost/benefit ratio and the expected increase in downstream impacts.

5. Detention at the 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 and 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.

6. South Spur Ditch Bypass Channel

This bypass channel would be constructed along the Bay View Ridge hillside parallel to the South Spur Ditch. The purpose of this bypass channel would be to reduce the required capacity of the South Spur Pump Station. Because this bypass channel would be located on the hillside, a levee would need to be constructed on the downhill side of the new channel. The toe of the new bypass channel levee would be above existing South Spur Ditch.

The total length of the bypass channel would be approximately 7,400 LF, extending from the confluence of the Maben Ditch and the South Spur Ditch to the west boundary of the Kabalo Heights Plat. The bypass channel would cross property owned by the Sakuma Brothers, Ed Knutzen, Jerry Nelson, and Ray Jensen. The bypass channel was assumed to be trapezoidal in shape with the following characteristics:

- 5 to 8-foot bottom width
- 2:1 side slopes inside the channel; 4:1 slopes on the outside of the levee
- Depth varies from 10 feet at the downstream end to 8 feet at the upstream end
- A constant slope of 0.0028 percent
- Manning's 'n' roughness coefficient of 0.045

A hydraulic model of this bypass channel showed that the South Spur Pump Station would be most effective at reducing flooding from storms at of below the 10-year recurrence interval. The proposed South Spur Pump Station would still be needed to control flooding at higher recurrence intervals, but the maximum capacity could be reduced. The hydraulic analysis estimated the pump station capacity could be reduced to 20 cfs for existing conditions and 25 cfs for future conditions.

7. Alternatives Eliminated from Detailed Analysis

Two measures that may reduce flooding in the Joe Leary Basin were considered briefly but no detailed analysis was performed:

- **Upland Detention**—According to current land use projections, the upland area, especially the area within the UGA boundary, will experience an increase in impervious area under future conditions. We would expect detention facilities will be required as part of any development proposals in the upland areas. The sloping terrain may make it difficult to construct one large detention pond. However, terraced detention pond cells, such as those along Higgins-Airport Way, are possible. Detention ponds would have the effect of reducing the peak runoff, which will facilitate operation of a pump station on the South Spur Ditch.

- **Dikes**—Adding dikes to the lower portion would decrease the occurrence of flooding from flows overtopping the banks of the slough. However, flooding would still occur in adjacent fields with low ground elevations due to the lack of drainage. Pumping would be required to drain the fields during periods of high water levels in the slough. The model for this basin does not include lateral drainages from the fields and would need significant modification to analyze a diking alternative.

D. Little Indian Slough

Flooding in Little Indian Slough appears to be limited to the upper portion of the basin. This is expected due to the potential increase in impervious area within the upper portion of the basin. **Figure 7-5** shows the location of the flood reduction alternatives that were evaluated. The following conceptual alternatives are proposed to relieve flooding in the area.

1. Upstream Culvert and Channel Upgrades

Based on the modeling results, the entire drainage system upstream of Farm-to-Market Road should be upgraded. Culverts LI-C-1 and LI-C-2 both would pass the 25-year future-conditions flood if upgraded to 48-inch circular corrugated metal pipe culverts. In addition, the capacity of the slough should be increased. The improved channel was modeled as trapezoidal in shape and having the following characteristics:

- 3-foot bottom width
- 2:1 side slopes
- 3 feet of total depth
- A constant slope of 0.2 percent
- Manning's 'n' roughness coefficient of 0.050.

With these culvert and channel upgrades, all flooding upstream of Farm-to-Market Road can be eliminated. The changes would not have a significant impact on downstream flooding.

2. Upstream Regional Detention

Subbasin C-2 is the only part of the Little Indian Slough that is expected to experience development under future conditions. Therefore, detention in the upper portion of the basin to mitigate for the additional runoff was investigated.

Table 7-11 and **Table 7-12** show the expected increase in peak flow and runoff volume in Subbasin C-2 under future conditions. Based on the modeling, a pond volume of approximately 15 acre-feet is needed to attenuate flows and eliminate flooding in the upper portion of the slough. The detention pond volume is controlled by the size of the existing downstream culverts (24-inch circular).

Upstream detention, without the proposed culvert and channel improvements, would eliminate flooding upstream of Farm-to-Market Road. It would not have a significant effect on flooding downstream of Farm-to-Market Road. High tide elevations coupled with low ground elevations appear to control flooding in the lower slough.

Figure 7-6 shows the modeled effect of these flood reduction alternatives.

Table 7-11: Little Indian Slough Existing and Future Conditions Peak Flow

SWMM Model Node	Approximate Location	Peak Flow Conditions					
		No Improvements			Higgins Overflow Removed		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-10	Outlet of Little Indian Slough	83 cfs	100 cfs	121 cfs	86 cfs	103 cfs	127 cfs
LI-32	Bay View-Edison Road to Farm-to-Market Road	16 cfs	23 cfs	35 cfs	25 cfs	35 cfs	50 cfs
LI-40	Farm-to-Market Road	13 cfs	18 cfs	26 cfs	23 cfs	30 cfs	41 cfs

See **Figure 5-4** for node locations.

Table 7-12: Little Indian Slough Existing and Future Conditions Runoff Volume

Subbasin	Runoff Volume					
	Existing Conditions			Future Conditions		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
C-1a	9 ac-ft	14 ac-ft	22 ac-ft	9 ac-ft	14 ac-ft	22 ac-ft
C-1b	11 ac-ft	18 ac-ft	29 ac-ft	11 ac-ft	18 ac-ft	29 ac-ft
C-1c	4 ac-ft	6 ac-ft	9 ac-ft	4 ac-ft	6 ac-ft	9 ac-ft
C-2	12 ac-ft	18 ac-ft	27 ac-ft	17 ac-ft	23 ac-ft	32 ac-ft

See **Figure 5-4** for basin boundaries.

E. Big Indian Slough

Flooding in Big Indian Slough appears to be concentrated near the confluence of the runoff from the Urban Growth Area (including Skagit Regional Airport) and the main stem of Big Indian Slough. This confluence of large flows apparently overwhelms the existing conveyance system, specifically culverts, and causes flooding in the general vicinity. The following conceptual alternatives are proposed to relieve flooding in the area. **Figure 7-7** shows the location of the flood reduction alternatives that were evaluated.

1. State Route 20 Bypass Channel

A bypass along State Route 20 (SR 20) northeast of Bradshaw Road was examined as a way of reducing flooding of the culverts north of SR 20. Routing flow along the south side of SR 20 may reduce peak flows and flooding. It was assumed that the portion of Subbasin C-3 that is south of SR 20 would drain directly into the bypass. The bypass channel was assumed to be trapezoidal in shape with a length of approximately 3,100 LF. The following characteristics were used to define the bypass:

- 5-foot bottom width
- 3:1 side slopes
- 9 feet of total depth
- A constant slope of 0.1 percent
- Manning's 'n' roughness coefficient of 0.045.

According to the modeling results, the bypass would have the effect of reversing flow in the portion of the slough east of the inflow point from the Port's property (Node BI-200). The peak flow from the Port's stormwater facility along Higgins Airport Way (Subbasin C-7) lags slightly behind the peak flow from Subbasins C-4 and C-5. As the slough downstream of the inflow point fills to capacity, water would begin to flow east into the extra capacity of that channel and flow through the bypass as it drains to the outfall. This would reduce flooding along that portion of the slough for a normal tide cycle.

In general, the bypass lowers water surface elevations by less than 1 foot in the upper portion of the model, but some flooding was eliminated. Based on the topographic mapping, the remaining flooding in this portion of the slough does affect some agricultural fields in this area, but there is no impact to any homes, structures or major roads. Flooding would be limited to overtopping the smaller field access culverts and would largely remain in the slough corridor. Elevations in the fields are on the order of 12.5 feet, and the water surface elevation for the 100-year future conditions storm would not exceed 11 feet.

According to the modeling results, the Higgins Airport Way culvert (BI-C-4) would no longer flood at the 100-year existing or future conditions event with the SR 20 Bypass Channel in place. Overtopping would occur at culverts BI-C-2, BI-C-3, and BI-C-5. Culvert capacity does not appear to be a direct cause of flooding at culvert BI-C-2, but replacing the three culverts at this location with one large culvert would improve hydraulic efficiency. At this location, flooding appears to occur as a result of the backwater downstream and the low overtopping elevation of the culvert (approximately 7.5 feet). Flooding at culverts BI-C-3 and BI-C-5 appears to be a result of the culverts being undersized and their low overtopping elevation. The main function of these culverts is to provide access to the agricultural fields between the slough and SR 20. Due to the limited traffic that uses this access, flooding at the recurrence intervals seen here may be acceptable.

2. Ovenell Road Bypass Channel

A more effective solution to reduce flooding in the upper reaches of the Big Indian Slough Basin is to construct a new bypass channel along Ovenell Road that starts west of the Skagit Golf & Country Club and extends west and south, connecting to Big Indian Slough near the intersection with State Routes 20 and 536. The purpose of this bypass channel would be to divert stormwater runoff from the entering Big Indian Slough near the location of several undersized culverts and low farm fields.

Discharge from the new bypass channel would enter the Big Indian Slough near the beginning of the manmade section of the existing channel. Below this point there appears to be sufficient capacity to accommodate additional stormwater runoff.

The total length of the Ovenell Road Bypass Channel is approximately 11,600 LF. The following characteristics were used to define the bypass:

- 3½ to 6-foot bottom width
- 2:1 side slopes
- Depth varies from 8 feet at the downstream end to 3.5 feet at the upstream end
- A constant slope of 0.1 percent
- Manning's 'n' roughness coefficient of 0.045

Table 7-13 and **Table 7-14** show the modeled effect of the bypass channel on water surface elevations for existing and future conditions, respectively. In general, the bypass channel lowers water surface elevations by 1 to 4 feet in the upper portion of the basin that was modeled. Based on the topographic mapping, the remaining flooding in this portion of the slough does not appear to significantly affect the agricultural fields in this area, nor does it impact any homes, structures or major roads. Flooding would be limited to overtopping the smaller culverts and would largely remain in the slough corridor. Elevations in the fields are on the order of 12.5 feet, and the water surface elevation for the 100-year future conditions storm would not exceed 8.3 feet.

The environmental concerns of this bypass channel are significant. The Big Indian Slough is a fish bearing watershed. Therefore, any channel construction and improvements will require accommodation of fisheries habitat. This may include, but not limited to, construction of ponds, fish friendly culverts and spawning beds, and establishment of vegetated buffers. Permitting negotiations for this project are expected to take several years.

Table 7-13: Big Indian Slough Existing Conditions Water Surface Elevations With and Without Bypass Channel

SWMM Model Node	Water Surface Elevation No Improvements			Water Surface Elevation With Bypass Channel			Difference		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Main Stem - Below SR 20									
BI-20	3.0 feet	3.7 feet	3.8 feet	3.1 feet	3.7 feet	3.8 feet	0.1 feet	0.0 feet	0.0 feet
BI-30	3.0 feet	3.7 feet	3.9 feet	3.1 feet	3.7 feet	3.9 feet	0.1 feet	0.0 feet	0.0 feet
BI-40	3.0 feet	3.8 feet	3.9 feet	3.1 feet	3.8 feet	3.9 feet	0.1 feet	0.0 feet	0.0 feet
BI-50	3.1 feet	3.8 feet	3.9 feet	3.1 feet	3.8 feet	3.9 feet	0.0 feet	0.0 feet	0.0 feet
BI-60	3.1 feet	3.8 feet	4.0 feet	3.1 feet	3.8 feet	4.0 feet	0.0 feet	0.0 feet	0.0 feet
BI-70	3.2 feet	3.9 feet	4.1 feet	3.2 feet	3.9 feet	4.1 feet	0.0 feet	0.0 feet	0.0 feet
BI-80	3.2 feet	4.0 feet	4.2 feet	3.2 feet	4.0 feet	4.2 feet	0.0 feet	0.0 feet	0.0 feet
BI-90	3.2 feet	4.0 feet	4.2 feet	3.3 feet	4.0 feet	4.2 feet	0.1 feet	0.0 feet	0.0 feet
BI-100	3.7 feet	4.6 feet	5.0 feet	3.7 feet	4.5 feet	5.0 feet	0.0 feet	-0.1 feet	0.0 feet
BI-110	4.4 feet	5.4 feet	6.0 feet	4.4 feet	5.3 feet	6.0 feet	0.0 feet	-0.1 feet	0.0 feet
Main Stem - Above SR 20									
BI-120	4.4 feet	5.4 feet	6.0 feet	4.4 feet	5.3 feet	6.0 feet	0.0 feet	-0.1 feet	0.0 feet
BI-130	5.5 feet	6.8 feet	7.6 feet	4.4 feet	5.4 feet	6.1 feet	-1.1 feet	-1.4 feet	-1.5 feet
BI-140	6.4 feet	7.9 feet	8.8 feet	4.5 feet	5.5 feet	6.3 feet	-1.9 feet	-2.4 feet	-2.5 feet
BI-150	6.5 feet	7.9 feet	8.9 feet	4.5 feet	5.5 feet	6.3 feet	-2.0 feet	-2.4 feet	-2.6 feet
BI-160	6.9 feet	8.4 feet	9.4 feet	4.6 feet	5.6 feet	6.3 feet	-2.3 feet	-2.8 feet	-3.1 feet
BI-170	6.9 feet	8.4 feet	9.4 feet	4.6 feet	5.6 feet	6.4 feet	-2.3 feet	-2.8 feet	-3.0 feet
BI-180	7.4 feet	8.9 feet	10.6 feet	4.7 feet	5.7 feet	6.5 feet	-2.7 feet	-3.2 feet	-4.1 feet
BI-190	7.5 feet	9.0 feet	10.7 feet	4.7 feet	5.7 feet	6.5 feet	-2.8 feet	-3.3 feet	-4.2 feet
JL-200	7.8 feet	9.3 feet	10.9 feet	5.0 feet	5.7 feet	6.5 feet	-2.8 feet	-3.6 feet	-4.4 feet
JL-210	7.8 feet	9.4 feet	11.0 feet	5.1 feet	5.8 feet	6.6 feet	-2.7 feet	-3.6 feet	-4.4 feet
JL-220	7.8 feet	9.4 feet	11.1 feet	5.9 feet	6.3 feet	6.9 feet	-1.9 feet	-3.1 feet	-4.2 feet
JL-230	7.8 feet	9.4 feet	11.1 feet	5.9 feet	6.3 feet	6.9 feet	-1.9 feet	-3.1 feet	-4.2 feet
JL-240	7.8 feet	9.4 feet	11.1 feet	5.9 feet	6.4 feet	7.0 feet	-1.9 feet	-3.0 feet	-4.1 feet
JL-250	7.8 feet	9.4 feet	11.1 feet	6.0 feet	6.4 feet	7.0 feet	-1.8 feet	-3.0 feet	-4.1 feet
JL-260	7.8 feet	9.4 feet	11.1 feet	6.0 feet	6.5 feet	7.1 feet	-1.8 feet	-2.9 feet	-4.0 feet
JL-270	7.9 feet	9.4 feet	11.1 feet	6.2 feet	6.7 feet	7.3 feet	-1.7 feet	-2.7 feet	-3.8 feet

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

Table 7-14: Big Indian Slough Future Conditions Water Surface Elevations With and Without Bypass Channel									
SWMM Model Node	Water Surface Elevation No Improvements			Water Surface Elevation With Bypass Channel			Difference		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Main Stem - Below SR 20									
BI-20	3.4 feet	3.8 feet	3.9 feet	3.3 feet	3.8 feet	3.8 feet	-0.1 feet	0.0 feet	-0.1 feet
BI-30	3.4 feet	3.8 feet	3.9 feet	3.3 feet	3.8 feet	3.9 feet	-0.1 feet	0.0 feet	0.0 feet
BI-40	3.4 feet	3.8 feet	3.9 feet	3.4 feet	3.8 feet	3.9 feet	0.0 feet	0.0 feet	0.0 feet
BI-50	3.4 feet	3.8 feet	3.9 feet	3.4 feet	3.8 feet	3.9 feet	0.0 feet	0.0 feet	0.0 feet
BI-60	3.4 feet	3.9 feet	4.0 feet	3.4 feet	3.9 feet	4.0 feet	0.0 feet	0.0 feet	0.0 feet
BI-70	3.5 feet	4.0 feet	4.1 feet	3.5 feet	4.0 feet	4.1 feet	0.0 feet	0.0 feet	0.0 feet
BI-80	3.5 feet	4.0 feet	4.2 feet	3.5 feet	4.0 feet	4.2 feet	0.0 feet	0.0 feet	0.0 feet
BI-90	3.6 feet	4.0 feet	4.3 feet	3.5 feet	4.1 feet	4.3 feet	-0.1 feet	0.1 feet	0.0 feet
BI-100	4.0 feet	4.7 feet	5.2 feet	3.9 feet	4.7 feet	5.1 feet	-0.1 feet	0.0 feet	-0.1 feet
BI-110	4.7 feet	5.5 feet	6.2 feet	4.6 feet	5.5 feet	6.1 feet	-0.1 feet	0.0 feet	-0.1 feet
Main Stem - Above SR 20									
BI-120	5.8 feet	7.0 feet	7.9 feet	4.7 feet	5.6 feet	6.3 feet	-1.1 feet	-1.4 feet	-1.6 feet
BI-130	6.8 feet	8.1 feet	9.3 feet	4.9 feet	5.7 feet	6.6 feet	-1.9 feet	-2.4 feet	-2.7 feet
BI-140	6.8 feet	8.1 feet	9.3 feet	4.9 feet	5.7 feet	6.6 feet	-1.9 feet	-2.4 feet	-2.7 feet
BI-150	7.3 feet	8.6 feet	9.9 feet	5.1 feet	5.9 feet	6.8 feet	-2.2 feet	-2.7 feet	-3.1 feet
BI-160	7.3 feet	8.7 feet	9.9 feet	5.1 feet	6.0 feet	6.8 feet	-2.2 feet	-2.7 feet	-3.1 feet
BI-170	7.8 feet	9.2 feet	10.9 feet	5.4 feet	6.2 feet	7.1 feet	-2.4 feet	-3.0 feet	-3.8 feet
BI-180	7.9 feet	9.3 feet	11.1 feet	5.4 feet	6.2 feet	7.1 feet	-2.5 feet	-3.1 feet	-4.0 feet
BI-190	8.1 feet	9.6 feet	11.3 feet	5.7 feet	6.4 feet	7.3 feet	-2.4 feet	-3.2 feet	-4.0 feet
JL-200	8.2 feet	9.7 feet	11.5 feet	5.8 feet	6.5 feet	7.4 feet	-2.4 feet	-3.2 feet	-4.1 feet
JL-210	8.3 feet	9.7 feet	11.5 feet	6.4 feet	7.0 feet	7.8 feet	-1.9 feet	-2.7 feet	-3.7 feet
JL-220	8.3 feet	9.7 feet	11.5 feet	6.4 feet	7.0 feet	7.8 feet	-1.9 feet	-2.7 feet	-3.7 feet
JL-230	8.3 feet	9.7 feet	11.5 feet	6.5 feet	7.0 feet	7.8 feet	-1.8 feet	-2.7 feet	-3.7 feet
JL-240	8.3 feet	9.7 feet	11.5 feet	6.5 feet	7.1 feet	7.9 feet	-1.8 feet	-2.6 feet	-3.6 feet
JL-250	8.3 feet	9.8 feet	11.5 feet	6.6 feet	7.2 feet	8.0 feet	-1.7 feet	-2.6 feet	-3.5 feet
JL-260	8.4 feet	9.8 feet	11.5 feet	6.8 feet	7.4 feet	8.2 feet	-1.6 feet	-2.4 feet	-3.3 feet
JL-270	8.4 feet	9.8 feet	11.5 feet	6.9 feet	7.5 feet	8.3 feet	-1.5 feet	-2.3 feet	-3.2 feet
See **Figure 5-5** for node locations.									

3. Downstream Detention

A detention pond located near the outlet of the slough was originally proposed as a means of mitigating additional runoff volume from future land uses. A pond at this location could absorb additional volume and lower water surface elevations when the tide gates are closed. The area of the parcel considered for storage is approximately 23 acres. The average elevation of the ground surface in this parcel is 3.3 feet, although there are some areas as low as 2.5 feet. Two detention options were evaluated: one using the existing field level for storage and one using an excavated pond.

Field Level Storage: For modeling purposes, it was assumed that 5 acres would be available for storage at an elevation of 2.5 feet and the full 23 acres would be available at elevations above 3.3 feet.

The results of the modeling indicate that field level detention would have no impact on the overflow to Higgins Slough or on the upstream flooding culverts for the modeled tide cycle. **Table 7-15** shows predicted water surface elevations. The modeling predicts a peak water surface elevation of 3.2 feet at the slough outlet for the 100-year storm event. At this elevation, the detention pond would provide approximately 8 acre-feet of storage, which does not appear to be enough to impact the water surface elevations in the slough.

Table 7-15: Big Indian Slough Field Level Detention Analysis Results

SWMM Model Node	Approximate Location	Future Conditions Water Surface Elevation					
		Without Detention			With Field Level Detention		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet of Big Indian Slough	3.0 feet	3.2 feet	3.2 feet	3.0 feet	3.2 feet	3.2 feet
BI-90	Farm-to-Market Road	3.1 feet	3.4 feet	3.6 feet	3.0 feet	3.4 feet	3.6 feet
BI-120	Higgins Slough Overflow	4.4 feet	5.4 feet	6.1 feet	4.4 feet	5.4 feet	6.1 feet
BI-200	Higgins Airport Way	8.1 feet	9.6 feet	10.9 feet	8.1 feet	9.6 feet	10.9 feet
BI-270	Upstream Model Boundary	8.2 feet	9.7 feet	11.1 feet	8.2 feet	9.7 feet	11.1 feet

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

Excavated Detention: For this analysis, the pond was assumed to be excavated to a constant elevation of 0 feet. It was assumed that below this elevation the pond would not drain consistently and any additional excavation would only provide dead storage. This analysis assumes that the full 23 acres would be excavated, providing approximately 92 acre-feet of storage at a water surface elevation of 4 feet. **Table 7-16** shows predicted water surface elevations.

The results of the modeling indicate that for the modeled tide cycle excavated detention would have no impact on the overflow to Higgins Slough or on the upstream flooding culverts, although peak water surface elevations would be reduced along the flatter lower portion of the slough. The detention pond would provide approximately 80 acre-feet of storage for the 100-year future conditions storm event, which is approximately 16 percent of the total volume of water passing through the outfall for that event. This suggests that under the modeled tidal conditions the overflow to Higgins Slough and the flooding upstream are not controlled by volume, but by the capacity of the channel to handle peak flows at those locations.

Table 7-16: Big Indian Slough Excavated Detention Analysis Results							
SWMM Model Node	Approximate Location	Future Conditions Water Surface Elevation					
		Without Detention			With Excavated Level Detention		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet of Big Indian Slough	3.0 feet	3.2 feet	3.2 feet	2.0 feet	2.3 feet	2.5 feet
BI-90	Farm-to-Market Road	3.1 feet	3.4 feet	3.6 feet	2.4 feet	2.8 feet	3.2 feet
BI-120	Higgins Slough Overflow	4.4 feet	5.4 feet	6.1 feet	4.4 feet	5.4 feet	6.1 feet
BI-200	Higgins Airport Way	8.1 feet	9.6 feet	10.9 feet	8.1 feet	9.6 feet	10.9 feet
BI-270	Upstream Model Boundary	8.2 feet	9.7 feet	11.1 feet	8.2 feet	9.7 feet	11.1 feet
See **Figure 5-5** for node locations.							

4. Slough Capacity Analysis

Of particular interest in this basin is the capacity of the slough to handle additional flows. Currently under high-flow conditions, a portion of the runoff from Big Indian Slough flows into Higgins Slough. Because of the severity of flooding in Higgins Slough, the capacity of Big Indian Slough to handle all runoff from within the basin without overflowing to Higgins Slough is important. For this analysis, the overflow to Higgins Slough was removed to determine what impacts, if any, this would have on Big Indian Slough.

The modeling shows that under the modeled tidal condition removal of the Higgins Slough overflow would not increase the frequency of flooding in the system. **Table 7-17** and a **Table 7-18** compare the water surface elevations for existing and future conditions, respectively, at select points along the length of the slough if the Higgins overflow were to be eliminated.

Table 7-17: Higgins Overflow Removal Results - Existing Conditions							
SWMM Model Node	Approximate Location	Existing Conditions Water Surface Elevation					
		No Improvements			Higgins Overflow Removed		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet of Big Indian Slough	3.0 feet	3.2 feet	3.2 feet	3.0 feet	3.2 feet	3.2 feet
BI-90	Farm-to-Market Road	3.1 feet	3.4 feet	3.6 feet	3.1 feet	3.5 feet	3.6 feet
BI-120	Higgins Slough Overflow	4.1 feet	5.2 feet	5.8 feet	4.2 feet	5.4 feet	6.1 feet
BI-200	Higgins Airport Way	7.7 feet	9.3 feet	10.8 feet	1.1 feet	9.3 feet	10.9 feet
BI-270	Upstream Model Boundary	7.8 feet	9.4 feet	11.0 feet	7.8 feet	9.4 feet	11.1 feet
See **Figure 5-5** for node locations.							

Table 7-18: Higgins Overflow Removal Results - Future Conditions							
SWMM Model Node	Approximate Location	Future Conditions Water Surface Elevation					
		No Improvements			Higgins Overflow Removed		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet of Big Indian Slough	3.0 feet	3.2 feet	3.2 feet	3.0 feet	2.3 feet	3.2 feet
BI-90	Farm-to-Market Road	3.1 feet	3.4 feet	3.6 feet	3.2 feet	3.5 feet	3.7 feet
BI-120	Higgins Slough Overflow	4.4 feet	5.4 feet	6.1 feet	4.5 feet	5.6 feet	6.5 feet
BI-200	Higgins Airport Way	8.1 feet	9.6 feet	10.9 feet	8.1 feet	9.6 feet	11.0 feet
BI-270	Upstream Model Boundary	8.2 feet	9.7 feet	11.1 feet	8.2 feet	9.8 feet	11.2 feet
See **Figure 5-5** for node locations.							

Although the modeling shows that water surface elevations would increase if the Higgins Slough overflow were removed, the impact on the slough would not be significant. Flood elevations would increase, but the frequency of flooding would remain the same for both existing and future conditions.

This result is due to the fact that Big Indian Slough's outlet structure appears to have the capacity to convey significant flow—in excess of 400 cfs—with minimal head loss. This allows the slough to pass the peak flows without causing a significant backwater effect. Because the slough has levees in the lower reach to elevations in the range of 6.5 to 9.0 feet, the water surface elevation in the slough can increase to a level that allows the slough to flow at high tides, without causing flooding. The levees extend upstream to a point where ground elevations exceed flood stage, except in the case of the low culverts previously identified.

In order to verify this result, the capacity of the slough and its relationship to the outlet conditions was examined in more detail in two additional analyses:

- The software program CulvertMaster was used to check the capacity of the outlet culverts.
- A sensitivity analysis was conducted to determine if variations in the outlet boundary conditions would have a significant impact on the upstream water surface elevations.

a. Outlet Capacity Analysis

The software program CulvertMaster, developed by Haested Methods, was used to evaluate the capacity of the seven 48-inch CMP culverts at the outlet of the slough. Based on the survey data, the following parameters were used to model the culverts:

- The upstream and downstream invert elevations of the culverts are at approximately -3.5 and -3.6 feet, respectively.
- The culverts are approximately 5 feet long.
- A Manning's 'n' value of 0.024 was used.
- A constant tailwater elevation of 4.0 feet was used, 0.2 feet above the mean higher high water elevation used in the modeling.

Table 7-18 shows a rating table that was developed by varying the headwater elevation. The results indicate that the outlet culverts have a capacity of approximately 427 cfs with 1 foot of positive head (headwater elevation of 5 feet). This result is consistent with the XP-SWMM results, which indicate the outlet culverts have a high capacity even at high tide. In addition, if the tide rises higher than 4 feet, there is sufficient freeboard along the length of the slough to accommodate a water surface elevation that would generate sufficient head to keep the slough draining until the tide goes down.

Table 7-19: Big Indian Slough Outlet Culvert Rating Table

Headwater Elevation	Tailwater Elevation	Culvert Capacity
4.2 feet	4.0 feet	189 cfs
4.6 feet	4.0 feet	329 cfs
5.0 feet	4.0 feet	427 cfs
5.5 feet	4.0 feet	525 cfs

b. Water Surface Elevation Sensitivity Analysis

To estimate the sensitivity of the simulation to changes in tailwater conditions at the slough outlet, tidal elevations were adjusted by adding 3 feet to the tidal cycle. The tide cycled from a low tide of approximately 2.5 feet to a high tide of 6.8 feet. The model was only run for the 100-year, future conditions event. With this tide cycle, minor flooding was observed at Node BI-230 for the 100-year future conditions event. The slough appeared to have sufficient capacity to handle the 100-year event even for this condition.

These results indicate that there is enough capacity in the slough to pass the 100-year future conditions flood without causing any additional flooding in the slough. According to available survey data, the elevation of the top of the dike ranges from 6.5 to 9.0 feet.

Chapter 8
Capital Improvement Plan

The stormwater projects presented here are proposed for consideration to reduce or eliminated existing and/or future flooding conditions within the Bay View Watershed area. Some projects are simple, consisting of replacing or upsizing existing outfalls and 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 two primary sources, the Drainage District's property assessments and the Skagit County Drainage Utility. A breakdown of estimated financial contributions by these two 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 a very gross funding "goals". Detailed project cost estimates will need to be developed during the project planning and design phases.

All project costs are adjusted to January 2006 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 [CWCCIS]*[16] prepare by the US Army Corp of Engineers was used to adjust historical construction cost to January 2006 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.

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.

[16] *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, 2005)

2. Pump Station Construction Costs

Nine 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 2006 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.

$$\text{Pump Station Project Cost} = 0.0634 \times (\text{Capacity})^{0.8054}$$

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. Outfalls and Culverts

Proposed outfalls and culverts are estimated based on a schematic layout. Construction costs are estimated based on 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, 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 outfalls or culverts. The historical costs were adjusted to 2006 cost using the CWCCIS Cost Index.

4. Channels and Detention Ponds

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, 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 8-1 provides a proposed Capital Improvement Plan for planning, design and construction of the stormwater drainage facilities in the Bay View Watershed. 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 5 years has been approximately 4 percent for the *Civil Works Construction Cost Index*.

Table 8-1: Recommended Capital Improvements for the Bay View Watershed

Drainage Basin and Proposed Stormwater Capital Improvement	Project Cost Estimate	Projected Capital Imporvement Costs with Escalation					
		FY 2006	FY 2007	FY 2008	FY 2009	FY 2010	FY 2011
No Name Slough Basin							
Outfall Culvert Addition NN-C-Out	$ 100,000	$ 25,000	$ 26,000	$ 27,000	$ 28,000		
100-cfs Outfall Pump Station	$ 2,600,000			$ 541,000	$ 2,362,000		
84" Culvert Replacement NN-C-3	$ 21,000		$ 22,000				
54" Culvert Replacement NN-C-5	$ 14,000		$ 15,000				
32 ac-ft Marihugh Road Detention Pond	$ 1,257,000	$ 250,000	$ 260,000	$ 819,000			
Joe Leary Slough Basin							
Peth Property Bypass Channel	$ 820,000	$ 164,000	$ 171,000	$ 492,000			
300-cfs Outfall Pump Station	$ 6,300,000			$ 681,000	$ 709,000	$ 2,948,000	$ 3,066,000
60-cfs South Spur Pump Station	$ 1,700,000	$ 340,000	$ 1,414,000				
Little Indian Slough Basin							
Incease Channel Capacity Upstream of Farm-to-Market Rd	$ 45,000	$ 45,000					
48" Culvert Replacement LI-C-1	$ 56,000	$ 56,000					
48" Culvert Replacement LI-C-2	$ 23,000	$ 23,000					
Big Indian Slough Basin							
Ovenell Road Bypass Channel	$ 4,800,000	$ 800,000	$ 2,080,000	$ 2,163,000			
Modify Higgins Slough Bypass	$ 50,000	$ 50,000					
Totals	$ 17,786,000	$ 1,753,000	$ 3,988,000	$ 4,723,000	$ 3,099,000	$ 2,948,000	$ 3,066,000

1. No Name Slough Recommendations

All alternatives evaluated for this document were analyzed individually, without considering potential combinations of the alternatives. Before any project is carried forward, the hydraulic model should be updated to account for any other projects that have been implemented 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 alternatives are recommended for the No Name Slough basin:

a. Increased Pumping Capacity

Pumping remains the best option for reducing the flooding in the slough's lowland areas. A screw-type pump, where the water does not flow through an impeller or pump housing, would be the most ecologically sensitive and fish-friendly pump station. All subbasins would contribute to the outfall pump station.

b. Detention Pond at Marihugh Road

Marihugh Road appears to provide the best location for a detention pond. The ground is relatively flat and drainage from a large portion of the basin can be collected. Detention at Marihugh Road would reduce peak flows in No Name Creek, reducing erosion in the creek and sedimentation in the slough. Subbasins A-11b would contribute to a detention pond in this location.

c. Culvert Replacement

Replacement of all the undersized culverts is recommended. The recommended culvert sizes are based on the results of the hydraulic modeling. Additional local topography is needed as part of the final design to verify that the specified shape and material are appropriate for that location. Since it was not known which if any culverts would be replaced, the culverts were analyzed singly, assuming that no other culverts or structures had been updated. As part of the culvert design and construction, the hydraulic models should be updated with the most recent information. All subbasins would contribute to any additional outfall culverts. All subbasins, except subbasins A-4, A-5, and A-12, would contribute to Culvert NN-C-3. Only subbasins A-7 and A-8 would contribute to culvert NN-C-5.

Regional detention at subbasin A-11a is not recommended at this time. The County already requires new developments to match existing peak runoff; therefore the County should work with developers to ensure that these regulations are met. A new regional detention pond for subbasin A-8 will reduce the impact along the south stem from proposed development on the Port Property. Modifications to the existing detention pond on the Paccar Technical Center (subbasin A-7) may provide some benefit in reducing ditch erosion along Farm-to-Market Road.

2. Joe Leary Slough Recommendations

As with the other drainage basins in the Bay View Watershed area, 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, the number of alternatives that can reduce flooding are limited. The following alternatives are recommended:

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 bypass would circumvent the restriction along D'Arcy Road where the channel is confined by the road and could lower water surface elevations in portions of the slough by over 1 foot. All subbasins, except subbasins B-1a and B-1b, would benefit from the slough bypass.

b. Pump Station at the Outlet

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. Outlet Detention Pond

This is not a separate capital improvement, but something to consider when studying the Outlet Pump Station. Detention at the outlet was examined briefly and should be evaluated further as part of any outlet pump station project. Given the large volume of water that is generated during a peak event, considerable storage would be required to have an effect, and because ground elevations near the outlet are so low, a very large storage area would be needed. However, increased storage capacity in the slough could reduce the maximum pumping capacity needed to reduce flooding. If land were available for storage and if the construction cost and operation of a pump station at the outlet exceeds the available budget constraints, this option could be explored during the design phase of the outlet pump station as a way to reduce overall project costs.

d. Pump Station on South Spur Ditch

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.

A pump station with a reduced capacity could be constructed if the South Spur Bypass Channel is constructed. However, it may take several years to acquire the necessary right-of-way and environmental permits before this bypass channel can be constructed. A smaller pump station should be expandable in case the bypass channel does not became a reality.

Before any project is implemented, the hydraulic analysis 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.

3. Little Indian Slough Recommendations

Below 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 channel and culverts. Modeled results with upgrades to the channel and culvert capacity in the upper slough did not consider the effects of any existing detention. Therefore the result may be conservative.

Before any project is implemented, the analysis presented in this document should be updated to account for any new projects 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 alternative is recommended:

a. Culvert Replacement and Increase Channel Capacity

Culvert replacement and channel enlargement appears to be the most cost-effective alternative in reducing flooding upstream of Farm-to-Market Road. Although not specifically examined, 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 would 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, given the cost involved in a flood reduction project of the required scale.

4. Big Indian Slough Recommendations

Of the four basins modeled as part of this project, Big Indian Slough had the least information available to verify and calibrate the model. Therefore, it is recommended that before any projects are implemented in this basin, additional hydrologic data is collected to be used in model calibration or the model results be closely examined by the stakeholders who are most familiar with historical conditions in the basin.

Before any project is 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.

The following alternatives are recommended:

a. Ovenell Road Bypass Channel

The Ovenell Road Bypass Channel appears to have the ability to relieve the hydraulic bottleneck caused by the culvert pipes that exists within the existing drainage slough. According to the modeling results, the bypass channel would lower the water surface elevation in the upper portion of the existing slough by more than 1 foot and would eliminate flooding at most locations. If possible, additional topographic data collection is recommended to verify and/or calibrate the model in this area before the bypass is constructed. All subbasins, except subbasins C-2a and C-6, would benefit from construction of this new channel section. In addition, the project should be modeled at a variety of tidal conditions to better understand its effects.

b. Modify Higgins Slough Overflow

Since Higgins Slough currently experiences flooding, and because the lower reaches of Big Indian Slough appears to have the capacity to handle the additional flow, it is recommended that the this overflow be modified to control downstream flooding in Higgins Slough.

The Ovenell Road Bypass Channel was selected over the SR 20 Bypass Channel with field culvert replacement because it showed higher reductions in water surface elevations in the upper reaches of Big Indian Slough. The Ovenell Road Bypass Channel also directly serves the commercial areas within the Bay View ridge UGA. However, if right-of-way acquisition or other issue regarding the Ovenell Road Bypass Channel delays the project construction, then the SR 20 Bypass Channel is a viable alternative to consider.

Detention at the outlet of the slough is also not recommended at this time. While outlet detention would provide additional storage volume in the slough and might lower slough water surface elevations at the outlet, the benefits do not extend far enough upstream to impact any documented problem areas. Additional pumping may help reduce flooding during extended periods of high tide, however, this alternative was not analyzed and would likely require a pump station with a capacity of several hundred cfs. In fact, since the capacity of the pump station is far less than the outlet culverts, an analysis to determine if the pump station at the outlet is necessary seems warranted. Additional reduction in water surface elevation along the slough could be achieved by increasing hydraulic efficiency and increasing capacity in the middle and upper portions of the slough.

C. Stormwater Management Strategies

There are several stormwater management strategies that are recommended to be instituted in the Bay View Watershed. These strategies are intended to help ensure that the existing and future drainage facilities are adequately maintained so they will serve their purpose when a storm event occurs. The costs allocation in future years has been escalated to account for an estimated 4% per year inflation rate.

Table 8-2: Recommended Stormwater Management Strategies for the Bay View Watershed							
Stormwater Management Program Items	6-Year Program Estimate	Projected Stormwater Management Program Costs with Escalation					
		FY 2006	FY 2007	FY 2008	FY 2009	FY 2010	FY 2011
Entire Bay View Watershed							
Negotiate Interlocal Agreements with Drainage Districts	$ 50,000	$ 25,000	$ 26,000				
Develop Bay View Watershed Stormwater Coordination Plan	$ 25,000	$ 25,000					
Negotiate Floodway Easements	$ 25,000	$ 25,000					
Develop Bay View Watershed Stormwater Monitoring Plan	$ 100,000	$ 50,000	$ 10,000	$ 11,000	$ 11,000	$ 12,000	$ 12,000
Revise, Expand and Update Hydraulic Model	$ 100,000		$ 21,000	$ 22,000	$ 22,000	$ 23,000	$ 24,000
No Name Slough Basin							
Slough and Channel Cleaning and Maintenance	$ 54,000	$ 9,000	$ 9,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000
Pump Station Operation and Maintenance	$ 80,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000	$ 23,000	$ 24,000
Joe Leary Slough Basin							
Slough and Channel Cleaning and Maintenance	$ 216,000	$ 36,000	$ 37,000	$ 39,000	$ 40,000	$ 42,000	$ 44,000
Pump Station Operation and Maintenance	$ 50,000		$ 10,000	$ 11,000	$ 11,000	$ 12,000	$ 12,000
Little Indian Slough Basin							
Slough and Channel Cleaning and Maintenance	$ 54,000	$ 9,000	$ 9,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000
Big Indian Slough Basin							
Slough and Channel Cleaning and Maintenance	$ 180,000	$ 30,000	$ 31,000	$ 34,000	$ 38,000	$ 44,000	$ 54,000
Pump Station Operation and Maintenance	$ 60,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000	$ 12,000	$ 12,000
Totals	$ 994,000	$ 229,000	$ 173,000	$ 159,000	$ 164,000	$ 190,000	$ 204,000

1. Negotiate Interlocal Agreements with Drainage Districts

The County Commissioners should authorize the County Drainage Utility to negotiate interlocal agreements with the Dike and Drainage District No. 12, Drainage District No. 14, and Drainage District No. 19. These interlocal agreements would layout the framework for cost sharing on capital improvement projects, maintenance responsibilities between the County and the Drainage Districts, and reimbursement costs for maintenance of joint owned facilities. It is anticipated that the County would hire a financial consultant to assist with issues such as buy-in charges, impact fees, and debt financing.

2. Develop the Bay View Watershed Stormwater Coordination Plan

Several stakeholders, specifically the Drainage Districts, expressed an interest in developing a framework that facilitates an ongoing dialog regarding stormwater issues for new developments within the Bay View Watershed. This coordination element would take place during the permit review stage of a proposed project and would involve the developer, the Skagit County Planning & Development Services, the Skagit County Drainage Utility, and the impacted Drainage District.

3. Negotiate Floodway Easements

A floodway easement is a management tool that can be examined for application in any of the Bay View drainage basins. A floodway easement is a negotiated agreement between a drainage control party, such as the Skagit County Drainage Utility or the Drainage District, and a property owner. The floodway easement would describe the potential area that may be flooded during 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 facilities. Flooding easements may also be used as temporary measures to provide financial protection to property owners now while storm drainage improvements are studied, designed and constructed.

4. Develop the Bay View Watershed Stormwater Monitoring Plan

One characteristic of this stormwater study is that there is no physical rainfall data with corresponding channel flow rate data in order to calibrate the hydraulic model. A Stormwater Monitoring Plan would describe the framework for installation of stormwater measuring equipment and ongoing monitoring.

5. Revise, Expand and Update the Hydraulic Model

Four hydraulic models, one for each of the four drainage basins, were developed as part of this stormwater study. The hydraulic models were used to evaluate stormwater drainage facility options. As drainage facilities are constructed and physical stormwater runoff data is collected, the hydraulic models will need to be revised, expanded and updated. The hydraulic models can then be used to evaluate the effectiveness of constructed drainage facility as well as examine additional drainage facilities.

6. Slough and Channel Cleaning and Maintenance

Slough and channel cleaning and maintenance is an essential element in reducing the flooding potential within the drainage basins. These sloughs and channels are the major drainage facilities for properties both inside and outside the Drainage District Boundaries. In the past sloughs and channels have been cleaned and maintained solely by the Drainage Districts. The Skagit County Drainage Utility has a responsibility to financially contribute to the cleaning and maintenance of the sloughs and channels. Each Drainage District needs to enter into an interlocal agreement with Skagit

County to layout the framework for reimbursement of slough and channel cleaning and maintenance costs.

7. Pump Station Operation and Maintenance

There are two exiting stormwater pump stations and proposals for construction and/or expansion of additional pump stations. These pump station serve properties both inside and outside of the Drainage District boundaries. In the past existing pump station operation and maintenance has been performed solely by the Drainage Districts. Skagit County has a responsibility to financially contribute to the operation and maintenance of the existing and future pump stations. Each Drainage District need to enter into an interlocal agreement with Skagit County to layout the framework for reimbursement of pump station operation and maintenance costs.

8. NPDES Phase II Permitting

The issuance of a NPDES General Permit for Municipal Storm Sewers (Phase II) in late 2006 or early 2007 will increase the rules and regulations local governments must follow concerning the water quality of the stormwater in their drainage systems. This will have impacts, including financial, on Skagit County, the Drainage Districts, the City of Burlington, and the Port of Skagit, however, the extent of those impacts are not known at this time.

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Appendices

Chapter 1
Executive Summary

The Bay View area has developed with an inadequate understanding of the impacts of stormwater runoff. As a result, several stakeholders within the watershed have expressed concerns regarding the quality and quantity of stormwater being discharged to the adjacent sloughs and Padilla Bay. The purpose of the *Bay View Watershed Stormwater Management Plan* is to: 1) inventory stormwater drainage facilities within the watershed; 2) develop stormwater hydraulic models in order to understand current and future drainage impacts; 3) propose stormwater facility improvements; and 4) propose stormwater management strategies to manage drainage within the Bay View area and to reduce farmland flooding. Skagit County funded the preparation of this Plan from its Drainage Utility fund.

The Bay View Watershed Stormwater Management Planning 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 and around the Skagit Golf and County Club. 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 the Skagit County, dike and drainage districts, Port of Skagit County, 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 area. These documents include the *Padilla Bay/Bay View Watershed Non-Point Action Plan*, the *Port of Skagit County Stormwater Management Master Plan*, the report entitled *Hydrologic and Hydraulic Model of the No Name Slough Drainage,* the *Bayview Ridge Subarea Plan*, the *Joe Leary Slough Drainage Study*, and the *Inventory and Evaluation of Tide Gates and Pump Stations related to Alternatives #5 and #7 of the Skagit River Flood Damage Reduction Feasibility Study.* 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 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. The model results indicated that there are areas of potential flooding along each of the four major drainage sloughs. Conceptual stormwater drainage improvements were developed and evaluated that could correct capacity limiting facilities. Potential drainage facility improvements that were evaluated included the following:

- Enlarging and regrading slough channels,
- Regional detention,
- Stormwater pump stations,
- Bypass channels,
- Increasing levee heights, and
- Upsizing culverts.

The Capital Improvement Plan of the proposed drainage facilities improvements is presented in **Chapter 8** for each drainage basin. A summary of the proposed improvements are presented below.

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	Increase Pump Station Capacity	$ 2,600,000
	32 ac-ft Marihugh Road Detention Pond	$ 1,257,000
	Various Culvert and Outfall Additions and/or Replacement	$ 135,000
Joe Leary Slough Basin	Peth Property Bypass Channel	$ 820,000
	300-cfs Outfall Pump Station	$ 6,300,000
	60-cfs South Spur Pump Station	$ 1,700,000
Little Indian Slough Basin	Culvert Replacement and Increase Channel Capacity	$ 124,000
Big Indian Slough Basin	Ovenell Road Bypass Channel and Higgins Slough Overflow Modifications	$ 4,850,000
Total Capital Improvement Cost Estimate		$ 17,786,000

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 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, bioswales, and constructed wetlands.

Chapter 2
Introduction

The Bay View Watershed is located in the westerly portion of Skagit County, west of the City of Burlington. A Vicinity Map of the Bay View Watershed Management Planning Area is shown on **Figure 2-1.** The Vicinity Map shows the outline of the Study Area that was used for this Plan.

A. Purpose and Scope

The Bay View Watershed has developed with an inadequate understanding of the impacts of stormwater runoff. As a result, several stakeholders within the area have expressed concerns regarding the quality and quantity of stormwater being discharged to the adjacent sloughs and Padilla Bay. The purpose of the *Bay View Watershed Stormwater Management Plan* is to: 1) inventory stormwater drainage facilities within the watershed; 2) develop stormwater hydraulic models in order to understand current and future drainage impacts; 3) propose stormwater facility improvements; and 4) propose stormwater management strategies to manage drainage within the Bay View area and to reduce farmland flooding.

B. Stakeholders Purpose and Objectives

There are several entities that have a stake in stormwater drainage planning in the Bay View area. These entities are discussed below. The stormwater planning objectives of each stakeholder is also discussed.

1. Skagit County

Skagit County Surface Water Management (herein called the County) is the lead agency for the *Bay View Watershed Stormwater Management Plan* (herein called the Plan); providing project management, and project funding. The County provides representation for the residents and property owners within the Study Area. County government is developing the Plan to provide a means to minimize present and future stormwater impacts to the citizens of the County and their properties located within the Study Area. The County has many interests in the Study Area including development of a programmatic Environmental Impact Statement [EIS] for the proposed non-municipal Urban Growth Area (UGA). In addition, a large portion of the project area is located in the County's Drainage Utility service area and much of the stormwater from the Drainage Utility service area is discharged to adjacent Drainage District facilities. Therefore, the County has a vested interest in working with the Drainage Districts to mitigate potential impacts from stormwater runoff.

2. Drainage District No. 14

Drainage District No. 14 owns and maintains drainage ditches and outfalls in the farmland areas north and northeast of the Study Area. A portion of the service area for Drainage District No. 14 is shown on **Figure 2-2.** The District's primary drainage channel is Joe Leary Slough, which forms the north and northeast boundary of the Study Area. Joe Leary Slough discharges by gravity to Padilla Bay through tide gates located downstream from Bay View-Edison Road. Drainage from portions of the Study Area directly impact stormwater conveyance within Joe Leary Slough and its outfall to Padilla Bay. Many of the District's stormwater management objectives are presented in the *Joe Leary Slough Drainage Study*, which is discussed later in this chapter.

3. Drainage District No. 19

Drainage District No. 19 owns and maintains drainage ditches and outfalls in the farmland areas south and southeast of the Study Area. A portion of the service area for Drainage District No. 19 is shown on **Figure 2-2.** The District's primary drainage channels are Little Indian Slough and Big Indian Slough, which forms the south and southeast boundary of the Study Area, and Higgins Slough, which is located south and southwest of the Study Area. Little Indian Slough discharges by gravity to Padilla Bay through tide gates. Big Indian Slough discharges to Padilla Bay through tide gates and/or a pump station. Drainage from portions of the Study Area directly impact storage or conveyance within the sloughs and their outfall into Padilla Bay.

The District's objectives for stormwater management within the Study Area are twofold. First, the District supports measures that reduce erosion and sedimentation to and within its stormwater conveyance systems, which reduces its maintenance requirements. Second, the District supports measures that reduce peak stormwater runoff, which has the potential to overload its existing conveyance capacities resulting in localized lowland flooding.

4. Dike and Drainage District No. 8

On February 3, 2004, the property owners within Dike and Drainage District No. 8 voted to be incorporated in Dike District No. 12. The incorporation process was completed in August, 2004. Up until the vote, Dike and Drainage District No. 8 maintained a levee along Padilla Bay and several field ditches.

A notable feature within this District is the Padilla Bay Trail. This trail extends a distance of 2.2 miles along the top of the levee system adjacent to Padilla Bay. Skagit County Parks and Recreation Department and the Department of Ecology, as part of the Padilla Bay National Estuarine Research Reserve, maintain the trail.

5. Dike and Drainage District No. 12

As a result of incorporating Dike and Drainage District No. 8 in 2004, Dike District No. 12 renamed itself to Dike and Drainage District No. 12. This District historically owned and operated dikes, levees and outfalls along portions of the Skagit River and Padilla Bay. The drainage service area for Dike and Drainage District No. 12 is shown on **Figure 2-2.** The dike service area for Dike and Drainage District No. 12 is shown on **Figure 2-3.** The District's primary drainage channel is No Name Slough, which discharges to Padilla Bay through gravity tide gates and/or pump stations.

Specific responsibilities of Dike and Drainage District No. 12 are to maintain 1) the dike system along the north side of the Skagit River from the east end of the City of Burlington to the community of Avon and 2) levees along a portion of Padilla Bay and connecting sloughs. The purpose of these dike and levee systems is to protect properties from flood and seawater damage. Now, an additional responsibility is maintaining the stormwater drainage facilities within the No Name Slough basin.

6. Port of Skagit County

The Port of Skagit County (herein called the Port) owns approximately 1830 acres of property in the Bay View area. The boundaries for the Port are shown in **Figure 2-2.** The Port's primary purpose is to create jobs. In the Bay View area this is accomplished through two means: 1) to operate the Skagit Regional Airport; and 2) to develop light industry at the Bay View Business and Industrial Park. In the past, the Port has taken measures to reduce the impact of stormwater runoff from its property. Many of the Port's stormwater management objectives are presented in the *Port of Skagit County Stormwater Management Master Plan*, which is discussed later in this chapter.

7. City of Burlington

Most of the city limits of Burlington lie outside of the Study Area. There are currently ten homes along Peterson Road that contribute storm water drainage to Drainage District 19. A portion of Burlington's commercial area within the northern portion of the city does drain into the Maiben Ditch. The City and Drainage District No. 14 have a contractual arrangement where the City collect drainage utility fees from its commercial property owners in this area and transfers them on to Drainage District No. 14.

The City of Burlington currently provides sanitary sewer service to a portion of the Study Area. All commercial and light industrial developments within the Urban Growth Area will have sanitary sewer service. In addition, transportation impacts from development within the Study Area will effect transportation planning within the City; therefore, the City has an interest in development within its sewer service area. None of the existing or proposed stormwater drainage facilities within the Study Area impact stormwater facilities within the City of Burlington.

8. Large Tract Land Owners

There are several large tract landowners within the Study Area. It is anticipated that some of these large tract landowners will desire to someday develop their property. The designation of the Urban Growth Area will limit the development opportunities for those property owners outside of the UGA. Landowners inside the UGA will have more opportunities to develop their property and, therefore, will require increased attention to stormwater planning to accommodate anticipated growth.

9. Washington State Department of Ecology

The Washington State Department of Ecology is actively involved in the research and preservation of Padilla Bay through the Padilla Bay National Estuarine Research Reserve. The Reserve owns and manages the majority (11,000 acres) of Padilla Bay, including approximately 8,000 acres of eelgrass meadow. Padilla Bay is the receiving water for all stormwater drainage from the Study Area. The Reserve has been involved in drainage and stormwater quality issues of No Name Slough and Joe Leary Slough.

The Reserve also owns approximately 200 acres of land within the Study Area, primarily in and around the vicinity of the Padilla Demonstration Farm at the mouth of No Name Slough and the Breazeale-Padilla Bay Interpretive Center.

The Reserve's goals regarding stormwater management are to protect the natural resources of Padilla Bay and sustain agriculture on the adjacent flood plain by encouraging development and utility infrastructure that will facilitate proper stormwater controls.

10. Washington State Department of Fish and Wildlife

The Washington State Department of Fish and Wildlife is responsible for protecting and enhancing fish and wildlife habitats. Some storm drainage facilities, such as detention ponds and sloughs, can provide habitat for waterfowl. Some of the larger sloughs support various fish species. The Department of Fish and Wildlife is interested in the type and location of any new storm drainage facilities proposed by the *Bay View Watershed Stormwater Management Plan.* Specific objectives for the Washington State Department of Fish and Wildlife include maximizing and enhancing anadromous fish spawning and rearing habitat, reducing erosion and sedimentation, and minimizing impacts to wetlands.

11. Washington State Department of Transportation

The Washington State Department of Transportation [WSDOT] has jurisdiction over the design, operation and maintenance of State Route 20 [SR 20], which extends along the southern boundary of the Study Area. WSDOT is in the planning stages for widening SR 20 from the intersection with Memorial Highway [SR 536] to Interstate 5 in Burlington. The proposed widening will expand the highway from its current 2 lanes to four lanes. Stormwater mitigation measures for the proposed widening will need to be coordinated with proposed stormwater drainage improvements in this area.

12. Federal Aviation Administration

The Airport Planning Division of the Federal Aviation Administration [FAA] is responsible for providing guidance to airport operators regarding design and operation standards. With regards to stormwater management, the FAA has two concerns, both having to do with the location of detention ponds. First, the FAA has restrictions on what can be placed in the various flight path zones that are established around an airport. Second, the FAA also does not want any type of stormwater facility, such as a detention pond, to be located in an area that might attract waterfowl into the normal fight path of aircraft. The FAA is interested in the proposed location of future detention ponds around the Skagit Regional Airport. The FAA is opposed to the development or enhancement of wildlife habitat within 10,000 feet of runways. In addition, the FAA desires to minimize open water conditions within the Runway Protection Zone. To minimize open water conditions, the FAA requests that detention ponds be designed to drain completely within 48 hours after a storm event.

C. 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 area. The following are abstracts and summaries from these related documents.

1. Padilla Bay/Bay View Watershed Non-Point Action Plan[1]

In 1995 the Padilla Bay/Bay View Watershed Management Committee and Skagit County Department of Planning and Community Development prepared the *Padilla Bay/Bay View Watershed Non-Point Action Plan*. The committee included representatives of local residents, government agencies, environmental groups, members of the agricultural community, timber industry, Native American tribes, and other affected or interested parties. The mission of the Watershed Management Committee was to develop a Watershed Action Plan for the management of non-point source pollution in the Padilla Bay watershed as defined by the Washington State Administrative Code 400-12.

The Committee looked at several sources of potential contamination within the study area and recommended measures to control non-point pollution. The goal was to develop and implement a source control strategy for various non-point pollution sources. The Plan provides thirteen source control recommendations for stormwater drainage and erosion control. Some of these recommendations included modifications to existing county ordinances, the implementation of Best Management Practices [BMPs], and the restoration of existing drainage facilities that were contributing to pollution of Padilla Bay.

2. Port of Skagit County Stormwater Management Master Plan[2]

The *Stormwater Management Master Plan* was prepared for the Port of Skagit County by David Evans and Associates, Inc. and was completed in 1998. The Stormwater Management Master Plan is a comprehensive plan document that covers the entire area served by the Port of Skagit County, including the Bay View Business and Industrial Park and the Skagit Regional Airport.

The *Stormwater Management Master Plan* presents a review of existing stormwater facilities, including pipes, culverts, ditches, ponds, and channels. Capacities of existing and anticipated future stormwater conveyance facilities were evaluated using a hydraulic computer model. In addition to stormwater quantity calculations, stormwater quality characteristics are also addressed.

Based on the hydraulic analysis of existing stormwater facilities, water quality characteristics, and future developed conditions, a capital improvement plan was prepared which recommended specific stormwater capital improvements over the next few years. The primary emphasis of the capital improvement plan was to construct a series of regional detention facilities along Higgins Road and associated stormwater conveyance infrastructure.

[1] *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).

[2] *Stormwater Management Master Plan for the Bay View Business and Industrial Park and Skagit Regional Airport*, prepared by David Evens and Associates, Inc. (October 1998).

3. Hydrologic and Hydraulic Model of the No Name Slough Drainage[3]

The *Hydrologic and Hydraulic Model of the No Name Slough Drainage* was prepared for the Padilla Bay National Estuarine Research Reserve by Northwest Hydraulic Consultants in November 2000. The purpose of the study was to develop a hydraulic model to characterize the existing hydrology of the watershed and to allow future analysis of various land use scenarios and operational alternatives. A computer model was developed for the hydraulic modeling effort using the Hydrologic Simulation Program-Fortran (HSPF) model developed by the U.S. Environmental Protection Agency. The results of the modeling task provided some indication of the amount of runoff generated in the basin, discharge volumes to Padilla Bay, and frequency of flooding in the lower reaches near the levee and tide gates.

4. Bayview Ridge Subarea Plan[4]

Under the Growth Management Act, government entities are required to establish Urban Growth Areas [UGAs] and to set aside other areas as rural. Skagit County and the Port of Skagit County desire to establish an urban growth area around the Skagit Regional Airport. The UGA boundary and proposed land use is shown on **Figure 2-4.**

The purpose of this urban growth area is to develop a self-sufficient urban community to insure the continued viability of the Skagit Regional Airport. The *Bayview Ridge Subarea Plan*, currently being prepared by Reid-Middleton along with input from Skagit County, City of Burlington, and the Port of Skagit County, is intended to specifically address detailed community planning issues in the Bayview Ridge Urban Growth Area.

The *1997 Skagit County Comprehensive Plan* provides general guidelines for community development within Skagit County. The *Bayview Ridge Subarea Plan* coordinates and provides consistency with the *1997 Skagit County Comprehensive Plan* while providing detailed guidelines to facilitate future growth within the UGA.

The *Bayview Ridge Subarea Plan* documents in detail the critical planning characteristics within the area. These characteristics include the following:

- Land use
- Business and industrial development
- Commercial and community centers
- Housing
- Transportation elements
- Capital facilities elements
- Utilities elements

[3] *Padilla Bay Hydrology – Hydrologic and Hydraulic Model of the No Name Slough Drainage*, prepared by Northwest Hydraulic Consultants (November 2000).

[4] *Bayview Ridge Subarea Plan,* prepared by Reid-Middleton (March 2005 Draft).

- Parks, recreation, and open space
- Natural environment
- Essential public facilities

Two of the characteristics, specifically land use and the natural environment characteristics, will have a direct impact on stormwater drainage planning in this area.

5. Joe Leary Slough Drainage Study[5]

A letter report dated January 29, 2002, entitled *Joe Leary Slough, Maiben Road Ditch and South Spur Ditch Drainage Analysis and Findings* was prepared for Drainage District No. 14 by Semrau Engineering & Surveying. The letter report presents the findings from a study that 1) inventoried and surveyed drainage structures within the District's boundaries, 2) delineated and characterized the drainage subbasins, and 3) presented the results of a preliminary hydraulic model for the drainage basin.

The hydraulic model identified several deficiencies in the stormwater conveyance systems. The capacity of the Joe Leary Slough outfall is approximately 900 cfs at mean tide, but several of the upstream culverts are limited to approximately 330 cfs. Capacity restrictions are also present on the South Spur Ditch. Several recommendations were presented in the letter report and are summarized as follows:

- Berms at the Joe Leary Slough outfall to raise the stored water surface elevation and increase outfall capacity,
- Increase the conveyance capacity through upstream culverts in Joe Leary Slough,
- Provide additional storage at the Joe Leary Slough outfall,
- Provide additional storage along the South Spur ditch between Josh Wilson Road and Joe Leary Slough,
- Investigate if Bay View Ridge properties should mitigate for runoff volumes instead of peak discharge flow rates.

6. Tide Gate and Pump Station Study[6]

This Study, entitled *Inventory and Evaluation of Tide Gates and Pump Stations related to Alternatives #5 and #7 of the Skagit River Flood Damage Reduction Feasibility Study,* was performed in conjunction with the Skagit River Flood Protection/Salmon Restoration Project. The first draft was completed in November 2002. Skagit County and the U.S. Army Corps of Engineers selected two preferred alternatives for conveying the 100-year flood event in the Skagit River. One alternative, known as Alternative 5, is a proposal to set back the existing levees along the Skagit

[5] *Joe Leary Slough, Maiben Road Ditch and South Spur Ditch Drainage Analysis and Findings*, letter report prepared by Semrau Engineering & Surveying, PLLC (January 29, 2002).

[6] *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).

River. The concept behind Alternative 5 is that the 100-year flood event would then be contained within the river channel. The project would involve the setting back of levees from Burlington through Mount Vernon and downstream to the mouth of the North Fork and South Fork of the Skagit River.

Another alternative, known as Alternative 7, involves the construction of a 1600 to 2000-foot wide bypass channel that would be used to convey peak stormwater flows from the main river channel. This new channel would have a capacity to divert up to 80,000 cfs and would discharge into the Swinomish Channel instead of Skagit Bay.

Both proposed alternatives would greatly impact several storm drainage facilities within the Skagit Valley. The purpose of this study was fourfold:

- Provide an inventory, including location and condition, of existing tide gates, culverts, and pump stations within the project "footprint" of the two alternatives.
- Identify new and additional storm drainage facilities that may be required by either alternative.
- Identify those storm drainage facilities that may require modification and/or relocation.
- Identify the nature and condition of any potential habitat landward of the existing storm drainage facilities.

7. No Name Slough Watershed Characterization Report[7]

This report, completed in May 2004, was prepared by the Skagit Conservation District and the Padilla Bay National Estuarine Research Reserve. The objectives of the study were to 1) prepare a detailed characterization of existing hydrology and water quality, 2) provide public education and outreach, and 3) propose a comprehensive collection of projects to improve water quality, provide more consistent stream flows, and support fish and wildlife habitat. Proposed projects include wetland enhancement with stormwater storage components, conversion of ditches to bioswales, tree buffer installations, septic tank replacement, detention pond modifications, culvert replacement, and slough channel dredging.

[7] *No Name Slough Watershed Characterization,* prepared by the Skagit Conservation District and the Padilla Bay National Estuarine Research Reserve (May 2004).

Chapter 3
Study Area

The *Bay View Watershed Stormwater Management Plan* area (herein referred to as the Study Area) is primarily bound to the west by Padilla Bay, to the south and southeast by Big Indian Slough, and to the north and northeast by Joe Leary Slough and its tributaries. The Study Area is approximately 11,277 acres. **Figure 3-1** is an aerial photograph of the Bay View area.

Table 3-1 summarizes the land use designations within the Study Area.

Table 3-1: 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 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.

A. Land Use and Development

Existing development varies within the Study Area. **Figure 3-2** provides an indication where development has occurred. Prominent developments in the Bay View 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 cluster developments.

1. Historical Development

Some history regarding past development is presented below[8].

[8] *Bay View: Pioneer City on the Sound – An Oral History,* Don Eklund, Occasional Paper #22, Center for Pacific Northwest Studies, Western Washington University, 1987.

- The Bay View area was solely inhabited by native tribes until about the middle of the 19th century. The native inhabitants did not significantly modify the existing drainage systems, and likely had little impact on stormwater runoff and discharge.

- Some of Skagit County's earliest pioneers established homesteads in the Bay View area in the late 1840s. The rural village of Bay View was named by William Mckenna, who platted the original town site in 1884. The rural village of Fredonia was platted in 1890.

- Samuel Calhoun and Michael J. Sullivan are widely accepted to be the first whites to establish permanent settlements in (what is now) Skagit County in 1867. They are also thought to be the visionaries for constructing dikes around the salt flats, a process which allowed reclamation of the tidelands for growing crops. Dike construction changed the natural stormwater drainage. For example, Joe Leary Slough, prior to the construction of dikes, was a fish bearing stream large enough to raft logs.

- By 1871, reclaimed tidelands were producing barley, oats, hops and potatoes. The biggest challenge faced by the settlers that were farming the reclaimed tidelands was to keep the dikes from breaking, which was an ongoing problem. At the end of World War II, modern machinery allowed for the revamping and extending of dikes, broadening them at their base and building some to a height of eight to nine feet, as they are today.

- Bay View State Park overlooks Padilla Bay and offers picnic tables, a playground, and camping. The Skagit County Agricultural Association, with the understanding that it would become a State Park, donated the original portion of the Bay View State Park to Washington State in 1925. Additional parcels were acquired up through 1968. The park site was formerly a baseball field and racetrack.

- Development at the Skagit Regional Airport site began in 1933 with a small airport that was constructed by the Public Works Administration and the Works Progress Administration. The present runway and taxiway system was constructed in 1943 by the United States Navy as an alternative airfield for the Whidbey Islands Naval Air Station. The airfield was transferred to the Skagit Board of County Commissioners in 1958, and later transferred to the Port Districts of Anacortes and Skagit County. In 1975 the sole ownership of the airport property was transferred to the Port of Skagit County.

- Suburban type residential development occurred in the eastern portion of the Bay View Ridge area with the extension of sanitary sewer service from the City of Burlington, which started in the 1970s. Sanitary sewer service has steadily expanded since that time. The City of Burlington has recently completed a new sanitary sewage lift station near the intersection of Peterson Road and Avon-Allen Road with a new forcemain extending to its wastewater treatment facility.

- Padilla Bay's Breazeale Interpretive Center overlooks Padilla Bay. The property was obtained from the Breazeale family in 1973 and the Interpretive Center opened in 1982. The recent expansion was completed in 2005. The Interpretive Center overlooks Padilla Bay and provides interpretive exhibits, a lecture hall and research facilities. The old Breazeale family barn and house are now used as a laboratory with overnight quarters for visiting researchers and offices for staff.

- In 1989, the Skagit County Parks and Recreation Department and the Department of Ecology (Padilla Bay National Estuarine Research Reserve) began discussions with Dike and Drainage District No. 8 regarding developing a 2¼-mile dike trail along the southeastern shore of Padilla Bay. Planning and construction grants were obtained from the Aquatic Lands Enhancement Account (Department of Natural Resources), Skagit County Pathway Funds, and Ecology/NOAA Section 315 Funds. The Padilla Bay Trail was opened in 1990.

2. Skagit County Planning Efforts

The Skagit County Comprehensive Plan describes the general development patterns that are proposed within all areas of the county. A map showing the land use designations in the Bay View area is presented in **Figure 3-3.**

The *Bayview Ridge Subarea Plan* provides a detailed discussion regarding development within the proposed Urban Growth Area. The concept of the *Bayview Ridge Subarea Plan* supports the existing urban land use patterns. The overall intent of the *Bayview Ridge Subarea Plan* is to create a rural cohesive community which functions as a small city, providing for an urban level of development along with an urban level of services.

Future land use within the Bay View Ridge Subarea will build on the existing land use pattern and will include residential, commercial, business/industrial, and park/open space related uses. Land use prohibitions in and around the Skagit Regional Airport will limit some use options.

The highest concentration of residential development has occurred along the east side of the Study Area within the UGA, most of which occurred through large tract plats. There is still some potential for higher density residential plats within the UGA along Peterson Road east of the Skagit Regional Airport; however, approximately 70 percent of the residential zone areas are already developed. Due to constraints of the airport safety zones, future densities are limited to four dwelling units per acre. Lot sizes are to be between 8,400 and 10,890 square feet.

Outside of the UGA, residential development will be limited due to the rural designation. Proposed residential developments outside of the UGA will be required to be clustered so as not to preclude future urban development.

B. Natural Features

Prominent natural features include Padilla Bay, No Name, Joe Leary and Big Indian Sloughs, Bay View Ridge area, and the alluvial surrounding farmland.

Padilla Bay is an estuary at the saltwater edge of the large delta of the Skagit River. It is about eight miles long and three miles across.

Most of Padilla Bay's watershed (23,000 acres) is low flat delta that is now farmland. In the late 1800's, the marshes of the Skagit River delta were drained and levees were constructed. Portions of the Skagit River were diverted and are now confined to channels that empty into Skagit Bay leaving Padilla Bay "orphaned" from the river that formed its mud flats. Today, Padilla Bay's freshwater comes from a number of agricultural sloughs. The Swinomish Channel connects Padilla Bay to Skagit Bay located to the south. Padilla Bay is bordered on the east and south by levees that protect adjacent farmland from flooding. To the north and west are the rocky San Juan Islands in northern Puget Sound.

The surrounding alluvial farmland is within the floodplain of the Skagit Valley. Much of this area was reclaimed tidelands through the construction of dikes and drainage sloughs. For this reason, this area will be more susceptible to flooding. Development within the floodplain has been limited through development restrictions, zoning, and other farmland protection measures. Farming activities are expected to continue to dominate land use activities within the floodplains surrounding the Bay View Ridge area.

1. Topography

The Bay View Ridge area is situated east of Padilla Bay. This glacial terrace is elevated 220 feet above the surrounding floodplain. The physical features within the Bay View Ridge area range between gentle sloping terrain and steep hillsides. Undeveloped areas tend to have a mix of trees stands and opened fields or meadows.

2. Soil

The general classification for soils in the Bay View Ridge area are described as Bow-Coveland-Swinomish and are characterized by "moderately deep and very deep, somewhat poorly drained and moderately well drained, level to steep soils; on terraces, plains, and hills"[9]. The predominated soil classification in the area is Bow Gravelly Loam. The soils have a high percentage of fine-grained material, are typically saturated with poor percolation yields, and 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 presence of a perched water table between November and May.

3. Climate

Climate data for the Bay View area was derived from data published by the National Climate Data Center, which collects climate data from National Oceanic and Atmospheric Administration.

The average rainfall in the Bay View area 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 and less rainfall 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).

[9] Soil Survey of Skagit County Area, Washington, prepared by the United States Department of Agriculture – Soil Conservation Service in cooperation with the Washington State Department of Natural Resources and the Washington State University, Agriculture Research Center (September 1989).

C. Critical Areas

Critical areas include aquifer recharging areas, flood hazard areas, geologically hazardous areas, and fish and wildlife habitat conservation areas. Some of these critical areas, such as the wetlands within the Bay View Business and Industrial Park, have been delineated. However, most critical areas within the Bay View area have not been precisely identified and their exact locations are not accurately mapped. Skagit County, like many other jurisdictions, relies on critical area site assessments performed by development project proponents.

1. Aquifer Recharging Areas

The Bay View area does not contain any identified critical aquifer recharged areas. Development within aquifer recharge areas may reduce groundwater infiltration of stormwater. Some areas in the north portion of the Bay View area 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 area that may also rely on groundwater wells for their source of water.

2. Flood Hazard Areas

The Bay View area 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.

3. 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 area. The *Bayview Ridge Subarea Plan* provides a detailed discussion regarding wetlands in the Bay View area. Much of the discussion regarding wetlands presented in this Plan is derived directly from the *Bayview Ridge Subarea Plan*. The map showing the wetlands in the Bay View area is presented in **Figure 3-4.**

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.

Many wetlands and associated buffers have been identified in the Bay View area. Some wetlands in the area have been fragmented or isolated by existing development, while others have been hydrologically modified by uncontrolled or poorly controlled stormwater runoff. In some cases this has led to the support of primarily invasive and undesirable plants and animal species.

Within most of the Bay View area, 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 area outside of the Port ownership. The precise boundaries of these wetlands are not known and would be delineated by project proponents as specific development projects are proposed.

4. Port of Skagit County Wetlands Management Plan

The Port of Skagit County 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 property.

5. Priority Habitat

The Priority Habitats and Species (PHS) Program, administered by the Washington State Department of Fish and Wildlife, provide 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 3-5 shows the priority habitat within the Bay View Ridge area for the bald eagle and fish that has been established by the Washington State Department of Fish and Wildlife.

Chapter 4
Storm Drainage Facilities

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 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. For modeling purposes, the Indian Slough Basin was also divided into two separate basins, the Little Indian Basin and the Big Indian Basin. 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 units.

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** under Basin A**.**

Drainage District: Dike and Drainage District No. 12.

Primary Drainage Facility: No Name Slough.

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

Ponds and Detention Facilities: There are 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 is being 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.

Culverts and Pipes. Storm water conveyance is primarily through roadside ditches and culverts. There are some 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

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. One notable exception is a portion of the Bay View Ridge proposed UGA that 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** under Basin B**.**

Drainage District: Drainage District No. 14.

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

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

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 County 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 County 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 County 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 County 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 result from this effort are 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** under Basin C**.**

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.

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 County 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

Big Indian Slough Outfall and Pump Station

Chapter 5
Stormwater Quantity Analysis

As part of this Stormwater Management Plan, a hydraulic modeling study was completed to identify existing and potential drainage problems and to develop a list of capital improvement projects to address identified problems. This chapter summarizes the methods and results of the hydraulic modeling. **Chapter 7** 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 8**.

A. Modeling Methodology

1. Computer Hydraulic Model

The XPSWMM-2000 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 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 basins in the study area drain directly into Padilla Bay and were not included in the modeling of the four primary slough-based drainage systems.

2. Model Input

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 high water and mean 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.

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%

Existing effective impervious area [EIA] for each basin was determined using current aerial photographs; future EIA was estimated assuming full buildout conditions under Skagit County's current zoning coverage as of January 2003. The EIA for each zoning classification is shown in **Table 5-1. Figure 5-1** shows the main drainages and the subbasin boundaries in the Study Area. **Table 5-2** lists existing and future EIA for each modeled subbasin.

Table 5-2: Existing and Future Effective Impervious Area

Subbasin	Total Area	Existing EIA	Future EIA	Subbasin	Total Area	Existing EIA	Future EIA
Basin A: No Name Slough							
A-4	490 acres	4.0%	6.1%	A-11a	417 acres	6.0%	21.3%
A-5	306 acres	5.0%	5.2%	A-11b	636 acres	6.0%	6.2%
A-6	100 acres	5.0%	5.1%	A-11c	126 acres	4.0%	5.5%
A-7	325 acres	7.0%	28.1%	A-12	138 acres	6.0%	7.3%
A-8	127 acres	4.0%	6.0%				
Basin B: Joe Leary Slough							
B-1a	90 acres	5.0%	6.0%	B-6b	266 acres	5.0%	6.0%
B-1b	100 acres	4.0%	6.0%	B-6c	213 acres	4.0%	4.0%
B-1c	189 acres	6.0%	6.0%	B-6d	112 acres	5.0%	6.0%
B-1d	112 acres	5.0%	6.0%	B-7	930 acres	4.0%	5.0%
B-1e	108 acres	5.0%	5.0%	B-8a	768 acres	6.0%	11.8%
B-2	245 acres	4.0%	6.0%	B-8b	100 acres	6.0%	30.0%
B-3	500 acres	5.0%	6.0%	B-9	1,870 acres	5.0%	9.0%
B-4	150 acres	5.0%	6.0%	B-10	590 acres	4.0%	5.0%
B-5	90 acres	5.0%	6.0%	B-11	910 acres	5.0%	5.0%
B-6a	305 acres	5.0%	5.0%	B-12	2,630 acres	6.0%	7.0%
Basin C: Little Indian Slough							
C-1a	54 acres	5.0%	5.0%	C-1c	156 acres	5.0%	5.0%
C-1b	218 acres	5.0%	5.0%	C-2	166 acres	15.0%	35.0%
Basin C: Big Indian Slough							
C-2a	135 acres	8.0%	35.0%	C-5	133 acres	4.0%	35.0%
C-3a	363 acres	5.0%	19.0%	C-6	116 acres	8.0%	27.0%
C-3b	220 acres	5.0%	5.0%	C-7	1,647 acres	18.0%	35.0%
C-4	422 acres	4.0%	24.1%	C-8	2,018 acres	8.0%	12.0%

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.

3. Model Basin Descriptions

The following sections describe each basin and the elements included in the models for each.

a. No Name Slough Modeling Basin

Located on the west side of the Study Area, No Name Slough modeling basin drains approximately 2,600 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. **Figure 5-2** shows the modeled elements in the No Name Slough Basin.

b. Joe Leary Slough Modeling Basin

The Joe Leary Slough modeling basin is the most northern basin in the study area and covers about 10,400 acres. This basin was subdivided into 19 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, just 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.

Figure 5-3 shows the modeled elements in the Joe Leary Slough Basin.

c. Little Indian Slough Modeling Basin

The Little Indian Slough modeling basin lies between No Name Slough and Big Indian Slough. This is the smallest of the modeled drainages with a basin area of approximately 600 acres. This basin was subdivided into 4 subbasins for the hydrologic modeling. The topography in Little Indian Slough is mostly flat, although there is some elevation gain in the upper portion of the basin.

Little Indian Slough was modeled from its outlet at Padilla Bay to beyond Farm-to-Market Road. Key culverts at Bay View-Edison Road and Farm-to-Market Road were included in the SWMM model, as well as a culvert crossing on a minor road to the east of Farm-to-Market Road. **Figure 5-4** shows the modeled elements in the Little Indian Slough Basin.

d. Big Indian Slough Modeling Basin

The Big Indian Slough modeling basin is in the southernmost part of the study site and has a drainage area of about 5,000 acres. The topography in most of the basin is flat; the northern part of the basin is part of Bay View Ridge and has steeper slopes. This basin was subdivided into 8 subbasins for the hydrologic modeling.

Big Indian Slough was modeled from its outlet at Padilla Bay to the crossing of SR 20 upstream of Higgins Airport Way. The model includes the key bridges and culverts in this portion for the drainage system. At higher water surface elevations, flow can escape from Big Indian Slough near SR 536 and flow into Higgins Slough. This overflow was included in the model in order to quantify the effects on Big Indian Slough and the possible impacts on Higgins Slough. **Figure 5-5** shows the modeled elements in the Big Indian Slough basin.

B. Modeling Results

Hydraulic modeling was completed for each of the four main drainages in the Bay View watershed: No Name Slough, Joe Leary Slough, Little Indian Slough, and Big Indian Slough.

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 5-3**. **Table 5-4** compares the existing and future peak runoff rates for the 24-hour storm event in the subbasins.

The existing condition peak flows calculated by the SWMM model were compared to the existing conditions peak flows reported in Northwest Hydraulic Consultant's (NHC) *Hydrologic and Hydraulic Model of the No Name Slough Drainage* (November 2000). **Table 5-5** shows a comparison of the flows at three locations for existing conditions. In general, the estimated peak flows calculated by the SWMM model were higher than the peak flows reported in the NHC study. The NHC study did not compute peak flows for future conditions.

The modeling indicated flooding at locations throughout the basin. The flooding is indicated at the 10-, 25-, and 100-year recurrence interval for both existing and future land use conditions. **Table 5-6** shows predicted flooding locations with no drainage improvements implemented.

Table 5-3: Existing and Future Conditions Peak Flows for No Name Slough

SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-10	Outlet of Slough (before pumps)	111 cfs	152 cfs	201 cfs	139 cfs	180 cfs	228 cfs
NN-85	Confluence of Tributaries	95 cfs	130 cfs	178 cfs	120 cfs	154 cfs	200 cfs
NN-130	Marihugh Road	32 cfs	42 cfs	52 cfs	32 cfs	42 cfs	52 cfs

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

Table 5-4: Existing and Future Conditions Peak Runoff for No Name Slough						
	Existing Conditions Peak Runoff			Future Conditions Peak Runoff		
Subbasin	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
A-4	10 cfs	12 cfs	16 cfs	12 cfs	16 cfs	22 cfs
A-5	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
A-7	27 cfs	35 cfs	47 cfs	37 cfs	48 cfs	64 cfs
A-8	10 cfs	13 cfs	18 cfs	11 cfs	14 cfs	20 cfs
A-11a	23 cfs	31 cfs	43 cfs	29 cfs	38 cfs	52 cfs
A-11b	33 cfs	44 cfs	60 cfs	50 cfs	67 cfs	90 cfs
A-11c	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
See **Figure 5-1** for subbasin locations.						

Table 5-5: Comparison of Existing Condition Peak Flow from SWMM and NHC Study						
	SWMM Existing Conditions Peak Flows for No Name Slough			NHC Exiting Conditions Peak Flows for No Name Slough		
Approximate Location	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Outlet of Slough	111 cfs	152 cfs	201 cfs	91 cfs	115 cfs	154 cfs
Confluence of Tributaries	95 cfs	130 cfs	178 cfs	81 cfs	101 cfs	132 cfs
Marihugh Road	32 cfs	42 cfs	52 cfs	12 cfs	16 cfs	23 cfs

Table 5-6: No Name Slough Flooding Locations with No Improvements							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows [cfs]			Future Conditions Peak Flows [cfs]		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Upstream of Outlet	●	●	●	●	●	●
NN-30	Upstream of Slough Outlet	●	●	●	●	●	●
NN-40	Downstream of Bay View-Edison Road	●	●	●	●	●	●
NN-50	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-60	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-65	Slough Upstream of Bay View-Edison Rd	●	●	●	●	●	●
NN-70	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-80	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-140	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-150	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-160	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-170	S, Stem Near Dahlstadt Farm	●	●	●	●	●	●
See **Figure 5-2** for node locations.					● denotes predicted flooding for the storm event		

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 existing and future peak runoff rates for the 24-hour storm event in the subbasins. Flooding locations are listed in **Table 5-9.**

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 the confluence with Maiben Ditch.

Table 5-7: Existing and Future Conditions Peak Flows for Joe Leary Slough						
	Existing Conditions Peak Flows			Future Conditions Peak Flows		
Approximate Locations	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Bay View-Edison Road	326 cfs	395 cfs	495 cfs	344 cfs	418 cfs	525 cfs
Farm-to-Market Road	185 cfs	240 cfs	325 cfs	202 cfs	262 cfs	352 cfs
Allen West Road	165 cfs	215 cfs	295 cfs	183 cfs	238 cfs	322 cfs
Downstream of Confluence	145 cfs	195 cfs	270 cfs	154 cfs	205 cfs	281 cfs
South Spur Ditch at Josh Wilson Road	52 cfs	68 cfs	92 cfs	57 cfs	74 cfs	98 cfs
Maiben Ditch at Thomas Road	76 cfs	104 cfs	146 cfs	78 cfs	106 cfs	149 cfs

Table 5-8: Existing and Future Conditions Peak Runoff for Joe Leary Slough						
	Existing Conditions Peak Runoff			Future Conditions Peak Runoff		
Subbasin	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
B-1a	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
B-1c	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
B-1e	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs
B-2	19 cfs	25 cfs	35 cfs	19 cfs	25 cfs	35 cfs
B-3	34 cfs	45 cfs	62 cfs	35 cfs	46 cfs	63 cfs
B-4	18 cfs	24 cfs	33 cfs	18 cfs	24 cfs	33 cfs
B-5	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
B-6b	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
B-6d	27 cfs	35 cfs	46 cfs	27 cfs	35 cfs	46 cfs
B-7	22 cfs	30 cfs	42 cfs	25 cfs	33 cfs	44 cfs
B-8	38 cfs	50 cfs	68 cfs	68 cfs	89 cfs	118 cfs
B-9	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
B-11	21 cfs	27 cfs	36 cfs	21 cfs	27 cfs	36 cfs
B-12	64 cfs	82 cfs	110 cfs	70 cfs	91 cfs	121 cfs
See **Figure 5-1** for subbasin locations.						

Table 5-9: Joe Leary Slough Flooding Locations with No Improvements						
SWMM Model Node	Existing Conditions Peak Flows			Future Conditions Peak Flows		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
JL-20	●	●	●	●	●	●
JL-24	●	●	●	●	●	●
JL-25	●	●	●	●	●	●
JL-26	●	●	●	●	●	●
JL-30	●	●	●	●	●	●
JL-32	●	●	●	●	●	●
JL-34	●	●	●	●	●	●
JL-36	●	●	●	●	●	●
JL-40	●	●	●	●	●	●
JL-44	●	●	●	●	●	●
JL-46	●	●	●	●	●	●
JL-48	●	●	●	●	●	●
JL-50		●	●	●	●	●
JL-52	●	●	●	●	●	●
JL-54	●	●	●	●	●	●
JL-55		●	●		●	●
JL-60		●	●		●	●
JL-64		●	●	●	●	●
JL-71		●	●	●	●	●
JL-72	●	●	●	●	●	●
JL-84		●	●		●	●
JL-90		●	●		●	●
JL-91		●	●		●	●
JL-92		●	●		●	●
JL-100	●	●	●	●	●	●
JL-105	●	●	●	●	●	●
JL-110		●	●		●	●
JL-111			●			●
See **Figure 5-3** for node locations.				● denotes predicted flooding for the storm event		

3. Little Indian Slough

Predicted peak flows in Little Indian Slough for the 10-, 25-, and 100-year storm events are shown in **Table 5-10. Table 5-11** compares the existing and future peak runoff rates for the 24-hour storm event in the subbasins. Flooding in Little Indian Slough is not expected to increase much under future conditions since it is not likely that there will be a large increase in impervious area at full buildout. The upstream subbasin (C-2) that drains into the slough is part of the Urban Growth Area (UGA), and some development there will increase the EIA in the basin from 15 to 35 percent.

Some flooding problems indicated by the modeling in the Little Indian Slough Basin are listed in **Table 5-12.** The flooding sites are in the ditch downstream of Farm-to-Market Road (modeling Node LI-60) and in a culvert under a private drive east of Farm-to-Market Road (modeling Node LI-80). According to the model, both locations experience flooding at the 10-, 25-, and 100-year recurrence intervals for future conditions, and the culvert at McFarland Road also experiences flooding for the 100-year recurrence interval for existing conditions.

The model does not predict roadway overtopping at the culvert at Farm-to-Market Road, due to the height of the road embankment; however, the culvert there appears to be undersized due to the relatively high water surface elevation and the high predicted flow velocity (12.3 feet per second) in the culvert for the 100-year future conditions rainfall event. The high water surface elevation at Farm to Market Road also may be contributing to flooding upstream.

Table 5-10: Existing and Future Conditions Peak Flows for Little Indian Slough

SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-10	Outlet of Slough	83 cfs	100 cfs	121 cfs	86 cfs	103 cfs	127 cfs
LI-32	Between Bay View-Edison Road & Farm to Market Road	16 cfs	23 cfs	35 cfs	25 cfs	35 cfs	50 cfs
LI-60	Farm to Market Road	13 cfs	18 cfs	26 cfs	23 cfs	30 cfs	41 cfs

See **Figure 5-4** for node locations.

Table 5-11: Existing and Future Conditions Peak Runoff for Little Indian Slough

Subbasin	Existing Conditions Runoff			Future Conditions Runoff		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
C-1a	4 cfs	7 cfs	11 cfs	4 cfs	7 cfs	11 cfs
C-1b	8 cfs	12 cfs	20 cfs	8 cfs	12 cfs	20 cfs
C-1c	6 cfs	11 cfs	17 cfs	6 cfs	11 cfs	17 cfs
C-2	14 cfs	19 cfs	27 cfs	23 cfs	31 cfs	43 cfs

See **Figure 5-1** for subbasin locations.

Table 5-12: Little Indian Slough Flooding Locations with No Improvements							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-20	Upstream of Bay View-Edison Road		●	●		●	●
LI-22	Lower Slough		●	●		●	●
LI-24	Lower Slough		●	●		●	●
LI-26	Lower Slough		●	●		●	●
LI-32	Middle Slough			●			●
LI-60	Upstream End of Culvert LI-C-1	●	●	●	●	●	●
LI-80	Downstream End of Culvert LI-C-2	●	●	●	●	●	●
LI-90	Upstream End of Culvert LI-C-2	●	●	●	●	●	●

See **Figure 5-4** for node locations. ● denotes predicted flooding for the storm event

4. Big Indian Slough

Predicted peak flows in Big Indian Slough for the 10-, 25-, and 100-year storm events are listed in **Table 5-13. Table 5-14** compares the existing and future peak runoff rates for the 24-hour storm event in the subbasins. Six flooding areas were identified in the Big Indian Slough Basin, concentrated around Higgins Airport Way, where runoff from the UGA (including the Skagit Regional Airport) enters the slough. According to the model, the slough is unable to handle the high flows at the confluence; the backwater from this constriction propagates upstream, causing additional flooding. The flooding locations and frequency are listed in **Table 5-15**.

The model indicates the most severe flooding at the culverts upstream and downstream of Higgins Airport Way (model Nodes BI-210, BI-240, and BI-250). Flooding at these locations is predicted at the 10-, 25-, and 100-year recurrence interval for both existing and future conditions. Flooding at the 100-year recurrence interval also is predicted at the Higgins Airport Way culvert (Node BI-230) and at the culvert upstream of SR 20 (Nodes BI-180 and BI-190) for future and existing conditions.

The hydraulic model did not predict flooding downstream of the SR 20 culvert. However, it is well documented that flooding of SR 20 west of Farm-to-Market Road occurred during the November 1990 storm event. The channel appears to have sufficient capacity in this part of the slough for both existing and future conditions. There is relatively little additional tributary area to the slough downstream of the SR 20 culvert, but the channel capacity increases significantly. Reasons for the SR 20 flooding may be from one or a combination of: 1) channel blockage, 2) tidal influence, 3) localized poor drainage, and 4) model calibration.

An overflow in the vicinity of SR 536 provides some relief during peak storm events, as water is diverted from Big Indian Slough south into Higgins Slough. The amount of overflow is presented in **Table 5-16** for each storm event. The model indicates that if the overflow were kept within Big Indian Slough, the downstream portion of the channel would still have adequate capacity for existing and future flow conditions based on the modeling assumptions. However, observed flooding of SR 20 indicates that some hydrologic condition exist that can overwhelm the outfall and lower reach of Big Indian Slough.

Table 5-13: Existing and Future Conditions Peak Flows for Big Indian Slough

SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-10	Outlet to Padilla Bay	311 cfs	352 cfs	403 cfs	404 cfs	564 cfs	631 cfs
BI-160	Downstream of SR 20	151 cfs	205 cfs	286 cfs	170 cfs	265 cfs	337 cfs
BI-230	Higgins Airport Way	53 cfs	72 cfs	104 cfs	84 cfs	87 cfs	131 cfs
BI-270	Above SR 20	49 cfs	63 cfs	85 cfs	54 cfs	69 cfs	93 cfs

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

Table 5-14: Existing and Future Conditions Peak Runoff for Big Indian Slough

Subbasin	Existing Conditions Peak Runoff			Future Conditions Peak Runoff		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
C-2a	9 cfs	15 cfs	22 cfs	28 cfs	32 cfs	50 cfs
C-3a	16 cfs	23 cfs	35 cfs	40 cfs	53 cfs	72 cfs
C-3b	8 cfs	11 cfs	20 cfs	8 cfs	12 cfs	16 cfs
C-4	17 cfs	26 cfs	41 cfs	55 cfs	72 cfs	98 cfs
C-5	6 cfs	13 cfs	17 cfs	25 cfs	33 cfs	45 cfs
C-6	15 cfs	22 cfs	34 cfs	25 cfs	36 cfs	50 cfs
C-7	110 cfs	175 cfs	220 cfs	110 cfs	175 cfs	220 cfs
C-8	47 cfs	60 cfs	82 cfs	60 cfs	76 cfs	106 cfs

See **Figure 5-1** for subbasin locations.

Table 5-15: Big Indian Slough Flooding Locations with No Improvements							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-180	Culvert Upstream of SR 20			●			●
BI-190	Culvert Upstream of SR 20			●			●
BI-210	Culvert Downstream of Higgins Airport Way	●	●	●	●	●	●
BI-230	Culvert at Higgins Airport Way			●			●
BI-240	Culvert Upstream of Higgins Airport Way	●	●	●	●	●	●
BI-250	Culvert Upstream of Higgins Airport Way	●	●	●	●	●	●

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

Table 5-16: Peak Overflow Rates from Big Indian Slough to Higgins Slough			
	Peak Flow to Higgins Slough		
Condition	10-Year	25-Year	100-Year
Existing Conditions	1 cfs	4 cfs	14 cfs
Future Conditions	7 cfs	16 cfs	35 cfs

Chapter 6
Stormwater Quality and Treatment

The purpose of the Bay View Watershed Stormwater Management Plan is to quantify the quantity of stormwater runoff within the Study Area in order to analyze 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 quality control is only cursorily addressed in this report. 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*[10] (Stormwater Manual) as Skagit County has not yet adopted the Department of Ecology's 2001 update to the Stormwater Manual.

The *Padilla Bay/Bay View Watershed Nonpoint Action Plan*[11] (Nonpoint Action Plan) is the most significant work to date regarding stormwater pollution in the Bay View area. 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.

A. Bay View Area Stormwater Quality

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 issued by Ecology. Ecology is expected to issue a NPDES General Permit for Municipal Storm Sewers (Phase II) in late 2006 or early 2007. This

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

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

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 Subarea will be subject to these augmented regulations.

B. Contamination Sources and Management Strategies

There are several potential sources of contamination for stormwater runoff. Below is a brief discussion of some of the obvious and abundant sources of stormwater contamination within the Study Area, followed by a brief discussion of stormwater management strategies for each potential contamination source. The stormwater treatment strategy for the Bay View area is based on recommendations presented in the Nonpoint Action Plan and recommended best management practices [BMPs] presented in the 2001 Stormwater Manual[12].

1. Pavement Runoff and Roadside Ditches

Roadside ditches serve a majority of the roadway system within the Study Area. Only recent residential plats have curbs, gutters, and catch basins. Common stormwater pollutants associated with direct stormwater input into roadside ditches include sediment, hydrocarbons, organic and inorganic particulates, and heavy metals. To minimize pollutant impacts, roadside ditches should be maintained to preserve their condition and design capacity while minimizing bare or thin vegetated surfaces.

Volume IV, Chapter 2 of the 2001 Stormwater Manual provides the BMPs for maintenance of roadside ditches. The Nonpoint Action Plan also has several recommendations for mitigating stormwater runoff quality from pavement and roadside ditches.

2. Septic Tanks

Sanitary sewers currently serve only the southeastern portion of the Study Area. The areas served by sanitary sewers are the commercial areas within and adjacent to the Port of Skagit County and medium density residential developments in the southeast quadrant of the Study Area. The remaining development within the Study Area is served by individual septic tanks.

Septic tanks are a principal means of wastewater treatment and disposal for rural and suburban areas. Septic tanks can be an effective means of wastewater treatment and disposal when properly designed, installed and maintained. However, improperly design, installed and/or maintained septic tanks and cesspools, for both human and animal wastes, can be a major source of ground water and surface water pollution. Individual pollution potential from septic tanks and/or cesspools may be of little significance, but the aggregate impact can be detrimental in specific areas. The principal contaminants from septic tanks are nutrients, fecal coliform, and other biological contaminants, but small quantities of household chemicals can also be a problem. The 303(d) listing of Big Indian Slough, Joe Leary Slough, and No Name Slough for fecal coliform provides supporting evidence of this problem within the Study Area.

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

The *Soil Survey of Skagit County Area, Washington* rates the soils in the Study Area as “severe” for septic tank absorption fields due to wetness and slow percolation characteristics. Soils of this type could be a factor in potential degradation of ground and surface water quality in the Study Area.

The Nonpoint Action Plan recommends several steps that can be taken to help reduce water quality degradation form septic tanks, including:

- Institute public education programs to encourage property owners to actively maintain their septic systems.
- Ensure regular septic system maintenance.
- Promote water conservation measures to improve performance and extend septic system life.
- Provide access to septage disposal facilities.
- Consider using recent advances in septic system technology is areas where conventional systems are inappropriate.
- Provide strong enforcement of septic system maintenance and prompt response to known problems.
- Require sanitary sewer service be provided in newly developed areas within the UGA.

3. Agricultural Activities

Agriculture is a predominant industry in the Bay View area. Agricultural activities include both crop production and livestock operations. Agricultural chemicals and contaminants can contribute sediment, fecal coliform, nutrients, pesticides, fungicides and herbicides to stormwater. *Washington’s Nonpoint Source Management Plan*[13], Chapter 5, offers BMPs for agricultural activities. Skagit County’s recently adopted Agricultural Critical Areas Ordance requires that agricultural operators ‘do not harm’ critical areas. Do not harm is defined as:

- Meeting the water quality standards required by RCW 90.48 (Water Pollution Control Act) and WAC 173-201A,
- Meeting the requirements of any Total Maximum daily load (TMDL) requirements established by the Department of Ecology,
- Meeting all applicable requirements of RCW 77.55 (Hydraulics Code) and WAC 220-110, and
- Meeting specific agricultural practice standards as defined in the ordinance.

The Padilla Bay Demonstration Farm, located in the Study Area, is a full-scale crop farming operation that is used for investigating and demonstrating the application of agricultural BMPs. It is

[13] *Washington’s Nonpoint Source Management Plan,* prepared by the Washington State Department of Ecology (April 2000).

a cooperative effort by the Skagit Valley farmers, Skagit Conservation District, Washington State University, and Padilla Bay Staff. Agricultural BMP's implemented on the demonstration farm that are shown to have significant water quality benefits could be implemented throughout the Study Area to help in reducing non-point source pollution.

The Nonpoint Action Plan recommends several BMPs for mitigating the water quality impacts of agricultural activities. These BMPs include erosion and sedimentation controls, management of runoff from confined animal facilities, nutrient and pesticide management measures, and grazing management practices.

C. Stormwater Treatment Techniques

1. Stormwater Ponds and Bioswales

There are two general types of stormwater ponds, dry ponds and wet ponds. There are also several types of hybrid ponds that utilize a combination of dry and wet pond characteristics.

A dry pond primarily provides temporary stormwater detention by holding stormwater and releasing it at a controlled rate over a period of time. Most of the time the pond is dry and there is very little stormwater treatment. The primary purpose is to reduce the peak stormwater runoff rate and reduce downstream erosion impacts. Water quality benefits, if any, result from settling of suspended solids and attached pollutants and absorption onto soils. Dissolved pollutants are most likely not removed.

A wet pond contains a permanent pool of water and provides both stormwater detention and treatment. Water within a wet pond may dry up during the dry season. Water quality benefits result from settling of suspended solids and attached pollutants, absorption onto soils, and transformation and uptake by bacteria and algae.

Bioswales are shallow grass-lined channels that stormwater runoff passes through. Water quality benefits result from settling of suspended solids and attached pollutants, absorption onto soils, and uptake by grass roots. Bioswales are often used in conjunction with dry ponds to provide stormwater treatment. Bioswales are most often used in small-scale developments, typically sloped to drain, and do not hold water.

From a maintenance standpoint, fewer, larger ponds are more advantageous than numerous smaller ponds.

2. Wetlands

Wetlands, both natural and constructed, have been demonstrated to provide good stormwater treatment. Water quality benefits from wetlands result from settling of suspended solids and attached pollutants, absorption into soils, and transformation and uptake by bacteria, algae and vegetation roots. Wetlands also provide wildlife habitat and are typically more aesthetically pleasing when compared to ponds.

Recognizing the valuable contribution of wetlands, both natural and constructed, their protection is extremely important. In addition, it is important to preserve the natural balance in a wetland. Any disruption of a wetland, both directly to the wetland and/or indirectly to the contributing drainage area, could alter its biological balance. When the biological balance it altered, the wetland's effectiveness for stormwater treatment could diminish.

D. West Nile Virus

Within the past few years, wetlands and detention ponds have been scrutinized for their possible contribution as a breeding ground for mosquitoes and the spread of the West Nile Virus. Though not yet prevalent in the Pacific Northwest, the West Nile Virus has been spreading at an alarming rate. Currently, it is thought the mosquitoes that are responsible for transmitting the West Nile Virus, such as the *Culex* species, are not common in wetlands. Research into this disease is in its infancy, however, some agencies, such as the US Environmental Protection Agency[14] and others[15] have publics some initial findings.

Wetlands, both natural and constructed, and detention ponds have a potential to provide a breeding ground for mosquitoes. Common characteristics include shallow water depths (less than 1 meter), dense aquatic vegetation, and stagnant water during summer conditions. It is thought that a healthy wetland can reduce the potential for mosquito breeding, but not eliminate it. It is sometimes difficult to maintain a healthy wetland in an urban environment.

Some design and maintenance measures to achieve a healthy wetland or detention pond include the following:

- For wet ponds, maintain a minimum depth of 1 meter and construct steep side slopes. This will limit the amount of area that can be used as mosquito breeding habitat.
- Design dry pond to drain completely within 72 hours.
- Maintain a constant supply of fresh water to the wetlands and wet pond to diminish stagnation.
- Aerate the wet pond to increase the concentration of dissolved oxygen and diminish stagnation.
- Drain or pump out flow control structures during the spring and summer mosquito breading period.
- Submerge inlet and outlet pipe to reduce surface area from mosquitoes to lay eggs.
- Control the growth and density of pond-edge vegetation that would inhibit mosquito predators. Also, adult mosquitoes are attracted to the dense vegetation near the waters edge to lay their eggs. Impermeable liners may be used to control pond-edge vegetation.

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[14] *Do Stormwater Retention Ponds Contribute to Mosquito Problems?,* Nonpoint Source News-Notes, US Environmental Protection Agency, Issue No. 71, May 2003.

[15] *Stormwater Management Could Combat West Nile Virus,* R. Dale Downey, PE, Cumming Cockburn Limited, September 2003.

In order to achieve these goals, a responsible entity, such as Skagit County or drainage districts, needs to understand the importance of routine maintenance to maximize the stormwater treatment potential of detention ponds and to minimize the potential for developing mosquito breeding habitat.

Chapter 7
Storm Drainage Alternatives Analysis

Several conceptual alternatives for relieving flooding in each basin are proposed and evaluated below. This is not considered to be a comprehensive list; conceptual alternatives were selected for evaluation based on their probability of correcting flooding problems. Input from stakeholders provided the design basis for some of the evaluated alternatives. Other alternatives were evaluated based on specific requests by stakeholders. Additional alternatives or variations on these alternatives may become viable solutions in solving flooding problems. The optimal solution will most likely require a combination of the alternatives.

A. Storm Drainage Structures

There are several types of hydraulic structures that are capable of conveying or controlling stormwater. These hydraulic structures include, but are not limited to, open channels, conduits, and pump stations.

1. Open Channels

Open channels are conduits with a free water surface that is exposed to the atmosphere. They are the most common type of stormwater drainage structures used and include sloughs and ditches, free flowing culverts and drainage pipes, and detention ponds. Culverts, when not completely submerged, act as open channels.

a. Sloughs and Ditches

Ditches are used extensively in rural areas to convey stormwater. Ditches can be constructed below the existing ground level or elevated above the existing ground through the construction of levees. Sloughs are typically large drainage channels found in sub-tidal areas. Sloughs sometimes follow historic river channels, while ditches are often manmade. In some cases, stormwater can be temporarily stored and/or treated within sloughs and ditches. Also in some cases, sloughs and ditches can be enhanced and managed for wildlife habitat.

Sloughs and ditches are an effective means of conveying large quantities of stormwater. Design consideration must be given to the location and size in order for them to be effective. There can be no hydraulic barrier that restricts the free flow of water. Over topping of sloughs and ditches during large storm events can have damaging results. Other hydraulic structures, such and culverts and tide gates, are often incorporated into a slough or ditch system.

b. Culverts and Pipes

Free flowing culverts and pipes are often used when it is desirable to place structures, such as roadways, over the stormwater conveyance system. Culverts and pipes can also be used in slough and ditch systems when a hydraulic barrier is encountered, such as tide-influenced outfalls. Culverts and pipes are not as effective and efficient as sloughs and ditches in conveying large quantities of stormwater runoff.

c. Detention Ponds

Detention ponds are used in stormwater conveyance to reduce the peak runoff rate during a storm event. This is accomplished by temporarily detaining stormwater runoff at or near the source and releasing is to the downstream conveyance system at a reduced rate. Detention ponds can also be designed to incorporate stormwater treatment measures such as sedimentation and nutrient absorption.

Detention ponds allow a new downstream conveyance system to be smaller or reduce existing capacity problems on existing downstream conveyance systems. The tradeoff is that detention ponds require available land for construction of the facility and ongoing maintenance to ensure their effective operation.

2. Conduits

Conduits are pressurized pipelines where the free water surface is almost never within the conduit itself. The most common conduit is stormwater conveyance is the discharge forcemain of a pump station. During storm events, culvert and gravity flow pipelines can experience surcharging, resulting in a temporary pressurized condition. During high tides, tide gates also act as pressurized conduits.

3. Pump Stations

Pump stations are used to overcome a hydraulic barrier. Typical hydraulic barriers in stormwater drainage include tide-influenced outfalls and terrain barriers. A pump station at a tide-influenced outfall provides the opportunity to discharge stormwater during periods of high tide when tide gates are closed. A pump station could also be employed to effectively drain stormwater from a low lying area.

Two pump options to consider are screw pumps and centrifugal pumps. Either option can effectively convey stormwater but they present different design and operational considerations.

a. Screw Pumps

Screw pumps are an efficient means of lifting large quantities of water at low heads and are ideal as a drainage pump in low-lying areas such as reclaimed land areas. These pumps have a flow range of 0.2 cfs to 200 cfs and can provide lift of 3 to 30 feet.

The operation of a screw pump is like that of a moving bucket conveyor. The volume of the "bucket" is formed between two flights on the screw with a trough acting as the bottom and sides. Since the screw is on an incline there is always a void space at the top of each bucket. Screw pumps offer the following advantages:

- They offer variable pump capacity while operating at a constant speed.

- They have high operating efficiencies over a greater range than other pumps.
- They can handle large objects so the pumps do not clog.
- They are fish friendly.
- They require minimal maintenance and upkeep.
- They do not require a wet well, piping or a pump house.

The main disadvantages of a screw pump are that the lift elevation is limited and that discharge from a screw pump is to atmosphere and cannot be delivered to a pressurized discharge location. These issues are not a concern for the expected application of pumps for the slough outfalls within the Bay View area.

b. Kinetic Pumps

Kinetic pumps use kinetic energy to impart velocity and pressure to a column of fluid as it move through the pump's impeller Current pump stations at the outlets of the No Name Slough and Big Indian Slough utilize kinetic type pumps. Kinetic type pumps offer the following advantages:

- They have a lower capital cost than screw pumps.
- Spare parts are more readily available.
- Operators are more familiar with operation and maintenance of this type of pump than other types of pumps.
- Their required footprint is smaller than that of screw pumps.

B. No Name Slough

According to local property owners, flooding in the No Name Slough drainage basin is widespread in the lower reaches. In addition, stormwater runoff in the steep portions of the drainage basin causes considerable erosion of the stream channel. The following conceptual alternatives were examined to relieve flooding and/or erosion in this drainage basin. Modeling results indicated that flooding in the lower basin of No Name Slough is controlled primarily by tidal elevations at the outlet.

1. Upgrade of Restricted Culverts

The 25-year 24-hour future conditions storm was used to identify capacity-restricted culverts and to determine the necessary culvert size to eliminate the restrictions. **Figure 5-2** identifies the culverts that were included in the hydraulic modeling. **Table 7-1** summarizes the restricted culverts and the size of the required replacement culvert; locations of these culverts are highlighted on **Figure 7-1.**

Table 7-1: No Name Slough Identified Culvert Restrictions					
	25-year	Est. Tailwater	Targeted Head	Recommended Replacement	
Culvert ID	Peak Flow	Elevation	Water Elevation	Culvert Size	Type/Material
NN-C-Out	160 cfs	2 feet	2 feet	4-ft x 12-ft	Box/Concrete
NN-C-5	60 cfs	6 feet	6 feet	54-inch	Circular/CMP
NN-C-3	120 cfs	104 feet	105 feet	84-inch	Circular/CMP
cfs = cubic feet per second; CMP = corrugated metal pipe					

2. Regional Detention

Three regional detention alternatives were evaluated. The location of these ponds are shown in **Figure 7-1.** The first two detention pond locations were east of Farm-to-Market Road to collect runoff from Subbasins A-7 and A-8, which discharge into the south stem of No Name Slough. The third detention pond location was north of Marihugh Road to collect runoff from Subbasin A-11b. The analysis of these two ponds indicated that flooding in the lower basin would not be eliminated by even complete detention of these upland flows. Therefore, the analysis focused on the amount of storage needed to mitigate for future development or, in the case of the Marihugh Road Pond, to potentially reduce erosion in No Name Creek. Three locations were identified as potential pond sites:

- **The Paccar Technical Center**—Since the Port has plans to provide detention for the parcels being developed in the southwest quadrant of its property, this area was not included in a regional detention analysis. A detention pond on the south stem was analyzed, but only to collect runoff from the Paccar Technical Center, assuming full buildout conditions. The pond was sized to detain the 100-year existing conditions peak flow from the Paccar Technical Center.
- **Northwest Corner of Port Property in Subbasin A-11a**—This pond was assumed to collect runoff from the Port's property in Subbasin A-11a. It was assumed that the entire parcel would be developed to full buildout conditions. This is a conservative assumption, since FAA regulations may restrict the amount of the parcel that can be developed. The pond was sized to detain the 100-year existing conditions peak flow.
- **North of Marihugh Road**—Subbasin A-11b is not currently within the UGA and the future-conditions density is assumed to remain close to that of existing conditions. Since erosion in No Name Creek is a documented concern and since there are relatively few sites were detention may be an option, a pond was analyzed at this location as possible mitigation for existing development impacts. This pond was sized to detain the 100-year peak flow from pre-development conditions, as documented in *Padilla Bay Hydrology of No Name Slough Drainage* (November 2000). A linear interpolation, by tributary area, of the peak flow documented at Bay View Road was used to estimate the pre-development flow at Marihugh Road. The pre-developed peak flow at Marihugh Road was estimated to be approximately 23 cubic feet per second (cfs).

Table 7-2 summarizes the approximate storage that would be needed to meet the design goals of the ponds.

Table 7-2: No Name Slough Detention Pond Volumes			
Location	25-year Pond Inflow	Peak Flow Target	Estimated Pond Volume
Paccar	62 cfs	28 cfs	14 ac-ft
Subbasin A-11a	85 cfs	35 cfs	13 ac-ft
Marihugh Road†	42 cfs	18 cfs	32 ac-ft
† Marihugh pond design is based on 25-year peak flow.			

3. High-Flow Bypass

Since flooding in the lowlands is driven primarily by high tides coinciding with peak runoff, there are few feasible options for reducing the flooding. A high-flow bypass was analyzed, and although it was shown to increase the capacity of the slough, lowland flooding during high tide was predicted to be only marginally better than without the bypass. The main benefit of the bypass during a high tide was providing additional storage. However, a high-flow bypass in conjunction with an improved outlet would allow No Name Slough to drain more quickly when the tide recedes, which would reduce flood duration. **Table 7-3** and **Figure 7-2** show flooding locations predicted by the high-flow bypass analysis. Flooding reduction from this alternative is most likely a result of the incremental increase in storage volume along the slough.

4. Increased Outlet Pumping Capacity

One of the few ways to reduce flooding in the slough during a high tide would be to increase the capacity of the pump station at the outlet to allow drainage when the tidal head exceeds flood stage in the slough. A pump station that would eliminate flooding at infrequent recurrence intervals would likely be prohibitively expensive; therefore, a pump station should be designed to reduce more frequent flooding events. This analysis examined the pumping capacity that would be needed to reduce flooding at the 10-year recurrence interval, which was the smallest storm analyzed for this study.

The capacity of the existing pump station at the outlet of No Name slough is approximately 35 cfs. The existing conditions 10-year peak flow at the outlet of the slough is estimated to be 111 cfs, and the future conditions 10-year peak flow is estimated to be 138 cfs.

The analysis of increased pumping capacity was performed at a conceptual level; further study will be required to refine the necessary pumping capacity and the proper operating scheme for an upgraded pump station. It also may be appropriate to examine pumping requirements to reduce flooding for a more frequent flood event, such as the 2-year event. Mitigating for more frequent events would likely result in a smaller pump station that would likely be less expensive to maintain and operate.

Table 7-3: No Name Slough Flooding Locations with High Flow Bypass							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Upstream of Outlet	●	●	●	●	●	●
NN-30	Upstream of Slough Outlet	●	●	●	●	●	●
NN-40	Downstream of Bay View-Edison Road	●	●	●	●	●	●
NN-60	Culvert at Bay View-Skagit River Road	●	●	●	●	●	●
NN-65	Slough Upstream of Bay View-Edison Rd	●	●	●	●	●	●
NN-70	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-80	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-140	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-160	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-170	S, Stem Near Dahlstadt Farm	●	●	●	●	●	●
See **Figure 5-2** for node locations. See **Table 5-6** for flooding locations with no improvements.				● denotes predicted flooding for the storm event			

The analysis of pumping improvements indicates that increasing the pumping capacity at the outlet of the slough will reduce water surface elevations in the slough and can reduce flooding in the lowland portion of the basin. **Table 7-4** and **Figure 7-2** show predicted flooding using a pump station with a capacity of 90 cfs that begins pumping when the water surface elevation at the outlet of the slough reaches approximately -2.0 feet NGVD (National Geodetic Vertical Datum).

While the results of the pump station analysis indicate that increased pumping at the slough outlet will reduce water surface elevations in No Name Slough, they do not indicate that flooding would be eliminated from the lowland basins. The simplified nature of the hydraulic model used for the analysis may lead to an overestimation of the effects of increased pumping capacity.

Under previous agreement that no lateral drainages would be modeled, the model assumes that all water draining from the lowland basin enters the slough and that flooding occurs only when the water surface elevations exceed the slough's banks. In reality, as the water surface elevation in the slough increases, the lateral systems will not drain into the slough because they are at lower elevations. Therefore, water will still pond in adjoining fields until the elevation in the slough is lowered.

Table 7-4: No Name Slough Flooding Locations with Increased Pumping Capacity							
SWMM Model Node	Approximate Location	Existing Conditions Peak Flows			Future Conditions Peak Flows		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Upstream of Outlet		●	●		●	●
NN-30	Upstream of Slough Outlet		●	●		●	●
NN-40	Downstream of Bay View-Edison Road		●	●		●	●
NN-60	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-65	Slough Upstream of Bay View-Edison Rd		●	●		●	●
NN-70	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-80	Culvert at Bay View-Skagit River Road		●	●		●	●
NN-83	Confluence of Tributaries	●	●	●	●	●	●
NN-140	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-160	S. Stem Upstream of No Name Slough	●	●	●	●	●	●
NN-170	S, Stem Near Dahlstadt Farm	●	●	●	●	●	●

See **Figure 5-2** for node locations. ● denotes predicted flooding for the storm event

See **Table 5-6** for flooding locations with no improvements.

The model does not indicate flooding until the slough water surface exceeds the estimated overtopping elevation of 3 feet NGVD; however, in reality water will not drain from the fields into the slough until the water surface elevation in the fields exceeds the water surface elevation in the slough. Since there are many areas in the fields that are at or below this elevation, flooding is likely to continue.

Operation of the pump station also plays a factor in flooding frequency. As the flood wave travels down the slough, it increases water surface elevations in the slough and causes flooding. Reducing flooding in the basin will require either keeping water surface elevations in the slough at a reduced level, or providing a large enough pumping capacity to eliminate flood volumes at a rate fast enough to have an impact on the upstream portion of the slough. Pump station operation will have to maximize the storage available in the slough. In order to operate effectively, the pump station must have a large enough capacity and begin pumping early enough to create enough drawdown at the slough's outlet to facilitate drainage upstream. This may mean that the pump station often will operate under conditions that would not have caused upstream flooding, which is inefficient and may be expensive.

C. Joe Leary Slough

The flooding problems in Joe Leary Slough indicate that the there is not enough capacity in the current system to convey peak flows. The existing hydraulic structures (bridges and culverts) appear to have enough capacity to convey peak flows, however, the low gradient of the slough and the large impact of tidal influence overly restrict the slough capacity.

Alternatives to reducing stormwater drainage impacts that were considered include a bypass channel along the lower reaches of Joe Leary Slough, a pump station at the slough outlet, a pump station at the South Spur Ditch upstream of its confluence with Maiben Ditch, and detention at the slough outlet. The location of these alternatives is shown on **Figure 7-3**.

1. Peth Property Bypass Channel

A bypass channel in the lower portion of the slough along the toe of the hill was examined as a way of reducing flooding. This bypass channel would cross existing farmland owned by John Peth & Sons, LLC. Routing flow more directly to the outlet may reduce drainage times for the low-lying fields and reduce peak flows where the slough is confined along D'Arcy Road. It was assumed for this analysis that a portion of Subbasin B-1 would drain directly into the bypass. The bypass channel would be excavated to the bottom elevation of the existing slough and would not have any levees. The bypass channel was assumed to be trapezoidal in shape with a length of 4,200 feet. The following characteristics were used to define the bypass:

- 6-foot bottom width
- 2:1 side slopes
- 10 feet of total depth
- A constant slope of 0.0028 percent
- Manning's 'n' roughness coefficient of 0.045

Table 7-5 and **Table 7-6** show the modeled effect of the bypass channel on water surface elevations for existing and future conditions, respectively. **Table 7-7** and **Figure 7-4** show the modeled flooding locations for existing and future conditions with the bypass. The modeling shows that the bypass would reduce water surface elevations along nearly the entire length of the slough. In some locations, levels would be reduced by 0.6 feet, 0.9 feet and 1.1 feet for the 10-, 25-, and 100-year future conditions events, respectively. At high tides the bypass would provide incremental storage, reducing the volume of water stored in the main stem of the slough. During low tides, the bypass would facilitate drainage in the fields by providing an additional drainage path to the outlet of the slough, directing some of the peak flow away from the channel restriction along D'Arcy Road.

This analysis assumed no upstream flooding, so that the maximum peak flow and volume reach the slough's outlet. In fact, upstream flooding would likely reduce peak flow in the channel and reduce the magnitude that water levels would be lowered by the bypass channel. However, the conclusion that levels would be reduced by the bypass still appears to be valid.

Table 7-5: Joe Leary Slough Existing Conditions Water Surface Elevations With and Without Slough Bypass									
SWMM Model Node	Water Surface Elevation No Improvements			Water Surface Elevation With Slough Bypass			Difference		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Main Stem									
JL-20	3.9 feet	3.9 feet	3.9 feet	3.8 feet	3.9 feet	3.9 feet	-0.1 feet	0.0 feet	0.0 feet
JL-30	3.9 feet	4.1 feet	4.3 feet	3.8 feet	4.0 feet	4.2 feet	-0.1 feet	-0.1 feet	-0.1 feet
JL-40	4.1 feet	4.4 feet	4.9 feet	3.9 feet	4.1 feet	4.4 feet	-0.2 feet	-0.3 feet	-0.5 feet
JL-50	4.6 feet	5.2 feet	5.9 feet	4.1 feet	4.4 feet	4.9 feet	-0.5 feet	-0.8 feet	-1.0 feet
JL-60	4.8 feet	5.5 feet	6.3 feet	4.4 feet	4.9 feet	5.5 feet	-0.4 feet	-0.6 feet	-0.8 feet
JL-70	5.1 feet	5.9 feet	7.0 feet	4.8 feet	5.4 feet	6.2 feet	-0.3 feet	-0.5 feet	-0.8 feet
JL-80	5.4 feet	6.2 feet	7.3 feet	5.0 feet	5.7 feet	6.7 feet	-0.4 feet	-0.5 feet	-0.6 feet
JL-90	5.8 feet	6.6 feet	7.9 feet	5.5 feet	6.3 feet	7.3 feet	-0.3 feet	-0.3 feet	-0.6 feet
JL-100	6.5 feet	7.5 feet	9.1 feet	6.3 feet	7.3 feet	8.7 feet	-0.2 feet	-0.2 feet	-0.4 feet
JL-110	6.5 feet	7.6 feet	9.2 feet	6.3 feet	7.4 feet	8.8 feet	-0.2 feet	-0.2 feet	-0.4 feet
JL-120	6.7 feet	7.8 feet	9.3 feet	6.5 feet	7.5 feet	9.0 feet	-0.2 feet	-0.3 feet	-0.3 feet
Maiben Ditch									
JL-130	7.2 feet	8.2 feet	9.8 feet	7.0 feet	8.1 feet	9.5 feet	-0.2 feet	-0.1 feet	-0.3 feet
JL-140	8.0 feet	9.0 feet	10.4 feet	7.9 feet	8.9 feet	10.2 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-150	8.6 feet	9.6 feet	10.9 feet	8.6 feet	9.5 feet	10.7 feet	0.0 feet	-0.1 feet	-0.2 feet
JL-160	10.0 feet	10.8 feet	11.9 feet	10.0 feet	10.8 feet	11.9 feet	0.0 feet	0.0 feet	0.0 feet
South Spur Ditch									
JL-161	7.1 feet	8.2 feet	9.7 feet	7.0 feet	8.0 feet	9.4 feet	-0.1 feet	-0.2 feet	-0.3 feet
JL-170	7.1 feet	8.2 feet	9.7 feet	7.0 feet	8.1 feet	9.5 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-180	7.2 feet	8.3 feet	9.8 feet	7.1 feet	8.1 feet	9.5 feet	-0.1 feet	-0.2 feet	-0.3 feet
JL-200	7.4 feet	8.4 feet	9.8 feet	7.3 feet	8.3 feet	9.6 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-220	7.4 feet	8.4 feet	9.8 feet	7.3 feet	8.3 feet	9.6 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-250	7.5 feet	8.5 feet	9.9 feet	7.4 feet	8.3 feet	9.6 feet	-0.1 feet	-0.2 feet	-0.3 feet
See **Figure 5-3** for node locations.									

Table 7-6: Joe Leary Slough Future Conditions Water Surface Elevations With and Without Slough Bypass									
SWMM Model Node	Water Surface Elevation No Improvements			Water Surface Elevation With Slough Bypass			Difference		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Main Stem									
JL-20	3.9 feet	3.9 feet	3.9 feet	3.8 feet	3.9 feet	3.9 feet	-0.1 feet	0.0 feet	0.0 feet
JL-30	4.0 feet	4.1 feet	4.4 feet	3.9 feet	4.0 feet	4.2 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-40	4.1 feet	4.5 feet	5.0 feet	4.0 feet	4.1 feet	4.5 feet	-0.1 feet	-0.4 feet	-0.5 feet
JL-50	4.7 feet	5.3 feet	6.1 feet	4.1 feet	4.4 feet	5.0 feet	-0.6 feet	-0.9 feet	-1.1 feet
JL-60	4.9 feet	5.6 feet	6.4 feet	4.5 feet	4.9 feet	5.6 feet	-0.4 feet	-0.7 feet	-0.8 feet
JL-70	5.2 feet	6.0 feet	7.1 feet	4.9 feet	5.5 feet	6.3 feet	-0.3 feet	-0.5 feet	-0.8 feet
JL-80	5.5 feet	6.3 feet	7.5 feet	5.2 feet	5.8 feet	6.8 feet	-0.3 feet	-0.5 feet	-0.7 feet
JL-90	5.9 feet	6.8 feet	8.0 feet	5.7 feet	6.4 feet	7.5 feet	-0.2 feet	-0.4 feet	-0.5 feet
JL-100	6.6 feet	7.7 feet	9.3 feet	6.4 feet	7.5 feet	8.9 feet	-0.2 feet	-0.2 feet	-0.4 feet
JL-110	6.7 feet	7.8 feet	9.4 feet	6.5 feet	7.6 feet	9.0 feet	-0.2 feet	-0.2 feet	-0.4 feet
JL-120	6.8 feet	7.9 feet	9.6 feet	6.7 feet	7.7 feet	9.2 feet	-0.1 feet	-0.2 feet	-0.4 feet
Maiben Ditch									
JL-130	7.3 feet	8.4 feet	10.0 feet	7.2 feet	8.3 feet	9.7 feet	-0.1 feet	-0.1 feet	-0.3 feet
JL-140	8.1 feet	9.1 feet	10.6 feet	8.0 feet	9.0 feet	10.4 feet	-0.1 feet	-0.1 feet	-0.2 feet
JL-150	8.7 feet	9.7 feet	11.0 feet	8.7 feet	9.6 feet	10.9 feet	0.0 feet	-0.1 feet	-0.1 feet
JL-160	10.0 feet	10.9 feet	12.1 feet	10.0 feet	10.9 feet	12.0 feet	0.0 feet	0.0 feet	-0.1 feet
South Spur Ditch									
JL-161	7.3 feet	8.4 feet	9.9 feet	7.1 feet	8.2 feet	9.6 feet	-0.2 feet	-0.2 feet	-0.3 feet
JL-170	7.3 feet	8.4 feet	10.0 feet	7.2 feet	8.3 feet	9.7 feet	-0.1 feet	-0.1 feet	-0.3 feet
JL-180	7.4 feet	8.5 feet	10.0 feet	7.3 feet	8.3 feet	9.7 feet	-0.1 feet	-0.2 feet	-0.3 feet
JL-200	7.6 feet	8.6 feet	10.1 feet	7.5 feet	8.5 feet	9.8 feet	-0.1 feet	-0.1 feet	-0.3 feet
JL-220	7.6 feet	8.6 feet	10.1 feet	7.5 feet	8.5 feet	9.8 feet	-0.1 feet	-0.1 feet	-0.3 feet
JL-250	7.7 feet	8.7 feet	10.1 feet	7.6 feet	8.6 feet	9.9 feet	-0.1 feet	-0.1 feet	-0.2 feet
See **Figure 5-3** for node locations.									

SWMM Model Node	Future Conditions Flooding Locations Without Improvements			Flooding Locations With Slough Bypass					
				Existing Conditions			Future Conditions		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
JL-20	●	●	●		●	●		●	●
JL-30	●	●	●	●	●	●	●	●	●
JL-40	●	●	●	●	●	●	●	●	●
JL-50	●	●	●			●			●
JL-60		●	●			●			●
JL-71	●	●	●		●	●		●	●
JL-80		●	●		●	●		●	●
JL-90		●	●			●			●
JL-105		●	●		●	●		●	●
JL-110		●	●		●	●		●	●

Table 7-7: Joe Leary Slough Main Stem Flooding With and Without Slough Bypass

See **Figure 7-4** for node locations. ● denotes predicted flooding for the storm event

2. Culvert Analysis

None of the culverts along the main stem of Joe Leary Slough appear to be significant flow restrictions. No culvert replacements are recommended. The outlet structure of twelve (12) 48-inch culverts seems to be adequate to drain the slough during a low tide. No changes are recommended at the outlet.

3. Pump Station at the Outlet

A pump station at the outlet 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 outlet. 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 outlet culvert can exceed the storage capacity of the slough and cause flooding or raise water levels in the slough to prevent drainage from adjacent fields that are at 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 cfs.

Table 7-8 and **Figure 7-4** summarize flooding locations for the 10-year existing and future conditions event with a pump capacity of approximately 300 cfs.

Table 7-8: Joe Leary Slough 10-Year Event Flooding Locations With and Without 300-cfs Outlet Pump Station			
SWMM Model Node	10-Year Flooding Locations Without Improvements	10-Year Flooding Locations With 300-cfs Pump Station at Outlet	
		Existing Conditions	Future Conditions
JL-20	●		
JL-24	●		
JL-25	●		
JL-26	●		
JL-30	●		
JL-32	●		
JL-34	●		
JL-36	●		
JL-40	●		
JL-44	●		●
JL-46	●		
JL-48	●		
JL-50	●	●	●
JL-52	●	●	●
JL-54	●	●	●
See **Figure 7-4** for node locations.		● denotes predicted flooding for the storm event	

Table 7-9 shows the effect of a pump station on slough elevations. According to the model results, a pump station at the outlet would provide the most benefit from the outlet 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 should be expected.

Table 7-9: Joe Leary Slough Future Conditions Water Surface Elevations With and Without Outlet Pump Station - 10-Year Storm Event						
	Existing Conditions Water Surface Elevations			Future Conditions Water Surface Elevations		
SWMM Model Node	No Improvements	With Pump Station	Difference	No Improvements	With Pump Station	Difference
Main Stem						
JL-20	3.9 feet	0.5 feet	-3.4 feet	3.9 feet	0.5 feet	-3.4 feet
JL-30	3.9 feet	1.2 feet	-2.7 feet	4.0 feet	1.2 feet	-2.8 feet
JL-40	4.1 feet	2.4 feet	-1.7 feet	4.1 feet	2.4 feet	-1.7 feet
JL-50	4.6 feet	3.8 feet	-0.8 feet	4.7 feet	3.9 feet	-0.8 feet
JL-60	4.8 feet	4.3 feet	-0.5 feet	4.9 feet	4.3 feet	-0.6 feet
JL-70	5.1 feet	4.7 feet	-0.4 feet	5.2 feet	4.8 feet	-0.4 feet
JL-80	5.4 feet	5.1 feet	-0.3 feet	5.5 feet	5.1 feet	-0.4 feet
JL-90	5.8 feet	5.6 feet	-0.2 feet	5.9 feet	5.7 feet	-0.2 feet
JL-100	6.5 feet	6.4 feet	-0.1 feet	6.6 feet	6.5 feet	-0.1 feet
JL-110	6.5 feet	6.5 feet	0.0 feet	6.7 feet	6.6 feet	-0.1 feet
JL-120	6.7 feet	6.7 feet	0.0 feet	6.8 feet	6.8 feet	0.0 feet
Maiben Ditch						
JL-130	7.2 feet	7.2 feet	0.0 feet	7.3 feet	7.3 feet	0.0 feet
JL-140	8.0 feet	8.0 feet	0.0 feet	8.1 feet	8.1 feet	0.0 feet
JL-150	8.6 feet	8.6 feet	0.0 feet	8.7 feet	8.7 feet	0.0 feet
JL-160	10.0 feet	10.0 feet	0.0 feet	10.0 feet	10.0 feet	0.0 feet
South Spur Ditch						
JL-161	7.1 feet	7.1 feet	0.0 feet	7.3 feet	7.2 feet	-0.1 feet
JL-170	7.1 feet	7.1 feet	0.0 feet	7.3 feet	7.3 feet	0.0 feet
JL-180	7.2 feet	7.2 feet	0.0 feet	7.4 feet	7.4 feet	0.0 feet
JL-200	7.4 feet	7.4 feet	0.0 feet	7.6 feet	7.6 feet	0.0 feet
JL-220	7.4 feet	7.4 feet	0.0 feet	7.6 feet	7.6 feet	0.0 feet
JL-250	7.5 feet	7.5 feet	0.0 feet	7.7 feet	7.7 feet	0.0 feet
See **Figure 5-3** for node locations.						

4. South Spur Pump Station

A pump station on the South Spur Ditch was examined as a way to reduce water surface elevations in that portion of the slough. It was assumed that a culvert with a flap gate would be installed on the South Spur Ditch along with the pump station. The flap gate would prevent back flow from the downstream portion of the slough, and the pump station would pass the peak flow in the slough when the flow in the culvert is reduced by downstream hydraulic conditions.

The modeling showed that a pump station on the South Spur ditch would be most effective at reducing flooding for storms at or below the 10-year recurrence interval. A pump station with a maximum capacity of 48 cfs appears to be effective at reducing flooding for the 10-year existing conditions storm event, while a pump station with a maximum capacity of 73 cfs appears to be effective at reducing flooding for the 10-year future conditions storm event. The model results indicate a restriction in the South Spur Ditch that reduces the pump station's effectiveness in reducing flooding. The restriction appears to be a narrowing of the channel approximately 800 feet downstream of the Josh Wilson Road culvert crossing (see **Figure 7-3**). The channel cross-section at the restriction is represented by Cross Section BX-18, which is smaller than Cross Section BX-17; the smallest cross-sectional required to convey the peak flow from the 10-year storm. The existing ditch cross-section geometry is shown in **Appendix A.** If the South Spur Ditch is enlarged to match the dimensions of Cross Section BX-17, flooding could be reduced along the entire length of the South Spur Ditch. If the restriction is not removed, flooding would still occur in the low area between the Josh Wilson Road culvert and the Michael Road culvert. **Table 7-10** shows the modeled effect of the pump station on water surface elevations for existing and future conditions in the South Spur Ditch at the 10-year event.

Table 7-10: Joe Leary Slough Water Surface Elevation With and Without South Spur Pump Station - 10-Year Storm Event

SWMM Model Node	Water Surface Elevation No Improvements		Water Surface Elevation With Pump Station		Difference	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
JL-161	7.1 feet	7.3 feet	7.1 feet	7.4 feet	0.0 feet	0.1 feet
JL-170	7.1 feet	7.3 feet	7.1 feet	7.4 feet	0.0 feet	0.1 feet
JL-180	7.2 feet	7.4 feet	3.7 feet	3.7 feet	-3.5 feet	-3.7 feet
JL-200	7.4 feet	7.6 feet	4.9 feet	5.1 feet	-2.5 feet	-2.5 feet
JL-220	7.4 feet	7.6 feet	5.0 feet	5.7 feet	-2.4 feet	-1.9 feet
JL-250	7.5 feet	7.7 feet	5.4 feet	6.4 feet	-2.1 feet	-1.3 feet

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

Downstream impacts from a pump station on the South Spur Ditch appear to be relatively small. Downstream water levels at some locations would increase by a maximum of 0.1 feet for the 10-year storm event. The increase in flow and water surface elevation downstream could increase flooding and drainage times in the downstream portion of the slough. These effects could be mitigated by implementing alternatives to address downstream flooding; however, additional modeling should be done to quantify downstream impacts for a variety of tidal conditions.

For the 25-year and 100-year storm events, additional pumping capacity would likely help to reduce the duration of flooding; however, flooding would still likely occur for these storm events due to the low elevations in this area and the limited capacity of the channel. A pump station to control the larger storm events was not analyzed in detail for this summary because of its lower expected cost/benefit ratio and the expected increase in downstream impacts.

5. Detention at the 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 and 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.

6. South Spur Ditch Bypass Channel

This bypass channel would be constructed along the Bay View Ridge hillside parallel to the South Spur Ditch. The purpose of this bypass channel would be to reduce the required capacity of the South Spur Pump Station. Because this bypass channel would be located on the hillside, a levee would need to be constructed on the downhill side of the new channel. The toe of the new bypass channel levee would be above existing South Spur Ditch.

The total length of the bypass channel would be approximately 7,400 LF, extending from the confluence of the Maben Ditch and the South Spur Ditch to the west boundary of the Kabalo Heights Plat. The bypass channel would cross property owned by the Sakuma Brothers, Ed Knutzen, Jerry Nelson, and Ray Jensen. The bypass channel was assumed to be trapezoidal in shape with the following characteristics:

- 5 to 8-foot bottom width
- 2:1 side slopes inside the channel; 4:1 slopes on the outside of the levee
- Depth varies from 10 feet at the downstream end to 8 feet at the upstream end
- A constant slope of 0.0028 percent
- Manning's 'n' roughness coefficient of 0.045

A hydraulic model of this bypass channel showed that the South Spur Pump Station would be most effective at reducing flooding from storms at of below the 10-year recurrence interval. The proposed South Spur Pump Station would still be needed to control flooding at higher recurrence intervals, but the maximum capacity could be reduced. The hydraulic analysis estimated the pump station capacity could be reduced to 20 cfs for existing conditions and 25 cfs for future conditions.

7. Alternatives Eliminated from Detailed Analysis

Two measures that may reduce flooding in the Joe Leary Basin were considered briefly but no detailed analysis was performed:

- **Upland Detention**—According to current land use projections, the upland area, especially the area within the UGA boundary, will experience an increase in impervious area under future conditions. We would expect detention facilities will be required as part of any development proposals in the upland areas. The sloping terrain may make it difficult to construct one large detention pond. However, terraced detention pond cells, such as those along Higgins-Airport Way, are possible. Detention ponds would have the effect of reducing the peak runoff, which will facilitate operation of a pump station on the South Spur Ditch.

- **Dikes**—Adding dikes to the lower portion would decrease the occurrence of flooding from flows overtopping the banks of the slough. However, flooding would still occur in adjacent fields with low ground elevations due to the lack of drainage. Pumping would be required to drain the fields during periods of high water levels in the slough. The model for this basin does not include lateral drainages from the fields and would need significant modification to analyze a diking alternative.

D. Little Indian Slough

Flooding in Little Indian Slough appears to be limited to the upper portion of the basin. This is expected due to the potential increase in impervious area within the upper portion of the basin. **Figure 7-5** shows the location of the flood reduction alternatives that were evaluated. The following conceptual alternatives are proposed to relieve flooding in the area.

1. Upstream Culvert and Channel Upgrades

Based on the modeling results, the entire drainage system upstream of Farm-to-Market Road should be upgraded. Culverts LI-C-1 and LI-C-2 both would pass the 25-year future-conditions flood if upgraded to 48-inch circular corrugated metal pipe culverts. In addition, the capacity of the slough should be increased. The improved channel was modeled as trapezoidal in shape and having the following characteristics:

- 3-foot bottom width
- 2:1 side slopes
- 3 feet of total depth
- A constant slope of 0.2 percent
- Manning's 'n' roughness coefficient of 0.050.

With these culvert and channel upgrades, all flooding upstream of Farm-to-Market Road can be eliminated. The changes would not have a significant impact on downstream flooding.

2. Upstream Regional Detention

Subbasin C-2 is the only part of the Little Indian Slough that is expected to experience development under future conditions. Therefore, detention in the upper portion of the basin to mitigate for the additional runoff was investigated.

Table 7-11 and **Table 7-12** show the expected increase in peak flow and runoff volume in Subbasin C-2 under future conditions. Based on the modeling, a pond volume of approximately 15 acre-feet is needed to attenuate flows and eliminate flooding in the upper portion of the slough. The detention pond volume is controlled by the size of the existing downstream culverts (24-inch circular).

Upstream detention, without the proposed culvert and channel improvements, would eliminate flooding upstream of Farm-to-Market Road. It would not have a significant effect on flooding downstream of Farm-to-Market Road. High tide elevations coupled with low ground elevations appear to control flooding in the lower slough.

Figure 7-6 shows the modeled effect of these flood reduction alternatives.

Table 7-11: Little Indian Slough Existing and Future Conditions Peak Flow

SWMM Model Node	Approximate Location	Peak Flow Conditions					
		No Improvements			Higgins Overflow Removed		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-10	Outlet of Little Indian Slough	83 cfs	100 cfs	121 cfs	86 cfs	103 cfs	127 cfs
LI-32	Bay View-Edison Road to Farm-to-Market Road	16 cfs	23 cfs	35 cfs	25 cfs	35 cfs	50 cfs
LI-40	Farm-to-Market Road	13 cfs	18 cfs	26 cfs	23 cfs	30 cfs	41 cfs

See **Figure 5-4** for node locations.

Table 7-12: Little Indian Slough Existing and Future Conditions Runoff Volume

Subbasin	Runoff Volume					
	Existing Conditions			Future Conditions		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
C-1a	9 ac-ft	14 ac-ft	22 ac-ft	9 ac-ft	14 ac-ft	22 ac-ft
C-1b	11 ac-ft	18 ac-ft	29 ac-ft	11 ac-ft	18 ac-ft	29 ac-ft
C-1c	4 ac-ft	6 ac-ft	9 ac-ft	4 ac-ft	6 ac-ft	9 ac-ft
C-2	12 ac-ft	18 ac-ft	27 ac-ft	17 ac-ft	23 ac-ft	32 ac-ft

See **Figure 5-4** for basin boundaries.

E. Big Indian Slough

Flooding in Big Indian Slough appears to be concentrated near the confluence of the runoff from the Urban Growth Area (including Skagit Regional Airport) and the main stem of Big Indian Slough. This confluence of large flows apparently overwhelms the existing conveyance system, specifically culverts, and causes flooding in the general vicinity. The following conceptual alternatives are proposed to relieve flooding in the area. **Figure 7-7** shows the location of the flood reduction alternatives that were evaluated.

1. State Route 20 Bypass Channel

A bypass along State Route 20 (SR 20) northeast of Bradshaw Road was examined as a way of reducing flooding of the culverts north of SR 20. Routing flow along the south side of SR 20 may reduce peak flows and flooding. It was assumed that the portion of Subbasin C-3 that is south of SR 20 would drain directly into the bypass. The bypass channel was assumed to be trapezoidal in shape with a length of approximately 3,100 LF. The following characteristics were used to define the bypass:

- 5-foot bottom width
- 3:1 side slopes
- 9 feet of total depth
- A constant slope of 0.1 percent
- Manning's 'n' roughness coefficient of 0.045.

According to the modeling results, the bypass would have the effect of reversing flow in the portion of the slough east of the inflow point from the Port's property (Node BI-200). The peak flow from the Port's stormwater facility along Higgins Airport Way (Subbasin C-7) lags slightly behind the peak flow from Subbasins C-4 and C-5. As the slough downstream of the inflow point fills to capacity, water would begin to flow east into the extra capacity of that channel and flow through the bypass as it drains to the outfall. This would reduce flooding along that portion of the slough for a normal tide cycle.

In general, the bypass lowers water surface elevations by less than 1 foot in the upper portion of the model, but some flooding was eliminated. Based on the topographic mapping, the remaining flooding in this portion of the slough does affect some agricultural fields in this area, but there is no impact to any homes, structures or major roads. Flooding would be limited to overtopping the smaller field access culverts and would largely remain in the slough corridor. Elevations in the fields are on the order of 12.5 feet, and the water surface elevation for the 100-year future conditions storm would not exceed 11 feet.

According to the modeling results, the Higgins Airport Way culvert (BI-C-4) would no longer flood at the 100-year existing or future conditions event with the SR 20 Bypass Channel in place. Overtopping would occur at culverts BI-C-2, BI-C-3, and BI-C-5. Culvert capacity does not appear to be a direct cause of flooding at culvert BI-C-2, but replacing the three culverts at this location with one large culvert would improve hydraulic efficiency. At this location, flooding appears to occur as a result of the backwater downstream and the low overtopping elevation of the culvert (approximately 7.5 feet). Flooding at culverts BI-C-3 and BI-C-5 appears to be a result of the culverts being undersized and their low overtopping elevation. The main function of these culverts is to provide access to the agricultural fields between the slough and SR 20. Due to the limited traffic that uses this access, flooding at the recurrence intervals seen here may be acceptable.

2. Ovenell Road Bypass Channel

A more effective solution to reduce flooding in the upper reaches of the Big Indian Slough Basin is to construct a new bypass channel along Ovenell Road that starts west of the Skagit Golf & Country Club and extends west and south, connecting to Big Indian Slough near the intersection with State Routes 20 and 536. The purpose of this bypass channel would be to divert stormwater runoff from the entering Big Indian Slough near the location of several undersized culverts and low farm fields.

Discharge from the new bypass channel would enter the Big Indian Slough near the beginning of the manmade section of the existing channel. Below this point there appears to be sufficient capacity to accommodate additional stormwater runoff.

The total length of the Ovenell Road Bypass Channel is approximately 11,600 LF. The following characteristics were used to define the bypass:

- 3½ to 6-foot bottom width
- 2:1 side slopes
- Depth varies from 8 feet at the downstream end to 3.5 feet at the upstream end
- A constant slope of 0.1 percent
- Manning's 'n' roughness coefficient of 0.045

Table 7-13 and **Table 7-14** show the modeled effect of the bypass channel on water surface elevations for existing and future conditions, respectively. In general, the bypass channel lowers water surface elevations by 1 to 4 feet in the upper portion of the basin that was modeled. Based on the topographic mapping, the remaining flooding in this portion of the slough does not appear to significantly affect the agricultural fields in this area, nor does it impact any homes, structures or major roads. Flooding would be limited to overtopping the smaller culverts and would largely remain in the slough corridor. Elevations in the fields are on the order of 12.5 feet, and the water surface elevation for the 100-year future conditions storm would not exceed 8.3 feet.

The environmental concerns of this bypass channel are significant. The Big Indian Slough is a fish bearing watershed. Therefore, any channel construction and improvements will require accommodation of fisheries habitat. This may include, but not limited to, construction of ponds, fish friendly culverts and spawning beds, and establishment of vegetated buffers. Permitting negotiations for this project are expected to take several years.

Table 7-13: Big Indian Slough Existing Conditions Water Surface Elevations With and Without Bypass Channel

SWMM Model Node	Water Surface Elevation No Improvements			Water Surface Elevation With Bypass Channel			Difference		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Main Stem - Below SR 20									
BI-20	3.0 feet	3.7 feet	3.8 feet	3.1 feet	3.7 feet	3.8 feet	0.1 feet	0.0 feet	0.0 feet
BI-30	3.0 feet	3.7 feet	3.9 feet	3.1 feet	3.7 feet	3.9 feet	0.1 feet	0.0 feet	0.0 feet
BI-40	3.0 feet	3.8 feet	3.9 feet	3.1 feet	3.8 feet	3.9 feet	0.1 feet	0.0 feet	0.0 feet
BI-50	3.1 feet	3.8 feet	3.9 feet	3.1 feet	3.8 feet	3.9 feet	0.0 feet	0.0 feet	0.0 feet
BI-60	3.1 feet	3.8 feet	4.0 feet	3.1 feet	3.8 feet	4.0 feet	0.0 feet	0.0 feet	0.0 feet
BI-70	3.2 feet	3.9 feet	4.1 feet	3.2 feet	3.9 feet	4.1 feet	0.0 feet	0.0 feet	0.0 feet
BI-80	3.2 feet	4.0 feet	4.2 feet	3.2 feet	4.0 feet	4.2 feet	0.0 feet	0.0 feet	0.0 feet
BI-90	3.2 feet	4.0 feet	4.2 feet	3.3 feet	4.0 feet	4.2 feet	0.1 feet	0.0 feet	0.0 feet
BI-100	3.7 feet	4.6 feet	5.0 feet	3.7 feet	4.5 feet	5.0 feet	0.0 feet	-0.1 feet	0.0 feet
BI-110	4.4 feet	5.4 feet	6.0 feet	4.4 feet	5.3 feet	6.0 feet	0.0 feet	-0.1 feet	0.0 feet
Main Stem - Above SR 20									
BI-120	4.4 feet	5.4 feet	6.0 feet	4.4 feet	5.3 feet	6.0 feet	0.0 feet	-0.1 feet	0.0 feet
BI-130	5.5 feet	6.8 feet	7.6 feet	4.4 feet	5.4 feet	6.1 feet	-1.1 feet	-1.4 feet	-1.5 feet
BI-140	6.4 feet	7.9 feet	8.8 feet	4.5 feet	5.5 feet	6.3 feet	-1.9 feet	-2.4 feet	-2.5 feet
BI-150	6.5 feet	7.9 feet	8.9 feet	4.5 feet	5.5 feet	6.3 feet	-2.0 feet	-2.4 feet	-2.6 feet
BI-160	6.9 feet	8.4 feet	9.4 feet	4.6 feet	5.6 feet	6.3 feet	-2.3 feet	-2.8 feet	-3.1 feet
BI-170	6.9 feet	8.4 feet	9.4 feet	4.6 feet	5.6 feet	6.4 feet	-2.3 feet	-2.8 feet	-3.0 feet
BI-180	7.4 feet	8.9 feet	10.6 feet	4.7 feet	5.7 feet	6.5 feet	-2.7 feet	-3.2 feet	-4.1 feet
BI-190	7.5 feet	9.0 feet	10.7 feet	4.7 feet	5.7 feet	6.5 feet	-2.8 feet	-3.3 feet	-4.2 feet
JL-200	7.8 feet	9.3 feet	10.9 feet	5.0 feet	5.7 feet	6.5 feet	-2.8 feet	-3.6 feet	-4.4 feet
JL-210	7.8 feet	9.4 feet	11.0 feet	5.1 feet	5.8 feet	6.6 feet	-2.7 feet	-3.6 feet	-4.4 feet
JL-220	7.8 feet	9.4 feet	11.1 feet	5.9 feet	6.3 feet	6.9 feet	-1.9 feet	-3.1 feet	-4.2 feet
JL-230	7.8 feet	9.4 feet	11.1 feet	5.9 feet	6.3 feet	6.9 feet	-1.9 feet	-3.1 feet	-4.2 feet
JL-240	7.8 feet	9.4 feet	11.1 feet	5.9 feet	6.4 feet	7.0 feet	-1.9 feet	-3.0 feet	-4.1 feet
JL-250	7.8 feet	9.4 feet	11.1 feet	6.0 feet	6.4 feet	7.0 feet	-1.8 feet	-3.0 feet	-4.1 feet
JL-260	7.8 feet	9.4 feet	11.1 feet	6.0 feet	6.5 feet	7.1 feet	-1.8 feet	-2.9 feet	-4.0 feet
JL-270	7.9 feet	9.4 feet	11.1 feet	6.2 feet	6.7 feet	7.3 feet	-1.7 feet	-2.7 feet	-3.8 feet

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

Table 7-14: Big Indian Slough Future Conditions Water Surface Elevations With and Without Bypass Channel									
SWMM Model Node	Water Surface Elevation No Improvements			Water Surface Elevation With Bypass Channel			Difference		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Main Stem - Below SR 20									
BI-20	3.4 feet	3.8 feet	3.9 feet	3.3 feet	3.8 feet	3.8 feet	-0.1 feet	0.0 feet	-0.1 feet
BI-30	3.4 feet	3.8 feet	3.9 feet	3.3 feet	3.8 feet	3.9 feet	-0.1 feet	0.0 feet	0.0 feet
BI-40	3.4 feet	3.8 feet	3.9 feet	3.4 feet	3.8 feet	3.9 feet	0.0 feet	0.0 feet	0.0 feet
BI-50	3.4 feet	3.8 feet	3.9 feet	3.4 feet	3.8 feet	3.9 feet	0.0 feet	0.0 feet	0.0 feet
BI-60	3.4 feet	3.9 feet	4.0 feet	3.4 feet	3.9 feet	4.0 feet	0.0 feet	0.0 feet	0.0 feet
BI-70	3.5 feet	4.0 feet	4.1 feet	3.5 feet	4.0 feet	4.1 feet	0.0 feet	0.0 feet	0.0 feet
BI-80	3.5 feet	4.0 feet	4.2 feet	3.5 feet	4.0 feet	4.2 feet	0.0 feet	0.0 feet	0.0 feet
BI-90	3.6 feet	4.0 feet	4.3 feet	3.5 feet	4.1 feet	4.3 feet	-0.1 feet	0.1 feet	0.0 feet
BI-100	4.0 feet	4.7 feet	5.2 feet	3.9 feet	4.7 feet	5.1 feet	-0.1 feet	0.0 feet	-0.1 feet
BI-110	4.7 feet	5.5 feet	6.2 feet	4.6 feet	5.5 feet	6.1 feet	-0.1 feet	0.0 feet	-0.1 feet
Main Stem - Above SR 20									
BI-120	5.8 feet	7.0 feet	7.9 feet	4.7 feet	5.6 feet	6.3 feet	-1.1 feet	-1.4 feet	-1.6 feet
BI-130	6.8 feet	8.1 feet	9.3 feet	4.9 feet	5.7 feet	6.6 feet	-1.9 feet	-2.4 feet	-2.7 feet
BI-140	6.8 feet	8.1 feet	9.3 feet	4.9 feet	5.7 feet	6.6 feet	-1.9 feet	-2.4 feet	-2.7 feet
BI-150	7.3 feet	8.6 feet	9.9 feet	5.1 feet	5.9 feet	6.8 feet	-2.2 feet	-2.7 feet	-3.1 feet
BI-160	7.3 feet	8.7 feet	9.9 feet	5.1 feet	6.0 feet	6.8 feet	-2.2 feet	-2.7 feet	-3.1 feet
BI-170	7.8 feet	9.2 feet	10.9 feet	5.4 feet	6.2 feet	7.1 feet	-2.4 feet	-3.0 feet	-3.8 feet
BI-180	7.9 feet	9.3 feet	11.1 feet	5.4 feet	6.2 feet	7.1 feet	-2.5 feet	-3.1 feet	-4.0 feet
BI-190	8.1 feet	9.6 feet	11.3 feet	5.7 feet	6.4 feet	7.3 feet	-2.4 feet	-3.2 feet	-4.0 feet
JL-200	8.2 feet	9.7 feet	11.5 feet	5.8 feet	6.5 feet	7.4 feet	-2.4 feet	-3.2 feet	-4.1 feet
JL-210	8.3 feet	9.7 feet	11.5 feet	6.4 feet	7.0 feet	7.8 feet	-1.9 feet	-2.7 feet	-3.7 feet
JL-220	8.3 feet	9.7 feet	11.5 feet	6.4 feet	7.0 feet	7.8 feet	-1.9 feet	-2.7 feet	-3.7 feet
JL-230	8.3 feet	9.7 feet	11.5 feet	6.5 feet	7.0 feet	7.8 feet	-1.8 feet	-2.7 feet	-3.7 feet
JL-240	8.3 feet	9.7 feet	11.5 feet	6.5 feet	7.1 feet	7.9 feet	-1.8 feet	-2.6 feet	-3.6 feet
JL-250	8.3 feet	9.8 feet	11.5 feet	6.6 feet	7.2 feet	8.0 feet	-1.7 feet	-2.6 feet	-3.5 feet
JL-260	8.4 feet	9.8 feet	11.5 feet	6.8 feet	7.4 feet	8.2 feet	-1.6 feet	-2.4 feet	-3.3 feet
JL-270	8.4 feet	9.8 feet	11.5 feet	6.9 feet	7.5 feet	8.3 feet	-1.5 feet	-2.3 feet	-3.2 feet

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

3. Downstream Detention

A detention pond located near the outlet of the slough was originally proposed as a means of mitigating additional runoff volume from future land uses. A pond at this location could absorb additional volume and lower water surface elevations when the tide gates are closed. The area of the parcel considered for storage is approximately 23 acres. The average elevation of the ground surface in this parcel is 3.3 feet, although there are some areas as low as 2.5 feet. Two detention options were evaluated: one using the existing field level for storage and one using an excavated pond.

Field Level Storage: For modeling purposes, it was assumed that 5 acres would be available for storage at an elevation of 2.5 feet and the full 23 acres would be available at elevations above 3.3 feet.

The results of the modeling indicate that field level detention would have no impact on the overflow to Higgins Slough or on the upstream flooding culverts for the modeled tide cycle. **Table 7-15** shows predicted water surface elevations. The modeling predicts a peak water surface elevation of 3.2 feet at the slough outlet for the 100-year storm event. At this elevation, the detention pond would provide approximately 8 acre-feet of storage, which does not appear to be enough to impact the water surface elevations in the slough.

Table 7-15: Big Indian Slough Field Level Detention Analysis Results

SWMM Model Node	Approximate Location	Future Conditions Water Surface Elevation					
		Without Detention			With Field Level Detention		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet of Big Indian Slough	3.0 feet	3.2 feet	3.2 feet	3.0 feet	3.2 feet	3.2 feet
BI-90	Farm-to-Market Road	3.1 feet	3.4 feet	3.6 feet	3.0 feet	3.4 feet	3.6 feet
BI-120	Higgins Slough Overflow	4.4 feet	5.4 feet	6.1 feet	4.4 feet	5.4 feet	6.1 feet
BI-200	Higgins Airport Way	8.1 feet	9.6 feet	10.9 feet	8.1 feet	9.6 feet	10.9 feet
BI-270	Upstream Model Boundary	8.2 feet	9.7 feet	11.1 feet	8.2 feet	9.7 feet	11.1 feet

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

Excavated Detention: For this analysis, the pond was assumed to be excavated to a constant elevation of 0 feet. It was assumed that below this elevation the pond would not drain consistently and any additional excavation would only provide dead storage. This analysis assumes that the full 23 acres would be excavated, providing approximately 92 acre-feet of storage at a water surface elevation of 4 feet. **Table 7-16** shows predicted water surface elevations.

The results of the modeling indicate that for the modeled tide cycle excavated detention would have no impact on the overflow to Higgins Slough or on the upstream flooding culverts, although peak water surface elevations would be reduced along the flatter lower portion of the slough. The detention pond would provide approximately 80 acre-feet of storage for the 100-year future conditions storm event, which is approximately 16 percent of the total volume of water passing through the outfall for that event. This suggests that under the modeled tidal conditions the overflow to Higgins Slough and the flooding upstream are not controlled by volume, but by the capacity of the channel to handle peak flows at those locations.

Table 7-16: Big Indian Slough Excavated Detention Analysis Results							
SWMM Model Node	Approximate Location	Future Conditions Water Surface Elevation					
		Without Detention			With Excavated Level Detention		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet of Big Indian Slough	3.0 feet	3.2 feet	3.2 feet	2.0 feet	2.3 feet	2.5 feet
BI-90	Farm-to-Market Road	3.1 feet	3.4 feet	3.6 feet	2.4 feet	2.8 feet	3.2 feet
BI-120	Higgins Slough Overflow	4.4 feet	5.4 feet	6.1 feet	4.4 feet	5.4 feet	6.1 feet
BI-200	Higgins Airport Way	8.1 feet	9.6 feet	10.9 feet	8.1 feet	9.6 feet	10.9 feet
BI-270	Upstream Model Boundary	8.2 feet	9.7 feet	11.1 feet	8.2 feet	9.7 feet	11.1 feet
See **Figure 5-5** for node locations.							

4. Slough Capacity Analysis

Of particular interest in this basin is the capacity of the slough to handle additional flows. Currently under high-flow conditions, a portion of the runoff from Big Indian Slough flows into Higgins Slough. Because of the severity of flooding in Higgins Slough, the capacity of Big Indian Slough to handle all runoff from within the basin without overflowing to Higgins Slough is important. For this analysis, the overflow to Higgins Slough was removed to determine what impacts, if any, this would have on Big Indian Slough.

The modeling shows that under the modeled tidal condition removal of the Higgins Slough overflow would not increase the frequency of flooding in the system. **Table 7-17** and a **Table 7-18** compare the water surface elevations for existing and future conditions, respectively, at select points along the length of the slough if the Higgins overflow were to be eliminated.

Table 7-17: Higgins Overflow Removal Results - Existing Conditions							
SWMM Model Node	Approximate Location	Existing Conditions Water Surface Elevation					
		No Improvements			Higgins Overflow Removed		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet of Big Indian Slough	3.0 feet	3.2 feet	3.2 feet	3.0 feet	3.2 feet	3.2 feet
BI-90	Farm-to-Market Road	3.1 feet	3.4 feet	3.6 feet	3.1 feet	3.5 feet	3.6 feet
BI-120	Higgins Slough Overflow	4.1 feet	5.2 feet	5.8 feet	4.2 feet	5.4 feet	6.1 feet
BI-200	Higgins Airport Way	7.7 feet	9.3 feet	10.8 feet	1.1 feet	9.3 feet	10.9 feet
BI-270	Upstream Model Boundary	7.8 feet	9.4 feet	11.0 feet	7.8 feet	9.4 feet	11.1 feet
See **Figure 5-5** for node locations.							

Table 7-18: Higgins Overflow Removal Results - Future Conditions							
SWMM Model Node	Approximate Location	Future Conditions Water Surface Elevation					
		No Improvements			Higgins Overflow Removed		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet of Big Indian Slough	3.0 feet	3.2 feet	3.2 feet	3.0 feet	2.3 feet	3.2 feet
BI-90	Farm-to-Market Road	3.1 feet	3.4 feet	3.6 feet	3.2 feet	3.5 feet	3.7 feet
BI-120	Higgins Slough Overflow	4.4 feet	5.4 feet	6.1 feet	4.5 feet	5.6 feet	6.5 feet
BI-200	Higgins Airport Way	8.1 feet	9.6 feet	10.9 feet	8.1 feet	9.6 feet	11.0 feet
BI-270	Upstream Model Boundary	8.2 feet	9.7 feet	11.1 feet	8.2 feet	9.8 feet	11.2 feet
See **Figure 5-5** for node locations.							

Although the modeling shows that water surface elevations would increase if the Higgins Slough overflow were removed, the impact on the slough would not be significant. Flood elevations would increase, but the frequency of flooding would remain the same for both existing and future conditions.

This result is due to the fact that Big Indian Slough's outlet structure appears to have the capacity to convey significant flow—in excess of 400 cfs—with minimal head loss. This allows the slough to pass the peak flows without causing a significant backwater effect. Because the slough has levees in the lower reach to elevations in the range of 6.5 to 9.0 feet, the water surface elevation in the slough can increase to a level that allows the slough to flow at high tides, without causing flooding. The levees extend upstream to a point where ground elevations exceed flood stage, except in the case of the low culverts previously identified.

In order to verify this result, the capacity of the slough and its relationship to the outlet conditions was examined in more detail in two additional analyses:

- The software program CulvertMaster was used to check the capacity of the outlet culverts.
- A sensitivity analysis was conducted to determine if variations in the outlet boundary conditions would have a significant impact on the upstream water surface elevations.

a. Outlet Capacity Analysis

The software program CulvertMaster, developed by Haested Methods, was used to evaluate the capacity of the seven 48-inch CMP culverts at the outlet of the slough. Based on the survey data, the following parameters were used to model the culverts:

- The upstream and downstream invert elevations of the culverts are at approximately -3.5 and -3.6 feet, respectively.
- The culverts are approximately 5 feet long.
- A Manning's 'n' value of 0.024 was used.
- A constant tailwater elevation of 4.0 feet was used, 0.2 feet above the mean higher high water elevation used in the modeling.

Table 7-18 shows a rating table that was developed by varying the headwater elevation. The results indicate that the outlet culverts have a capacity of approximately 427 cfs with 1 foot of positive head (headwater elevation of 5 feet). This result is consistent with the XP-SWMM results, which indicate the outlet culverts have a high capacity even at high tide. In addition, if the tide rises higher than 4 feet, there is sufficient freeboard along the length of the slough to accommodate a water surface elevation that would generate sufficient head to keep the slough draining until the tide goes down.

Table 7-19: Big Indian Slough Outlet Culvert Rating Table

Headwater Elevation	Tailwater Elevation	Culvert Capacity
4.2 feet	4.0 feet	189 cfs
4.6 feet	4.0 feet	329 cfs
5.0 feet	4.0 feet	427 cfs
5.5 feet	4.0 feet	525 cfs

b. Water Surface Elevation Sensitivity Analysis

To estimate the sensitivity of the simulation to changes in tailwater conditions at the slough outlet, tidal elevations were adjusted by adding 3 feet to the tidal cycle. The tide cycled from a low tide of approximately 2.5 feet to a high tide of 6.8 feet. The model was only run for the 100-year, future conditions event. With this tide cycle, minor flooding was observed at Node BI-230 for the 100-year future conditions event. The slough appeared to have sufficient capacity to handle the 100-year event even for this condition.

These results indicate that there is enough capacity in the slough to pass the 100-year future conditions flood without causing any additional flooding in the slough. According to available survey data, the elevation of the top of the dike ranges from 6.5 to 9.0 feet.

Chapter 8
Capital Improvement Plan

The stormwater projects presented here are proposed for consideration to reduce or eliminated existing and/or future flooding conditions within the Bay View Watershed area. Some projects are simple, consisting of replacing or upsizing existing outfalls and 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 two primary sources, the Drainage District's property assessments and the Skagit County Drainage Utility. A breakdown of estimated financial contributions by these two 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 a very gross funding "goals". Detailed project cost estimates will need to be developed during the project planning and design phases.

All project costs are adjusted to January 2006 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 [CWCCIS]*[16] prepare by the US Army Corp of Engineers was used to adjust historical construction cost to January 2006 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.

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.

[16] *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, 2005)

2. Pump Station Construction Costs

Nine 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 2006 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.

$$\text{Pump Station Project Cost} = 0.0634 \times (\text{Capacity})^{0.8054}$$

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. Outfalls and Culverts

Proposed outfalls and culverts are estimated based on a schematic layout. Construction costs are estimated based on 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, 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 outfalls or culverts. The historical costs were adjusted to 2006 cost using the CWCCIS Cost Index.

4. Channels and Detention Ponds

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, 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 8-1 provides a proposed Capital Improvement Plan for planning, design and construction of the stormwater drainage facilities in the Bay View Watershed. 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 5 years has been approximately 4 percent for the *Civil Works Construction Cost Index*.

Table 8-1: Recommended Capital Improvements for the Bay View Watershed

Drainage Basin and Proposed Stormwater Capital Improvement	Project Cost Estimate	Projected Capital Imporvement Costs with Escalation					
		FY 2006	FY 2007	FY 2008	FY 2009	FY 2010	FY 2011
No Name Slough Basin							
Outfall Culvert Addition NN-C-Out	$ 100,000	$ 25,000	$ 26,000	$ 27,000	$ 28,000		
100-cfs Outfall Pump Station	$ 2,600,000			$ 541,000	$ 2,362,000		
84" Culvert Replacement NN-C-3	$ 21,000		$ 22,000				
54" Culvert Replacement NN-C-5	$ 14,000		$ 15,000				
32 ac-ft Marihugh Road Detention Pond	$ 1,257,000	$ 250,000	$ 260,000	$ 819,000			
Joe Leary Slough Basin							
Peth Property Bypass Channel	$ 820,000	$ 164,000	$ 171,000	$ 492,000			
300-cfs Outfall Pump Station	$ 6,300,000			$ 681,000	$ 709,000	$ 2,948,000	$ 3,066,000
60-cfs South Spur Pump Station	$ 1,700,000	$ 340,000	$ 1,414,000				
Little Indian Slough Basin							
Incease Channel Capacity Upstream of Farm-to-Market Rd	$ 45,000	$ 45,000					
48" Culvert Replacement LI-C-1	$ 56,000	$ 56,000					
48" Culvert Replacement LI-C-2	$ 23,000	$ 23,000					
Big Indian Slough Basin							
Ovenell Road Bypass Channel	$ 4,800,000	$ 800,000	$ 2,080,000	$ 2,163,000			
Modify Higgins Slough Bypass	$ 50,000	$ 50,000					
Totals	$ 17,786,000	$ 1,753,000	$ 3,988,000	$ 4,723,000	$ 3,099,000	$ 2,948,000	$ 3,066,000

1. No Name Slough Recommendations

All alternatives evaluated for this document were analyzed individually, without considering potential combinations of the alternatives. Before any project is carried forward, the hydraulic model should be updated to account for any other projects that have been implemented 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 alternatives are recommended for the No Name Slough basin:

a. Increased Pumping Capacity

Pumping remains the best option for reducing the flooding in the slough's lowland areas. A screw-type pump, where the water does not flow through an impeller or pump housing, would be the most ecologically sensitive and fish-friendly pump station. All subbasins would contribute to the outfall pump station.

b. Detention Pond at Marihugh Road

Marihugh Road appears to provide the best location for a detention pond. The ground is relatively flat and drainage from a large portion of the basin can be collected. Detention at Marihugh Road would reduce peak flows in No Name Creek, reducing erosion in the creek and sedimentation in the slough. Subbasins A-11b would contribute to a detention pond in this location.

c. Culvert Replacement

Replacement of all the undersized culverts is recommended. The recommended culvert sizes are based on the results of the hydraulic modeling. Additional local topography is needed as part of the final design to verify that the specified shape and material are appropriate for that location. Since it was not known which if any culverts would be replaced, the culverts were analyzed singly, assuming that no other culverts or structures had been updated. As part of the culvert design and construction, the hydraulic models should be updated with the most recent information. All subbasins would contribute to any additional outfall culverts. All subbasins, except subbasins A-4, A-5, and A-12, would contribute to Culvert NN-C-3. Only subbasins A-7 and A-8 would contribute to culvert NN-C-5.

Regional detention at subbasin A-11a is not recommended at this time. The County already requires new developments to match existing peak runoff; therefore the County should work with developers to ensure that these regulations are met. A new regional detention pond for subbasin A-8 will reduce the impact along the south stem from proposed development on the Port Property. Modifications to the existing detention pond on the Paccar Technical Center (subbasin A-7) may provide some benefit in reducing ditch erosion along Farm-to-Market Road.

2. Joe Leary Slough Recommendations

As with the other drainage basins in the Bay View Watershed area, 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, the number of alternatives that can reduce flooding are limited. The following alternatives are recommended:

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 bypass would circumvent the restriction along D'Arcy Road where the channel is confined by the road and could lower water surface elevations in portions of the slough by over 1 foot. All subbasins, except subbasins B-1a and B-1b, would benefit from the slough bypass.

b. Pump Station at the Outlet

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. Outlet Detention Pond

This is not a separate capital improvement, but something to consider when studying the Outlet Pump Station. Detention at the outlet was examined briefly and should be evaluated further as part of any outlet pump station project. Given the large volume of water that is generated during a peak event, considerable storage would be required to have an effect, and because ground elevations near the outlet are so low, a very large storage area would be needed. However, increased storage capacity in the slough could reduce the maximum pumping capacity needed to reduce flooding. If land were available for storage and if the construction cost and operation of a pump station at the outlet exceeds the available budget constraints, this option could be explored during the design phase of the outlet pump station as a way to reduce overall project costs.

d. Pump Station on South Spur Ditch

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.

A pump station with a reduced capacity could be constructed if the South Spur Bypass Channel is constructed. However, it may take several years to acquire the necessary right-of-way and environmental permits before this bypass channel can be constructed. A smaller pump station should be expandable in case the bypass channel does not became a reality.

Before any project is implemented, the hydraulic analysis 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.

3. Little Indian Slough Recommendations

Below 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 channel and culverts. Modeled results with upgrades to the channel and culvert capacity in the upper slough did not consider the effects of any existing detention. Therefore the result may be conservative.

Before any project is implemented, the analysis presented in this document should be updated to account for any new projects 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 alternative is recommended:

a. Culvert Replacement and Increase Channel Capacity

Culvert replacement and channel enlargement appears to be the most cost-effective alternative in reducing flooding upstream of Farm-to-Market Road. Although not specifically examined, 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 would 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, given the cost involved in a flood reduction project of the required scale.

4. Big Indian Slough Recommendations

Of the four basins modeled as part of this project, Big Indian Slough had the least information available to verify and calibrate the model. Therefore, it is recommended that before any projects are implemented in this basin, additional hydrologic data is collected to be used in model calibration or the model results be closely examined by the stakeholders who are most familiar with historical conditions in the basin.

Before any project is 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.

The following alternatives are recommended:

a. Ovenell Road Bypass Channel

The Ovenell Road Bypass Channel appears to have the ability to relieve the hydraulic bottleneck caused by the culvert pipes that exists within the existing drainage slough. According to the modeling results, the bypass channel would lower the water surface elevation in the upper portion of the existing slough by more than 1 foot and would eliminate flooding at most locations. If possible, additional topographic data collection is recommended to verify and/or calibrate the model in this area before the bypass is constructed. All subbasins, except subbasins C-2a and C-6, would benefit from construction of this new channel section. In addition, the project should be modeled at a variety of tidal conditions to better understand its effects.

b. Modify Higgins Slough Overflow

Since Higgins Slough currently experiences flooding, and because the lower reaches of Big Indian Slough appears to have the capacity to handle the additional flow, it is recommended that the this overflow be modified to control downstream flooding in Higgins Slough.

The Ovenell Road Bypass Channel was selected over the SR 20 Bypass Channel with field culvert replacement because it showed higher reductions in water surface elevations in the upper reaches of Big Indian Slough. The Ovenell Road Bypass Channel also directly serves the commercial areas within the Bay View ridge UGA. However, if right-of-way acquisition or other issue regarding the Ovenell Road Bypass Channel delays the project construction, then the SR 20 Bypass Channel is a viable alternative to consider.

Detention at the outlet of the slough is also not recommended at this time. While outlet detention would provide additional storage volume in the slough and might lower slough water surface elevations at the outlet, the benefits do not extend far enough upstream to impact any documented problem areas. Additional pumping may help reduce flooding during extended periods of high tide, however, this alternative was not analyzed and would likely require a pump station with a capacity of several hundred cfs. In fact, since the capacity of the pump station is far less than the outlet culverts, an analysis to determine if the pump station at the outlet is necessary seems warranted. Additional reduction in water surface elevation along the slough could be achieved by increasing hydraulic efficiency and increasing capacity in the middle and upper portions of the slough.

C. Stormwater Management Strategies

There are several stormwater management strategies that are recommended to be instituted in the Bay View Watershed. These strategies are intended to help ensure that the existing and future drainage facilities are adequately maintained so they will serve their purpose when a storm event occurs. The costs allocation in future years has been escalated to account for an estimated 4% per year inflation rate.

Table 8-2: Recommended Stormwater Management Strategies for the Bay View Watershed							
Stormwater Management Program Items	6-Year Program Estimate	Projected Stormwater Management Program Costs with Escalation					
		FY 2006	FY 2007	FY 2008	FY 2009	FY 2010	FY 2011
Entire Bay View Watershed							
Negotiate Interlocal Agreements with Drainage Districts	$ 50,000	$ 25,000	$ 26,000				
Develop Bay View Watershed Stormwater Coordination Plan	$ 25,000	$ 25,000					
Negotiate Floodway Easements	$ 25,000	$ 25,000					
Develop Bay View Watershed Stormwater Monitoring Plan	$ 100,000	$ 50,000	$ 10,000	$ 11,000	$ 11,000	$ 12,000	$ 12,000
Revise, Expand and Update Hydraulic Model	$ 100,000		$ 21,000	$ 22,000	$ 22,000	$ 23,000	$ 24,000
No Name Slough Basin							
Slough and Channel Cleaning and Maintenance	$ 54,000	$ 9,000	$ 9,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000
Pump Station Operation and Maintenance	$ 80,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000	$ 23,000	$ 24,000
Joe Leary Slough Basin							
Slough and Channel Cleaning and Maintenance	$ 216,000	$ 36,000	$ 37,000	$ 39,000	$ 40,000	$ 42,000	$ 44,000
Pump Station Operation and Maintenance	$ 50,000		$ 10,000	$ 11,000	$ 11,000	$ 12,000	$ 12,000
Little Indian Slough Basin							
Slough and Channel Cleaning and Maintenance	$ 54,000	$ 9,000	$ 9,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000
Big Indian Slough Basin							
Slough and Channel Cleaning and Maintenance	$ 180,000	$ 30,000	$ 31,000	$ 34,000	$ 38,000	$ 44,000	$ 54,000
Pump Station Operation and Maintenance	$ 60,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000	$ 12,000	$ 12,000
Totals	$ 994,000	$ 229,000	$ 173,000	$ 159,000	$ 164,000	$ 190,000	$ 204,000

1. Negotiate Interlocal Agreements with Drainage Districts

The County Commissioners should authorize the County Drainage Utility to negotiate interlocal agreements with the Dike and Drainage District No. 12, Drainage District No. 14, and Drainage District No. 19. These interlocal agreements would layout the framework for cost sharing on capital improvement projects, maintenance responsibilities between the County and the Drainage Districts, and reimbursement costs for maintenance of joint owned facilities. It is anticipated that the County would hire a financial consultant to assist with issues such as buy-in charges, impact fees, and debt financing.

2. Develop the Bay View Watershed Stormwater Coordination Plan

Several stakeholders, specifically the Drainage Districts, expressed an interest in developing a framework that facilitates an ongoing dialog regarding stormwater issues for new developments within the Bay View Watershed. This coordination element would take place during the permit review stage of a proposed project and would involve the developer, the Skagit County Planning & Development Services, the Skagit County Drainage Utility, and the impacted Drainage District.

3. Negotiate Floodway Easements

A floodway easement is a management tool that can be examined for application in any of the Bay View drainage basins. A floodway easement is a negotiated agreement between a drainage control party, such as the Skagit County Drainage Utility or the Drainage District, and a property owner. The floodway easement would describe the potential area that may be flooded during 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 facilities. Flooding easements may also be used as temporary measures to provide financial protection to property owners now while storm drainage improvements are studied, designed and constructed.

4. Develop the Bay View Watershed Stormwater Monitoring Plan

One characteristic of this stormwater study is that there is no physical rainfall data with corresponding channel flow rate data in order to calibrate the hydraulic model. A Stormwater Monitoring Plan would describe the framework for installation of stormwater measuring equipment and ongoing monitoring.

5. Revise, Expand and Update the Hydraulic Model

Four hydraulic models, one for each of the four drainage basins, were developed as part of this stormwater study. The hydraulic models were used to evaluate stormwater drainage facility options. As drainage facilities are constructed and physical stormwater runoff data is collected, the hydraulic models will need to be revised, expanded and updated. The hydraulic models can then be used to evaluate the effectiveness of constructed drainage facility as well as examine additional drainage facilities.

6. Slough and Channel Cleaning and Maintenance

Slough and channel cleaning and maintenance is an essential element in reducing the flooding potential within the drainage basins. These sloughs and channels are the major drainage facilities for properties both inside and outside the Drainage District Boundaries. In the past sloughs and channels have been cleaned and maintained solely by the Drainage Districts. The Skagit County Drainage Utility has a responsibility to financially contribute to the cleaning and maintenance of the sloughs and channels. Each Drainage District needs to enter into an interlocal agreement with Skagit

County to layout the framework for reimbursement of slough and channel cleaning and maintenance costs.

7. Pump Station Operation and Maintenance

There are two exiting stormwater pump stations and proposals for construction and/or expansion of additional pump stations. These pump station serve properties both inside and outside of the Drainage District boundaries. In the past existing pump station operation and maintenance has been performed solely by the Drainage Districts. Skagit County has a responsibility to financially contribute to the operation and maintenance of the existing and future pump stations. Each Drainage District need to enter into an interlocal agreement with Skagit County to layout the framework for reimbursement of pump station operation and maintenance costs.

8. NPDES Phase II Permitting

The issuance of a NPDES General Permit for Municipal Storm Sewers (Phase II) in late 2006 or early 2007 will increase the rules and regulations local governments must follow concerning the water quality of the stormwater in their drainage systems. This will have impacts, including financial, on Skagit County, the Drainage Districts, the City of Burlington, and the Port of Skagit, however, the extent of those impacts are not known at this time.