Bay View Watershed Stormwater Management Plan Phase 1:

The Bayview Ridge Urban Growth Area

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Board of County Commissioners

Sharon Dillon, Chair
Don Munks
Ken Dahlstedt

Skagit County Public Works

David R. Brookings, Natural Resources Division Manager
Ric Boge, Surface Water Manager
Janice Flagan, Assistant Surface Water Manager
Erick Aurand, Drainage Utility Coordinator

Consultants

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

Technical Contributors

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

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Chapter 1
Executive Summary

The Bay View Watershed's first land use plan was prepared in 1965, though with a limited understanding of the impacts of stormwater runoff. Skagit County is preparing the Bayview Ridge Subarea Plan that will guide growth within a portion of the Bayview Ridge. This Subarea is 4,011 acres. Because of the potential for further development within the Bayview Ridge, several stakeholders within the watershed have expressed concerns regarding the quality and quantity of stormwater being discharged to the adjacent sloughs and Padilla Bay. To respond to these concerns, the Skagit County Drainage Utility has embarked on studying stormwater drainage within the Bay View Watershed and to develop drainage facilities to mitigate potential development within the Bayview Ridge Subarea.

The purpose of the *Bay View Watershed Stormwater Management Plan Phase 1* is to evaluate the stormwater impacts due to development within the UGA. This evaluation involves: 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 Watershed and to reduce farmland flooding. Skagit County funded the preparation of this Plan from its Drainage Utility fund.

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

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

Past development in the Study Area has been considered to be rural in nature. More concentrated residential development has occurred in the community of Bay View 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 Watershed. 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 improve capacity limiting facilities. Potential drainage facility improvements that were evaluated included the following:

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

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 in **Table 1-1**.

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.

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 Existing Channel Capacity	$ 119,000
	Construct Bypass Channel	$ 293,000
	Increase Pump Station Capacity by 54 cfs	$ 1,600,000
Joe Leary Slough Basin	Construct Peth Bypass Channel	$ 820,000
	Increase Joe Leary Slough Capacity	$ 684,000
	Increase South Spur Ditch Capacity	$ 101,000
Little Indian Slough Basin	Culvert Replacement and Increase Channel Capacity	$ 143,000
Big Indian Slough Basin	Culvert Replacement and Increase Channel Capacity	$ 670,000
	Outfall Detention Pond	$ 3,100,000
Total Capital Improvement Cost Estimate		$ 7,530,000

Chapter 2
Introduction

The Bay View Watershed is located in the westerly portion of Skagit County, west of the City of Burlington. This area has four drainage sloughs that convey stormwater runoff from the Bay View Watershed to Padilla Bay. Within the Bay View Watershed is the proposed Bayview Ridge Subarea. 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

In December 2006, Skagit County adopted an Urban Growth Area (UGA) for a portion of the Bay View Watershed. The UGA encompasses 3,633 acres, or 32% of the 11,277-acre watershed. The boundaries of the UGA were drawn following a multiple-year study of the slightly larger (4,011-acres) Bayview Ridge Subarea. The UGA includes the Skagit Regional Airport and many existing light industrial and residential uses. The Bayview Ridge Subarea Plan sets forth the development goals for the urbanized portion of the larger watershed.

The purpose of the *Bay View Watershed Stormwater Management Plan Phase 1* is to identify the stormwater drainage impacts from development within the Bayview Ridge Subarea and to make recommendations to mitigate for those impacts. This evaluation involves: 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 to mitigate impacts of flooding on the drainage sloughs and adjacent farmland; and 4) propose stormwater management strategies to manage drainage within the Bay View Watershed.

B. Stakeholders Purpose and Objectives

There are several entities that have a stake in stormwater drainage planning in the Bay View Watershed. 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 this stormwater drainage planning effort; providing project management and project funding. The County provides representation for the residents and property owners within Skagit County. County government is developing this drainage plan to provide a means to minimize present and future stormwater impacts to the citizens of the County and their properties located within these drainage basins. Some portions of the UGA are located in the County's Drainage Utility service area and all of the stormwater 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 Bayview Ridge. 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. Stormwater from approximately 537 acres of the Bayview UGA will drain into the Joe Leary Slough conveyance system. 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 stormwater from majority of the Bayview UGA area, approximately 2605 acres, will drain into Little Indian and Big Indian Sloughs.

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. 12

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, the lower reaches on No Name Slough, and several field ditches.

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 by the Skagit River and seawater damage along Padilla Bay. Now, an additional responsibility is maintaining the stormwater drainage facilities within the No Name Slough basin. Stormwater from approximately 869 acres of the Bayview UGA will drain into the No Name Slough conveyance system.

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. Port of Skagit County

The Port of Skagit County (herein called the Port) owns approximately 1,830 acres of property in the Bay View Watershed. The boundaries for the Port are shown in **Figure 2-2.** The Port's primary purpose is to create jobs. In the Bayview Ridge 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.

6. City of Burlington

Small portions of the Burlington city limits lie inside the drainage basins for Big Indian Slough and Joe Leary Slough. 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 collects drainage utility fees from its commercial property owners in this area and transfers the proceeds on to Drainage District No. 14.

The City of Burlington currently provides sanitary sewer service to a portion of the UGA. All commercial and industrial developments within the UGA will have sanitary sewer service. In addition, transportation impacts from development within the UGA will affect 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 these drainage basins impact existing stormwater facilities within the City of Burlington. The primary drainage conveyance system for the City of Burlington is Gages Slough, which discharges into the Skagit River.

7. Large Tract Land Owners

There are several large tract landowners within the UGA and it is anticipated that most of these large tract landowners will desire to develop their property. The designation of the UGA will provide opportunities to develop these large tracts and, therefore, will require increased attention to stormwater planning to accommodate anticipated growth.

8. 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 four stormwater drainage sloughs. The Reserve has recently 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 drainage basin, 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.

9. 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. 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.

In 2005, the Washington State Department of Fish and Wildlife purchase approximately 64 acres of lowlands in the No Name Slough Basin. The property is located between the Padilla Bay Trail and Bay View-Edison Road at the north end of the trail. The department does not currently have plans for this property, but development of fish habitat has been discussed.

10. 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 through the Big Indian drainage basin. 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 drainage basin.

11. 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 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 Watershed. 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.

[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).

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.

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 a UGA around the Skagit Regional Airport. The UGA boundary and proposed land use is shown on **Figure 2-4.**

The purpose of this UGA is to accommodate a small portion of the anticipated future county population. In addition to new residential areas, the *Bayview Ridge Subarea Plan* includes public services, commercial and industrial areas, and is designed to be compatible with the Skagit Regional Airport.

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

With regards to drainage, the *Bayview Ridge Subarea Plan* establishes the following goals, objectives, and policies.

[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 (December 2006).

Goal 2A **Provide for urban development within the Bayview Ridge UGA, which integrates existing and proposed uses, creating a cohesive community.**

Objective 2A-3 Develop a Drainage Plan for the Subarea that accommodates urban run-off and is consistent with the needs of adjacent Drainage Districts and designated agricultural land.

Policy 2A-3.1 *Establish limits on new impervious surfaces created within the Subarea.*

Policy 2A-3.2 *Require all new development to comply with the Bayview Watershed Stormwater Management Plan.*

Policy 2A-3.3 *Encourage the use of permeable surfaces and other new technologies in building construction and property development, consistent with County drainage regulations.*

Policy 2A-3.4 *Require cost-sharing arrangements which include Skagit County, Drainage District, and developer participation in the funding of required drainage improvements.*

Policy 2A-3.5 *Provide adequate enforcement, maintenance, and inspection services for storm drainage facilities.*

Policy 2A-3.6 *Provide business and residents of the Subarea with information regarding water quality and potential impacts to water from development.*

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:

- Levees along Joe Leary Slough near the outfall to raise the stored water surface elevation and increase outfall capacity during falling tides,
- 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 Bayview Ridge properties should mitigate for runoff volumes instead of peak discharge flow rates.

[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. 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 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.

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[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).

[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 Phase 1* 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. The Bayview Ridge Subarea is approximately 4,011 acres with only 3,232 acres being developable. The remaining acres are considered to be wetlands and buffers that have been set aside. **Figure 3-1** is an aerial photograph of the Bayview Ridge and surrounding farmland.

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 Bayview UGA	2,829 Acres	25.1 %	N/A
Water Bodies	37 Acres	0.3 %	N/A
Totals	**11,277 Acres**	**100 %**	

Source: Skagit County Mapping Services. Acreage figures are derived based on best information and technology available. Accuracy may vary depending on the source of the information, changes in political boundaries or hydrological features, or the methodology used to map and calculate a particular land use.

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 Bayview Ridge area include the rural village Bay View, Bay View State Park, Padilla Bay's Breazeale Interpretive Center, Skagit Regional Airport, numerous industrial and commercial developments, and residential cluster developments.

1. Historical Development

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

- The Bay View Watershed 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 Watershed in the late 1840s. The rural village of Bay View was named by William J. 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 Bayview 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.

[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.

- 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 Study 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 *Bayview Ridge Subarea Plan* supports the existing urban land use patterns. The overall intent of the *Bayview Ridge Subarea Plan* is to create a 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 Bayview 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 4 to 6 dwelling units per acre.

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, Bayview 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 Bayview Ridge area.

1. Topography

The Bayview Ridge area is situated east of Padilla Bay. This glacial terrace is elevated 220 feet above the surrounding floodplain. The physical features within the Bayview 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 Bayview 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 Watershed 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 Watershed is approximately 30 inches per year. This estimate was determined after review of rainfall data records from gauging stations located in Anacortes and Mount Vernon. Typically there is slightly more rainfall in Mount Vernon and less rainfall in Anacortes.

[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).

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).

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 Watershed 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 Watershed 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 Watershed are currently not served by a public water system and, therefore, homeowners rely on groundwater wells for their water supply. There are other properties throughout the Bay View Watershed that may also rely on groundwater wells for their source of water.

2. Flood Hazard Areas

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

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 Bayview Ridge. The *Bayview Ridge Subarea Plan* provides a detailed discussion regarding wetlands on the Bayview Ridge. Much of the discussion regarding wetlands presented in this Plan is derived directly from the *Bayview Ridge Subarea Plan*. The map showing the wetlands on the Bayview Ridge 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 Bayview Ridge 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 Bayview Ridge, wetlands have been identified based on the National Wetlands Inventory and interpretations of aerial photography. Approximately 349 acres of wetlands and buffers have been identified in the Bayview Ridge 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 Bayview 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 drainage 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.

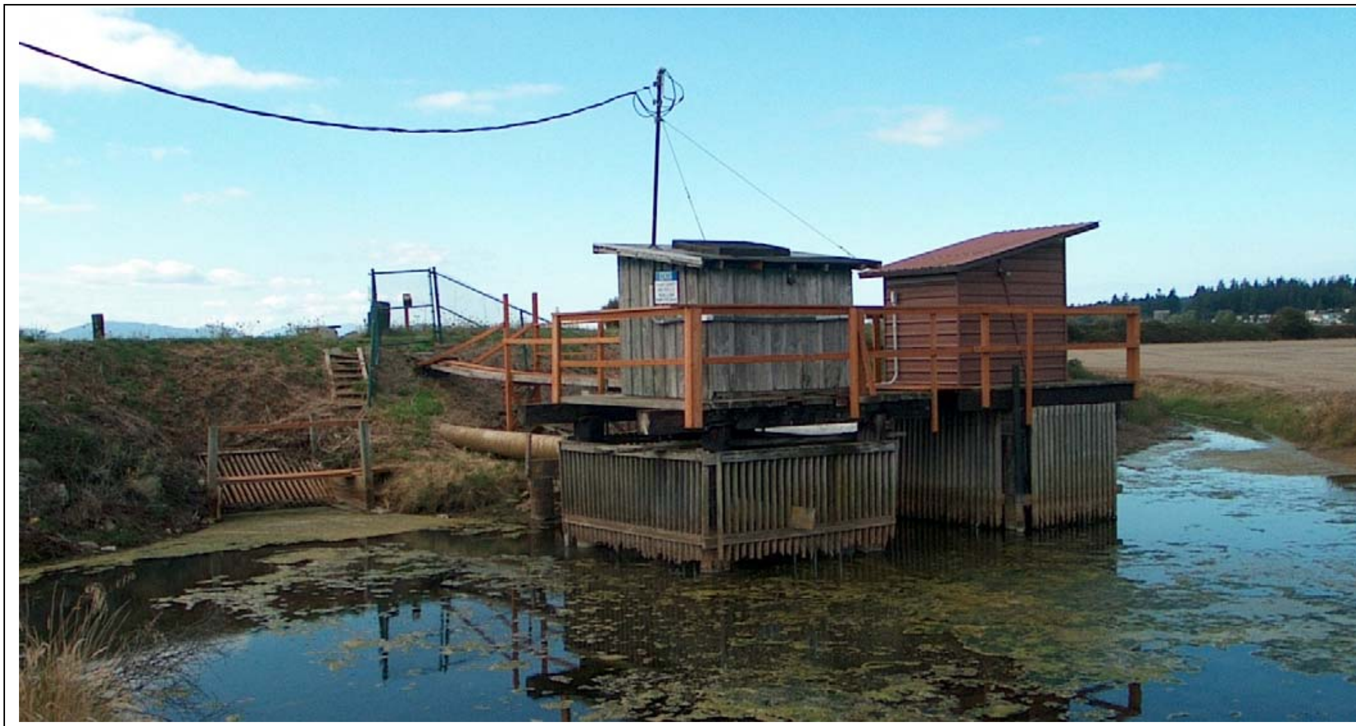
Gravity Outfall Structures: Four outfall structures total; one 5’x3’ box culvert with tide gate, one 48” HDPE with tide gate, and two 36” culverts with a common tide gate.

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

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 has been recently constructed on Port property northeast of the intersection of Ovenell Road and Farm-to-Market Road.

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

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 Bayview 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.

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

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

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

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

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

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

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

Culverts and Pipes: There are several storm drainage piping systems within this basin, primarily in the east portion within the newer residential developments. Some of the more recent improvements at the Port of Skagit 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, stormwater hydraulic models of the drainage sloughs were developed to identify existing and potential drainage problems. These hydraulic models were also used to analyze the benefits of potential drainage improvements to address identified problems. This chapter summarizes the methods and results of the hydraulic modeling. **Chapter 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. Hydraulic Model Development

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

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

1. Basin Development Scenarios

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

Predevelopment Scenario: This hydraulic model simulates stormwater drainage conditions prior to any development on the Bayview Ridge along with current farming operations within the floodplain. The Bayview Ridge area was modeled as a forest condition. This is consistent with analysis of predevelopment conditions outlined in the current Ecology Stormwater Management Manual[10].

Existing Development Scenario: This hydraulic model simulates stormwater drainage conditions from existing development on the Bayview Ridge and surrounding farmland. This

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

model, when compared to the Predevelopment Scenario, will provide the impact directly contributable to existing development on the Bayview Ridge.

UGA Development Scenario: This hydraulic model simulates stormwater drainage conditions resulting from development within the UGA, including the designated Urban Reserve. Other potential development outside of the UGA, either of the Bayview Ridge or in the surrounding farmland, is not accounted for in this model. This model, when compared to the Existing Development Scenario, will provide the impact directly contributable to development within the UGA.

2. Impervious Conditions

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.**

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

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

3. Tide Conditions

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

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

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

4. Rainfall Events

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

5. Model Basin Descriptions

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

a. No Name Slough Modeling Basin

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

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

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

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

Table 5-2: No Name Slough Effective Impervious Areas

Basin Name	Total Area	Effective Impervious Areas for Each Scenario		
		Predevelopment	Existing	UGA Development
A-4	489 acres	4.0%	4.0%	4.0%
A-5	306 acres	5.1%	5.1%	5.1%
A-6	100 acres	5.1%	5.1%	5.1%
A-7	325 acres	0.0%	10.2%	22.7%
A-8	127 acres	0.0%	4.0%	7.0%
A-11a	417 acres	0.0%	6.0%	15.9%
A-11b	672 acres	0.0%	6.0%	6.0%
A-11c	126 acres	0.0%	4.0%	4.0%
A-12	139 acres	0.0%	6.0%	6.0%
Totals	2,701 acres			

b. Joe Leary Slough Modeling Basin

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

The Joe Leary Slough basin drains about 10,300 acres. This basin was subdivided into 20 subbasins for the hydrologic modeling. The upper portion of the basin drains primarily agricultural land. The topography in the upper basin is very flat and drainage is facilitated by the use of agricultural drainage tiles. The lower portion of the basin, which gets most of its runoff from the Bayview Ridge area, is smaller than the upper portion of the basin. However, the topography along the north slope of Bayview 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. Key culverts along the slough are also included in the hydraulic model.

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

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

c. *Little Indian Slough Modeling Basin*

Figure 5-4 shows the modeled elements in the Little Indian Slough Basin and subbasin boundaries. The Little Indian Slough Basin is located on the southwest side of Bayview Ridge.

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.

Effective impervious areas for each subbasin were estimated for each development scenario based on past, existing or future land use. These were used in the hydraulic model to simulate stormwater runoff rates. **Table 5-4** lists the EIA for the modeled subbasin for each of the development scenarios.

Table 5-4: Little Indian Slough Effective Impervious Areas				
Basin Name	Total Area	Effective Impervious Areas for Each Scenario		
		Predevelopment	Existing	UGA Development
C-1a	54 acres	5.0%	5.0%	5.0%
C-1b	218 acres	5.0%	5.0%	5.0%
C-1c	156 acres	5.0%	5.0%	5.0%
C-2b	166 acres	0.0%	15.0%	35.0%
Totals	594 acres			

d. Big Indian Slough Modeling Basin

Figure 5-5 shows the modeled elements in the Big Indian Slough Basin and subbasin boundaries. The Big Indian Slough Basin is located on the south side of Bayview Ridge.

The Big Indian Slough modeling basin is in the southernmost part of the Study Area 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 Bayview Ridge UGA and has gradual 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, stormwater runoff can escape from Big Indian Slough near SR 536 and flow into Higgins Slough. The model indicated that as the slough water surface increased in Big Indian Slough, a disproportionate amount of stormwater runoff was diverted to Higgins Slough. Since there are known drainage problems downstream in Higgins Slough it was decided that allowing these overflows had an overall detrimental effect on the drainage system. It was determined that for modeling purposes, all stormwater runoff within Big Indian Slough would be discharged through the outfall at Bay View-Edison Road and overflow to Higgins Slough was not allowed.

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

Table 5-5: Big Indian Slough Effective Impervious Areas				
Basin Name	Total Area	Effective Impervious Areas for Each Scenario		
		Predevelopment	Existing	UGA Development
C-2a	135 acres	0.0%	8.0%	35.0%
C-3a	363 acres	4.0%	5.0%	19.2%
C-3b	220 acres	5.0%	5.0%	5.0%
C-4	422 acres	0.0%	4.0%	24.1%
C-5	133 acres	0.0%	4.0%	34.9%
C-6	116 acres	0.0%	8.0%	27.0%
C-7	1,646 acres	0.0%	29.1%	29.1%
C-8	2,017 acres	5.0%	8.0%	12.0%
Totals	5,052 acres			

B. Hydraulic Model 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. The results of the hydraulic modeling are presented below.

1. No Name Slough

Predicted peak flows in No Name Slough for the 10-, 25-, and 100-year storm events at various locations are listed in **Table 5-6**. **Table 5-7** compares the peak runoff rates for each development scenario in each subbasin.

The increase in stormwater flow rates between the Predevelopment Scenario and the Existing Development scenario are in the order of 5% at the outfall at Padilla Bay and 7% at the confluence with No Name Creek (node NN-83). The increase in stormwater flow rates between the Existing Development Scenario and the UGA Development Scenario are in the order of 45% at the outfall to Padilla Bay to 33% at the confluence with No Name Creek.

Peak flows calculated by the SWMM model were compared to the peak flows reported in Northwest Hydraulic Consultant's (NHC) *Hydrologic and Hydraulic Model of the No Name Slough Drainage* (November 2000). **Table 5-8** shows a comparison of the predicted peak flows at three locations for existing development conditions from both hydraulic models. In general, the predicted peak flows calculated by the SWMM model were higher than the predicted peak flows reported in the NHC study. The NHC study did not compute peak flows for other development conditions.

The SWMM hydraulic model indicated flooding at locations throughout the basin. The flooding is indicated at the 10-, 25-, and 100-year recurrence interval for all three development scenarios. **Table 5-9** shows predicted flooding locations with no drainage improvements implemented.

Table 5-6: Peak Flows for No Name Slough										
SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			UGA Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-10	Outlet of Slough (before pumps)	104 cfs	143 cfs	185 cfs	110 cfs	150 cfs	185 cfs	168 cfs	216 cfs	284 cfs
NN-83	Confluence of Tributaries	89 cfs	121 cfs	168 cfs	95 cfs	128 cfs	177 cfs	122 cfs	160 cfs	259 cfs
NN-110	Marihugh Road	29 cfs	40 cfs	57 cfs	32 cfs	43 cfs	57 cfs	32 cfs	43 cfs	57 cfs
See **Figure 5-2** for node locations.										

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

Table 5-8: Comparison of Existing Condition Peak Flow from SWMM and NHC Study						
	SWMM Existing Conditions Peak Flows for No Name Slough			NHC Existing Conditions Peak Flows for No Name Slough		
Approximate Location	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
Outlet of Slough	110 cfs	150 cfs	201 cfs	91 cfs	115 cfs	154 cfs
Confluence of Tributaries	95 cfs	129 cfs	177 cfs	81 cfs	101 cfs	132 cfs
Marihugh Road	32 cfs	43 cfs	59 cfs	12 cfs	16 cfs	23 cfs

Table 5-9: No Name Slough Flooding Locations with No Improvements										
SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			UGA Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Upstream of							●	●	●
NN-60	Upstream Culv. NN-C2			●			●	●	●	●
NN-65	Lower Slough			●			●	●	●	●
NN-67	Middle Slough		●	●		●	●	●	●	●
NN-80	Upstream Culv. NN-C3		●	●		●	●	●	●	●
NN-83	Confluence	●	●	●	●	●	●	●	●	●
NN-170	S, Stem Near Dahlstadt Farm	●	●	●	●	●	●	●	●	●
See **Figure 5-2** for node locations.							● denotes predicted flooding for the storm event			

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

2. Joe Leary Slough

Predicted peak flows in Joe Leary Slough for the 10-, 25-, and 100-year storm events at various locations are listed in **Table 5-10**. **Table 5-11** compares the peak runoff rates for each development scenario in each subbasin. Predicted flooding locations are illustrated in **Table 5-12.**

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

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

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

Table 5-10: Peak Flows for Joe Leary Slough

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

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

Table 5-11: Subbasin Peak Runoff for Joe Leary Slough

Subbasin	Predevelopment Scenario			Existing Development Scenario			UGA Development		
	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
B-1a	21 cfs	28 cfs	37 cfs	22 cfs	28 cfs	38 cfs	22 cfs	28 cfs	38 cfs
B-1b	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs
B-1c	18 cfs	24 cfs	32 cfs	18 cfs	24 cfs	34 cfs	18 cfs	24 cfs	34 cfs
B-1d	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs
B-1e	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs	4 cfs	5 cfs	7 cfs
B-2	18 cfs	24 cfs	34 cfs	19 cfs	25 cfs	35 cfs	19 cfs	25 cfs	35 cfs
B-3	33 cfs	44 cfs	60 cfs	34 cfs	45 cfs	62 cfs	34 cfs	45 cfs	62 cfs
B-4	17 cfs	23 cfs	31 cfs	18 cfs	24 cfs	33 cfs	18 cfs	24 cfs	33 cfs
B-5	10 cfs	13 cfs	18 cfs	10 cfs	13 cfs	18 cfs	10 cfs	13 cfs	18 cfs
B-6a	13 cfs	17 cfs	24 cfs	13 cfs	17 cfs	24 cfs	13 cfs	17 cfs	24 cfs
B-6b	31 cfs	41 cfs	56 cfs	31 cfs	41 cfs	56 cfs	31 cfs	41 cfs	56 cfs
B-6c	12 cfs	16 cfs	22 cfs	12 cfs	16 cfs	22 cfs	12 cfs	16 cfs	22 cfs
B-6d	26 cfs	33 cfs	44 cfs	27 cfs	35 cfs	46 cfs	27 cfs	34 cfs	46 cfs
B-7	22 cfs	30 cfs	42 cfs	22 cfs	30 cfs	42 cfs	22 cfs	30 cfs	42 cfs
B-8	37 cfs	48 cfs	66 cfs	38 cfs	50 cfs	68 cfs	69 cfs	90 cfs	122 cfs
B-9	33 cfs	43 cfs	57 cfs	33 cfs	43 cfs	57 cfs	33 cfs	43 cfs	57 cfs
B-10	19 cfs	25 cfs	35 cfs	19 cfs	25 cfs	35 cfs	19 cfs	25 cfs	35 cfs
B-11	21 cfs	27 cfs	36 cfs	21 cfs	27 cfs	36 cfs	21 cfs	27 cfs	36 cfs
B-12	64 cfs	82 cfs	110 cfs	64 cfs	82 cfs	110 cfs	64 cfs	82 cfs	110 cfs

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

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

Table 5-12: Joe Leary Slough Flooding Locations with No Improvements

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 at various locations are shown in **Table 5-13.** **Table 5-14** compares the peak runoff rates each development scenario in each subbasin. The upstream subbasin (C-2) that drains into the slough is part of the Urban Growth Area and some development there will increase the impervious area in the basin from approximately 15 to 35%. West of Farm-to-Market Road is ongoing agricultural activities that are expected to continue.

The increase in stormwater flow rates between the Predevelopment Scenario and the Existing Development scenario are in the order of 2% at the outfall at Padilla Bay and 78% at Farm-to-

Market Road (node LI-60). The increase in stormwater flow rates between the Existing Development Scenario and the UGA Development Scenario are in the order of 2% at the outfall to Padilla Bay to 65% at Farm-to-Market Road.

Some flooding problems predicted by the hydraulic model in the Little Indian Slough Basin are listed in **Table 5-15.** Some flooding is predicted in the ditch upstream of Farm-to-Market Road. According to the hydraulic model, some slough sections experience flooding at the 25-, and 100-year storm events due to development within the UGA (subbasin C-2). There is also some flooding of low-lying farm fields near the outfall.

Table 5-13: Peak Flows for Little Indian Slough

SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			UGA Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-10	Outlet of Slough	81 cfs	95 cfs	113 cfs	83 cfs	96 cfs	117 cfs	84 cfs	97 cfs	122 cfs
LI-32	Middle Slough	12 cfs	18 cfs	28 cfs	15 cfs	23 cfs	35 cfs	25 cfs	36 cfs	49 cfs
LI-60	Farm-to-Market Road	6 cfs	10 cfs	16 cfs	13 cfs	18 cfs	26 cfs	23 cfs	30 cfs	41 cfs

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

Table 5-14: Subbasin Peak Runoff for Little Indian Slough

Subbasin	Predevelopment Scenario			Existing Development Scenario			UGA Development		
	10-Year	25-Year	100-Year	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	4 cfs	7 cfs	11 cfs
C-1b	8 cfs	12 cfs	20 cfs	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	6 cfs	11 cfs	17 cfs
C-2	14 cfs	19 cfs	27 cfs	14 cfs	19 cfs	27 cfs	23 cfs	31 cfs	43 cfs

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

Table 5-15: Little Indian Slough Flooding Locations with No Improvements										
SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			UGA Development		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-20	Outfall		●	●		●	●		●	●
LI-22	Lower Slough		●	●		●	●		●	●
LI-24	Lower Slough									
LI-26	Lower Slough									
LI-32	Middle Slough									
LI-60	Culvert LI-C-1									●
LI-80	Upper Slough						●			●
LI-90	Culvert LI-C-2						●		●	●
See **Figure 5-4** for node locations.						● denotes predicted flooding for the storm event				

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 storm event. The high water surface elevation at Farm to Market Road also may be contributing to flooding upstream.

4. Big Indian Slough

Predicted peak flows in Big Indian Slough for the 10-, 25-, and 100-year storm events at various locations are listed in **Table 5-16.** **Table 5-17** compares the peak runoff rates for each development scenario in each subbasin. Several 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 hydraulic model, the slough is unable to contain the high flows within the channel. In addition, the backwater from this constriction propagates upstream, causing additional flooding. Predicted flooding locations are illustrated in **Table 5-18**.

The increase in stormwater flow rates between the Predevelopment Scenario and the Existing Development scenario are in the order of 2% at the outfall at Padilla Bay and 28% at Higgins-Airport Way (node BI-230). The increase in stormwater flow rates between the Existing Development Scenario and the UGA Development Scenario are in the order of 7% at the outfall to Padilla Bay to 91% at Higgins-Airport Way.

The model indicates the most severe flooding is at field access culverts upstream and downstream of Higgins Airport Way (nodes BI-210 and BI-250). Flooding at these locations is predicted at the 10-, 25-, and 100-year recurrence interval for all development scenarios. 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 (node BI-190).

The hydraulic model did not predict flooding downstream of the Higgins Slough bypass. However, it is documented that flooding of SR 20 west of Farm-to-Market Road occurred during the November 1990 storm event. The hydraulic model predicts this channel section has sufficient capacity stormwater runoff under all storm event and development scenarios. Drainage District No. 19 has widened this section of channel since 1990 and it appears that the increased capacity may prevent flooding of SR 20 in the future. There is relatively little additional tributary area to the slough downstream of Farm-to-Market Road, 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.

Table 5-16: Peak Flows for Big Indian Slough

SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			UGA Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Outlet to Padilla Bay	322 cfs	405 cfs	445 cfs	332 cfs	412 cfs	453 cfs	353 cfs	431 cfs	497 cfs
BI-160	Downstream of SR 20	155 cfs	246 cfs	289 cfs	163 cfs	257 cfs	301 cfs	184 cfs	282 cfs	364 cfs
BI-230	Higgins Airport Way	34 cfs	55 cfs	83 cfs	53 cfs	68 cfs	99 cfs	107 cfs	134 cfs	179 cfs
BI-270	Above SR 20	34 cfs	44 cfs	59 cfs	46 cfs	60 cfs	81 cfs	59 cfs	78 cfs	105 cfs

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

Table 5-17: Subbasin Peak Runoff for Big Indian Slough

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

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

Table 5-18: Big Indian Slough Flooding Locations with No Improvements										
SWMM Model Node	Approximate Location	Predevelopment Scenario			Existing Development Scenario			UGA Development Scenario		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-150	Culvert BI-C-1									
BI-170	Bridge BI-B-4		●	●		●	●		●	●
BI-180	Culvert BI-C-2			●			●			●
BI-190	Culvert BI-C-2			●			●		●	●
BI-200	Culvert BI-C-3		●	●		●	●		●	●
BI-210	Culvert BI-C-3		●	●		●	●		●	●
BI-220	Culvert BI-C-4									●
BI-230	Culvert BI-C-4						●			●
BI-240	Culvert BI-C-5		●	●		●	●		●	●
BI-250	Culvert BI-C-5		●	●		●	●		●	●
BI-260	Culvert BI-C-6									●
BI-270	Culvert BI-C-6									●
See **Figure 5-5** for node locations.						● denotes predicted flooding for the storm event				

a. Overflow to Higgins Slough

An overflow in the vicinity of node BI-120 provides some relief during peak storm events, as water is diverted from Big Indian Slough south into Higgins Slough. The amount of predicted overflow is presented in **Table 5-19** for each storm event and each development scenario. The model indicates that if the overflow were kept within Big Indian Slough, the downstream portion of the channel would still have adequate capacity under all storm events based on the modeling assumptions. However, observed flooding of SR 20 indicates that some hydrologic condition may exist that can overwhelm the outfall and lower reach of Big Indian Slough.

Table 5-19: Peak Overflow Rates from Big Indian Slough to Higgins Slough			
Condition	Peak Flow to Higgins Slough		
	10-Year	25-Year	100-Year
Predevelopment Scenario	8 cfs	27 cfs	37 cfs
Existing Development Scenario	10 cfs	29 cfs	39 cfs
UGA Development Scenario	14 cfs	34 cfs	53 cfs

b. *Outfall 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, which is 0.15 feet above the mean higher high water elevation used in the modeling. This is equivalent to a 8.55 tide

Table 5-20 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 some freeboard along the length of the lower slough to accommodate a water surface elevation that would generate sufficient head to keep the slough draining until the tide goes down.

Table 5-20: Big Indian Slough Outfall 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

Chapter 6

Stormwater Quality and Treatment

The purpose of the Bayview Ridge Subarea 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*[11] (Stormwater Manual) as Skagit County has not yet adopted the Department of Ecology's 2005 update to the Stormwater Manual.

The *Padilla Bay/Bay View Watershed Nonpoint Action Plan*[12] (Nonpoint Action Plan) is the most significant work to date regarding stormwater pollution in the Bay View Watershed. The Skagit County Department of Planning and Community Development, with the assistance of the Padilla Bay/Bay View Watershed Management Committee, prepared the Nonpoint Action Plan to provide a program of actions to reduce or prevent nonpoint source pollution and protect beneficial water uses. The Nonpoint Action Plan contains extensive background information on watershed characteristics, outlines goals and objectives for reducing nonpoint pollution, identifies and sometimes quantifies sources of nonpoint pollution, and outlines an implementation strategy. The Nonpoint Action Plan was reviewed and approved by the Washington State Department of Ecology on May 30, 1995. This plan is currently undergoing an implementation status review by the Skagit Conservation Education Alliance (SCEA), a non-profit foundation administered by the Skagit Conservation District to protect natural resources.

A. Bay View Watershed 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

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

[12] *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 Bayview Ridge 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 Watershed is based on recommendations presented in the Nonpoint Action Plan and recommended best management practices [BMPs] presented in the 2005 Stormwater Manual[13].

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 2005 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.

[13] *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 Watershed. 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*[14], 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

[14] *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.

4. Future Development

New residential, commercial or light industrial development or redevelopment activities are potential point and non-point sources of stormwater contamination. All development and redevelopment activity in this Study Area should be required as a minimum to include the best management practices (BMPs), the operational and structural source control BMPs, and the treatment BMPs included in the 2005 Stormwater Manual. The flow control BMPs for this Study Area should be modified as recommended in each of the four drainage basins outlined in Chapter 7 of this report.

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[15] and others[16] have published 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.

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

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

- 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.

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

A. Conceptual Alternatives

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

The following conceptual drainage facility improvements were considered for reducing flooding potential due to development within the Bayview Ridge Subarea:

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

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

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

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

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

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

- **Increase Outfall Capacity.** The tidal condition affects the capacity of the existing outfall pipes to Padilla Bay. If the capacity of the existing outfall pipes in not sufficient to discharge the required stormwater runoff during a given tidal cycle, then increasing the number and/or size of outfall culverts may reduce upstream flooding potential.

- **Construct Levees.** Consider constructing levees along drainage channels where the adjacent ground elevations are too low to prevent flooding. A drawback to levee construction is that natural drainage patterns from the farm fields are disrupted, potentially resulting in poor drainage during even small storm events.
- **Regional Detention.** A regional detention pond could reduce or delay the amount of runoff entering the slough and perhaps eliminate flooding throughout the area. Detention could also add the benefit of water quality treatment to remove sediment or other pollutants from reaching Padilla Bay. Further analysis would be needed to determine the optimal size and location of the pond.
- **Pump Station.** A stormwater pump station can decrease the upstream hydraulic grade elevation, resulting in an increase in flow in the upstream channel and a decrease in flooding. A pump station can also increase downstream hydraulic grades which will increase downstream channel flow rates but also increase flooding potential. The impacts of a proposed pump station on downstream flooding will need to be evaluated.

B. No Name Slough

Flooding in the No Name Slough drainage basin 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 are proposed to relieve flooding 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 outfall.

1. Culvert Replacement and Channel Widening

The hydraulic model indicated that there were channel reaches that were restricting the flow of stormwater. In addition, two existing culverts were also identified as being undersized for the peak stormwater runoff. Therefore, culvert replacement and channel widening were investigated to reduce the flooding potential in the low-lying farmland.

The hydraulic model was modified by making the slough channel wider from the node NN-70 to NN-180, a distance of approximately 4,000 LF. In addition, culverts NN-C-3 and NN-C-5 would need to be replaced. Dike & Drainage District No. 12 has indicated that they will be replacing culvert NN-C-3 with a bridge structure. Culvert NN-C-5 needs to be replaced with a 4-ft diameter circular corrugated metal pipe or equivalent culvert. It is currently 3-ft diameter CPE culvert pipe. The widened channel will be trapezoidal in shape with the following minimum characteristics:

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

This new channel cross-section more closely matches the channel cross-section downstream of node NN-70. The existing channel downstream of node NN-70 needs to be maintained with the same channel cross-section.

2. Bypass Channel

A bypass channel is proposed for the middle reach of the slough, starting at node NN-70 and reconnecting at NN-50 downstream of the Bay View-Edison Road culvert (NN-C-2). Most portions of a bypass channel have already been completed in this location by Dike & Drainage District No. 12. An additional 4-ft culvert under Bay View-Edison Road or a bridge structure is needed to prevent a channel restriction and optimize the bypass channel. The improved bypass channel was modeled as trapezoidal in shape with a length of 3,000 LF. The following characteristics were used to define the bypass:

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

3. Outfall Pump Station

Since flooding in the low-lying farmland is driven primarily by high tides coinciding with peak runoff. 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 outfall to allow drainage when the tidal head exceeds flood stage in the slough. No Name Slough already has two stormwater pumps at its outfall with a combined capacity of 36 cfs. The hydraulic model indicated that an additional pump capacity of 54 cfs is necessary to provide sufficient outfall conveyance at the 25-year storm event during a high tide cycle.

4. No Name Slough Drainage Improvement Recommendations

The hydraulic model indicates that there is wide spread flooding in the low-lying farmland during all three development scenarios. Drainage recommendations for No Name Slough include the following:

- Improve the conveyance capacity of the existing channel
- Add additional conveyance capacity through a bypass channel
- Increase outfall pumping capacity by 54 cfs.

Table 7-1 illustrates the reduction in flooding potential with the proposed improvements listed above for the No Name Slough Basin. The hydraulic model indicates that flooding will be eliminated at the 10-year storm event along the entire slough. Only in the upper reaches of the slough is flooding still predicted at the 25-year storm event.

There are two existing detention ponds within the UGA; one on the Paccar property (subbasin A-7) and one on the Port property near the intersection of Farm-to-Market Road and Ovenell Road (subbasin A-8). Skagit County should investigate the operation of these two detention ponds to determine if they are operating at their optimum performance.

Table 7-1: No Name Slough Flooding Locations With and Without Proposed Improvements

SWMM Model Node	Approximate Location	Existing Development Scenario with No Improvement			UGA Development Scenario with No Improvement			UGA Development Scenario with Proposed Improvements		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
NN-20	Slough Outlet				●	●	●			●
NN-30	Lower Slough				●	●	●			●
NN-40	Lower Slough				●	●	●			●
NN-60	Middle Slough			●	●	●	●			●
NN-65	Middle Slough			●	●	●	●			●
NN-70	Culvert NN-C-3		●	●	●	●	●			●
NN-83	Confluence	●	●	●	●	●	●		●	●
NN-140	Upper Slough	●	●	●	●	●	●		●	●
NN-160	Upper Slough	●	●	●	●	●	●		●	●
NN-170	Culvert NN-C-5	●	●	●	●	●	●		●	●

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

5. No Name Slough Development Regulation Recommendations

All new development and redevelopment activities in the No Name Slough Basin should be required as a minimum to include the best management practices (BMPs), the operational and structural source control BMPs, and the treatment BMPs included in the 2005 Stormwater Manual for mitigation of the water quality impacts.

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

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

C. Joe Leary Slough

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

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

1. Bypass Channel

A bypass channel in the lower portion of the slough along the toe of the hill was examined as a way of reducing flooding and increased channel efficiency. Routing flow more directly to the outfall will reduce runoff times and allow more stormwater to be discharged during low tide cycles. It was assumed for this analysis that subbasins B-1c and B-2, approximately 434 acres, would drain into the bypass channel. The travel distance of this stormwater runoff would be reduced by approximately 7,000 LF with the bypass channel. The bypass channel was assumed to be trapezoidal in shape with a length of 4,200 LF. 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

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 outfall 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 outfall. 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.

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 outfall 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 outfall. The culvert at Josh Wilson Road on the South Spur Ditch may be installed too high and may be a restriction during low flow conditions, but does not appear to be a restriction during storm event.

3. Joe Leary Slough Channel Widening

The hydraulic model indicated that a channel restriction was occurring within Joe Leary Slough from Allen West Road (node JL-80) to the confluence of Maiben Ditch and South Spur Ditch (node JL-126). There is a 15-foot wide arch culvert at Allen West Road and a 15-foot wide arch culvert at Benson Heights Place. These two culverts have more capacity than the channel in this vicinity. Widening the channel from Allen West Road to the confluence will make the channel capacity match the culvert capacity. The current width of the channel in this section is between 8.7 feet and 14.3 feet. The length of the proposed channel widening is approximately 9,000 LF. The widened channel will be trapezoidal in shape with the following minimum characteristics:

- 13-foot bottom width
- 2:1 side slopes
- Manning's 'n' roughness coefficient of 0.045

As a result of the channel widening, four existing wooden bridges will need to be replaced with longer bridges or 15-foot wide arch culverts. This channel section could be widened an additional 2 feet to a 15-foot bottom width, which will closely match the channel capacity with the culvert capacity.

4. South Spur Ditch Channel Widening

In conjunction with the main channel widening, some narrow channel sections along the South Spur Ditch will need to be widened to increase capacity, specifically the section north of Josh Wilson Road from node JL-161 to JL-190. The hydraulic model indicates these restrictions reduce slough conveyance and increases the flooding potential of the adjacent farmland in the vicinity. There is a 14-foot wide arch culvert at Josh Wilson Road, but some sections of the channel to the north are as narrow as 4 feet. The widened channel will be trapezoidal in shape with the following minimum characteristics:

- 13-foot bottom width
- 2:1 side slopes
- Manning's 'n' roughness coefficient of 0.045

5. Pump Station at the Outfall

A pump station at the outfall was examined as a way to reduce water surface elevations in the slough during high tide. A range of peak pump capacities were examined to estimate the effectiveness of a pump station at the outfall. The results indicate that in order for a pump station to be effective, its capacity must approach the peak flow in the slough. Because of the high volume of water produced by the design storm, even a small decrease in the capacity of the outfall 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 cubic feet per second (cfs).

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

6. Pump Station at the South Spur Ditch

A pump station on the South Spur Ditch near node JL-161 was examined as a way to reduce water surface elevations in that portion of the slough. A pump station with a capacity of 60 cfs could reduce water surface elevations by 3.5 feet at the pump station and by 2 feet at Michael Place. This is more that can be accomplished with any combination of channel widening and/or regional detention. However, the pump station would decrease the water surface elevation considerably more than the expected impact from the proposed development within the UGA.

Drainage District No. 14 believes that existing flooding impacts on farmland along the South Spur Ditch are unacceptable. Once the slough restrictions are removed from the system and additional hydraulic information has been collected, then the need for this pump station can be reevaluated.

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

7. Joe Leary Slough Drainage Improvement Recommendations

There are three recommendations for stormwater drainage improvements within the Joe Leary Slough Basin than address the impact of development within the proposed UGA. These three recommendations address drainage problems within three different sections of the drainage systems as listed below:

- Peth bypass channel improves stormwater drainage within the lower reaches of Joe Leary Slough. This is an existing problem and development within the UGA does not measurably increase water surface elevations in this reach of the channel. However, improving the drainage efficiency of this section of the slough will ensure that the additional development within the UGA will not further impact existing flooding potential.
- Widening Joe Leary Slough south of Allen West Road up to the confluence with Maiben Ditch and South Spur Ditch will improve drainage efficiency in the middle section of Joe Leary slough up to the confluence with South Spur Ditch. This increase in drainage efficiency will reduce the backwater effects the Maiben Ditch has on South Spur Ditch. To accommodate this channel widening, four existing wooded bridges also need to be replaced.
- Widening the South Spur Ditch from the confluence with Joe Leary Slough to Josh Wilson Road. Development within the UGA does have measurable impacts on water surface elevations within the South Spur Ditch, which has several restriction points in this channel section. Removing these restrictions will reduce the flooding potential along South Spur Ditch from stormwater runoff from the UGA.

Implementation of the recommended projects listed above will decrease water elevations in the South Spur Ditch. However, local property owners have expressed an interest in capturing stormwater at the base of the ridge and conveying it along the toe of the hillside, rather than utilizing the existing field drainage ditches as is currently being done. Developers are encouraged to consider such options in evaluation of their drainage conveyance systems.

Table 7-2 illustrates the reduction in flooding potential with the proposed improvements listed above for the Joe Leary Slough Basin. The hydraulic model indicates that flooding above Farm-to-Market Road is reduced with the proposed improvements, including the South Spur Ditch. Most of the 10-year storm event flooding is eliminated and there is improvement in the 25-year storm event flooding potential. The hydraulic model also indicates that the flooding potential near the outfall will not be improved as a result of these proposed improvements. Dikes, flood easement, storage or an outfall pump station will be needed to reduce flooding near the outfall.

8. Joe Leary Slough Development Regulation Recommendations

All new development and redevelopment activities in the Joe Leary Slough Basin should be required as a minimum to include the best management practices (BMPs), the operational and structural source control BMPs, and the treatment BMPs included in the 2005 Stormwater Manual for mitigation of the water quality impacts. Channel erosion is not an issue for the main stem of Joe Leary Slough and the South Spur Ditch portion of the drainage system.

If the recommended Peth Bypass, Joe Leary Slough channel widening – Allen West to Maiben confluence, and the South Spur channel widening are built to accommodate the future development in the UGA, the flow control requirement for this basin could be less stringent than what is required in the 2005 Stormwater Manual. Project sites must be drained by a conveyance system comprised entirely of manmade conveyance elements (e.g., pipes, ditches, outfall protection, etc.) and extend to the main stem of Joe Leary Slough or South Spur Ditch (e.g., Node JL-200). The conveyance system between the project site and the main stem of Joe Leary system shall have sufficient capacity to convey discharges from future build-out conditions in the UGA. Project proponents will need to analyze, design and build the conveyance system. The conveyance system could include enhancement of existing manmade facilities, or construction of new conveyance systems, such as the ditch at the toe of the hillside discussed in the previous section. In almost all cases the downstream analysis for all projects will need to extend at a minimum to the main stem of Joe Leary Slough or Node JL-183 downstream of Josh Wilson Road on the South Spur Ditch. The culvert under Josh Wilson Road at Node JL-190 causes a rise in the water surface elevation upstream to the east. Runoff from the UGA that can be collected up-gradient of this culvert and can be crossed under Josh Wilson Road to discharge down stream of Node JL-190 should be a priority in the UGA conveyance designs and planning.

Table 7-2: Joe Leary Slough Flooding Locations With and Without Proposed Improvements

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

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

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 UGA. The SWMM model schematic for the Little Indian Slough is presented in **Figure 5-4.** The following conceptual alternatives are proposed to relieve flooding in the area.

1. Upstream Culvert and Channel Upgrades

Based on the hydraulic model results, the entire drainage system upstream of Farm-to-Market Road should be improved. 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, flooding upstream of Farm-to-Market Road can be reduced. According to the hydraulic model, these improvements would result in a measurable increase in downstream flooding.

2. Upstream Regional Detention

The hydraulic model predicts that the lower reach of Little Indian Slough has the potential for flooding surrounding farmland at storm events greater than the 10-year storm event. This results from the backwater affect that occurs at high tide. The area that would be impacted is estimated to be less than 100 acres.

One alternative that may address this lowland flooding potential is a regional detention pond near Farm-to-Market Road. The portion of the Little Indian Slough Basin east of Farm-to-Market Road is within the UGA and is the only area that is expected to experience future development resulting in more stormwater runoff. In theory, a detention pond would retain a sufficient volume of stormwater runoff that the downstream flooding potential would be reduced. The detention pond could be located on either side of Farm-to-Market Road. The west side of Farm-to-Market Road would appear to be more favorable considering the higher land costs within the UGA boundary.

Based on hydraulic modeling, a detention pond would not be effective in reducing flooding in the lower portion of the Little Indian Slough. High tide elevations coupled with low ground elevations appear to control flooding in the lower portions of the slough.

3. Outfall Pump Station

A stormwater pump station at the outfall of Little Indian Slough appears to be the only alternative for reducing the flooding potential in the lower reaches of the slough. In the past, a pump station at this location was considered to not have a favorable benefit-cost ratio. The value of the farmland that could be protected by a stormwater pump station does not justify the cost of construction, operation, and maintenance of a pump station for such an infrequent storm event.

4. Little Indian Slough Drainage Improvement Recommendations

The recommendations for stormwater drainage improvements within Little Indian Slough are listed below:

- Upsize the culvert at Farm-to-Market Road (LI-C-1)
- Upsize the culvert at the Hughes Farm access road (LI-C-2)
- Increase channel capacity east of Farm-to-Market Road.

These capital improvements are all within the proposed UGA as well as entirely within the taxation boundary of Drainage District No. 19. However, it has been determined that Drainage District No. 19 does not have an easement or right-of-way for the drainage ditch east of Farm-to-Market Road; therefore, it is currently not maintaining these drainage facilities. To facilitate the proposed improvements, the Skagit County Drainage Utility could help facilitate the acquisition of the necessary easements east of Farm-to-Market Road.

Table 7-3 illustrates the reduction in flooding potential with the proposed improvements listed above for the Little Indian Slough Basin.

5. Little Indian Slough Development Regulation Recommendations

All new development and redevelopment activities in the Little Indian Slough basin should be required as a minimum to include the best management practices (BMPs), the operational and structural source control BMPs, and the treatment BMPs included in the 2005 Stormwater Manual for mitigation of the water quality impacts. Channel erosion is not an issue for Little Indian Slough.

If the recommended culverts are upsized and the downstream land owner agrees that the current level of flooding of the outfall area fields is acceptable, the flow control requirement for this basin could less stringent than what is required in the 2005 Stormwater Manual. Project sites must be drained by a conveyance system that is comprised entirely of manmade conveyance elements (e.g., pipes, ditches, outfall protection, etc.) and extend to the main stem of Little Indian Slough. The conveyance system between the project site and the main stem of Little Indian Slough shall have sufficient capacity to convey discharges from future build-out conditions in the UGA. Project proponents will need to analyze, design and build the conveyance system.

The Drainage District should also seek an easement or agreement for the low land field flooding.

Table 7-3: Little Indian Slough Flooding Locations With and Without Proposed Improvements										
SWMM Model Node	Approximate Location	Existing Development Scenario with No Improvement			UGA Development Scenario with No Improvement			UGA Development Scenario with Proposed Improvements		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
LI-20	Slough Outlet		●	●		●	●		●	●
LI-22	Lower Slough		●	●		●	●		●	●
LI-24	Lower Slough									
LI-26	Lower Slough									
LI-32	Middle Slough									
LI-60	Culvert LI-C1						●			●
LI-80	Upper Slough			●			●			●
LI-90	Culvert LI-C2			●		●	●		●	●
See **Figure 5-4** for node locations.					● denotes predicted flooding for the storm event					

E. Big Indian Slough

Flooding in Big Indian Slough appears to be concentrated near the confluence of the runoff from the UGA (including Skagit Regional Airport) and the main stem of Big Indian Slough. This confluence of two large flows appears to overwhelm the existing conveyance system, specifically culverts, and causes flooding in the general vicinity. This is also the location where future stormwater runoff from the UGA will enter the slough drainage system, specifically nodes BI-200 and BI-235. The hydraulic model schematic for the Big Indian Slough is presented in **Figure 5-5.** The following conceptual alternatives were investigated to relieve flooding in the area.

1. Channel Widening and Culvert Replacement

According to the hydraulic model, several small field access culverts, specifically BI-C2, BI-C3, and BI-C5, do not have sufficient capacity to convey peak stormwater runoff. In addition, the slough channel appears to have some restrictions upstream of Farm-to-Market Road (node BI-90). As a result of these restrictions, flooding of farm fields is occurring.

Culvert capacity at culvert BI-C1 under Bradshaw Road does not appear to be a restriction. At this location, flooding appears to occur as a result of the backwater downstream and the low overtopping elevation of the slough bank (approximately 7.5 feet). In addition, the existing channel has a grade restriction upstream at the natural gas pipeline crossing between BI-170 and BI-180.

Flooding and backwater effects at culverts BI-C2, BI-C3 and BI-C5 appear to be a result of the undersized culverts and their 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. It is recommended that these culverts be removed, and if necessary for maintaining field access, be replaced with bridges.

The channel downstream of Farm-to-Market Road appears to have sufficient capacity to convey peak stormwater runoff. Channel widths in this location range from 15 to 18 feet. However, the slough upstream of Farm-to-Market Road appears to have some channel restrictions. Channel widths between Farm-to-Market Road and Bradshaw Road range from 8 to 12 feet. Upstream of Bradshaw Road the channel widths are 10 feet or less.

The hydraulic model was modified with wider channels from Farm-to-Market Road beyond Higgins-Airport Way to node BI-230. The increase in channel width ranged from 4 to 6 feet. Along with removal of culverts BI-C2, BI-C3 and BI-C5, the hydraulic model indicated that the peak runoff could be accommodated in the existing widened channel.

2. State Route 20 Bypass

An alternative to replacing culverts and widening the existing channel is a new bypass along State Route 20 (SR 20) northeast of Bradshaw Road. 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 into the bypass. The bypass channel was assumed to be trapezoidal in shape with a length of approximately 3,100 feet. 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 0.2 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). 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.

In general, the bypass lowers water surface elevations by almost 1 foot in the upper portion of the model. Flooding at nodes BI-220, BI-225, BI-230, BI-260, and BI-270 is eliminated as a result of the bypass. 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 10.5 feet.

The problem with this option is that new easement or right-of-way will need to be acquired. This section of SR 20 is already being widened to accommodate additional lanes by WSDOT. The new drainage easement would need to be acquired south of the new WSDOT right-of-way.

3. Higgins Slough Bypass 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 hydraulic model shows that removal of the Higgins Slough overflow would not increase the frequency of flooding in the drainage system. Water surface elevations downstream of Higgins Slough would increase if the overflow were removed; but the impact on the slough would not be significant. Flood elevations would increase, but the frequency of flooding would remain the same.

This result is due to the fact that Big Indian Slough's outfall 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 Higgins Slough.

4. Downstream Detention

A detention pond located near the outfall 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 upstream flooding culverts. The modeling predicts a peak water surface elevation of 3.2 feet at the slough outfall 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.

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 to a depth of 4 feet, providing approximately 92 acre-feet.

The results of the modeling indicate that excavated detention will reduce the water surface along the lower portion of the slough to the level before diverting the bypass to Higgins Slough, but will have no impact on the upstream flooding. 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 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.

5. Big Indian Slough Drainage Improvement Recommendations

The recommendations for stormwater drainage improvements within Big Indian Slough are listed below:

- Widening the existing channel
- Removing the restrictive field culverts and replace with bridges, if necessary
- Excavating a detention pond at the outfall.

Widening the existing channel has some advantage over acquiring and constructing a new channel.

Table 7-4 illustrates the reduction in flooding potential with the proposed improvements listed above for the Big Indian Slough Basin.

6. Big Indian Slough Development Regulation Recommendations

All new development and redevelopment activities in the Big Indian Slough Basin should be required as a minimum to include the best management practices (BMPs), the operational and structural source control BMPs, and the treatment BMPs included in the 2005 Stormwater Manual for mitigation of the water quality impacts. Channel erosion is not an issue for the main stem of Big Indian Slough.

If the recommended channel widening, culvert removals and excavated detention are built to accommodate the future development, the flow control requirement elsewhere within this basin could be less stringent than what is required in the 2005 Stormwater Manual. Project sites must be drained by a conveyance system that is comprised entirely of manmade conveyance elements (e.g., pipes, ditches, outfall protection, etc.) and extend to the main stem of Big Indian Slough (e.g., node BI-235). The conveyance system between the project site and the main stem of Big Indian Slough shall have sufficient capacity to convey discharges from future build-out conditions in the UGA. Project proponents will need to analyze, design and build the downstream conveyance system. In almost all cases the downstream analysis for all projects will need to extend as a minimum to the main stem of Big Indian Slough.

Table 7-4: Big Indian Slough Flooding Locations With and Without Proposed Improvements

SWMM Model Node	Approximate Location	Existing Development Scenario with No Improvement			UGA Development Scenario with No Improvement			UGA Development Scenario with Proposed Improvements		
		10-Year	25-Year	100-Year	10-Year	25-Year	100-Year	10-Year	25-Year	100-Year
BI-20	Slough Outlet									
BI-90	Farm-to-Market Rd									
BI-120	Higgins Slough									
BI-150	Culvert BI-C1									
BI-170	Bridge BI-B-4		●	●		●	●			
BI-180	Culvert BI-C-2			●			●			
BI-190	Culvert BI-C-2			●		●	●			
BI-200	Culvert BI-C-3		●	●		●	●			
BI-210	Culvert BI-C-3		●	●		●	●			●
BI-220	Culvert BI-C-4						●			
BI-230	Culvert BI-C-4			●			●			
BI-240	Culvert BI-C-5		●	●		●	●			
BI-250	Culvert BI-C-5		●	●		●	●			●
BI-260	Culvert BI-C-6						●			
BI-270	Culvert BI-C6						●			

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

Chapter 8

Capital Improvement Plan

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

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

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

A. Cost Estimating Methodology

Cost estimates presented within this Capital Improvement Plan are considered "Concept Budgetary Estimates". Construction cost estimates are made without design plans. These project cost estimates should be considered 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 2007 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]*[17] prepare by the US Army Corp of Engineers was used to adjust historical construction cost to January 2007 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.

[17] *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

Stormwater pump station costs were estimated using parametric estimating, which is a technique using a statistical relationship between historical data and other variables such as pump station capacity. Data from nine existing stormwater pump station project costs were used to estimate the pump station project costs presented in this Plan. These nine pump station capacities ranged from 18 cfs to 1,250 cfs. The historical costs were adjusted to 2007 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.

$$\text{Pump Station Project Cost} = 0.0647 \times (\text{Capacity})^{0.8054}$$

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. Culverts

Proposed culverts construction costs are estimated based on a schematic layout. Construction costs include a gross estimate of excavation, hauling and disposal of earth material, pipe material, and imported fill material, along with an appropriate estimate of restoration. Indirect costs, which include planning, surveying, geotechnical investigations, design, permitting, project management, construction management, financing costs and construction cost contingencies, were estimated to be 50 percent of the construction cost estimate. No additional land costs are expected for culvert installations. The historical costs were adjusted to 2007 cost using the CWCCIS Cost Index.

4. Bridges

Bridge construction cost estimates are based on an assumed cost per square foot. Published data indicates that construction costs for wood-type bridges range from $75 to $100 per square foot. A one-lane bridge is assumed to be 15 feet wide while a two-lane bridge is assumes to be 30 feet wide. The length of the bridge is the width at the top of the channel section plus an additional 5 feet on each end. Indirect costs, which include planning, surveying, geotechnical investigations, design, permitting, project management, construction management, financing costs, existing bridge removal and disposal, 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 not estimated or included in these project cost estimates.

5. Channel and Detention Pond Excavation

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

B. Capital Improvements

Table 8-1 provides a proposed Capital Improvement Plan for planning, design and construction of the stormwater drainage facilities in the Bay View Watershed to accommodate growth within the UGA. 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*.

1. No Name Slough Recommendations

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

a. Channel Widening

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

b. Culvert Replacement

Replacement of two undersized culverts is recommended; culverts NN-C-3 and NN-C-5. Dike & Drainage District No. 12 has indicated that they plan to replace culvert NN-C-3 with a bridge. Culvert NN-C-5 should be replaced with a 4-ft culvert pipe. Local topography is needed as part of the final design to verify that the specified culvert shape and material are appropriate for that location.

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

Drainage Basin and Proposed Stormwater Capital Improvement	Project Cost Estimate (FY 2006)	Projected Capital Improvement Costs with Escalation[1]				
		FY 2007	FY 2008	FY 2009	FY 2010	FY 2011
No Name Slough Basin						
Improve Conveyance of the Existing Channel	$ 51,000	$ 53,000				
Replace Culvert NN-C3 with Bridge	$ 54,000	$ 56,000				
Replace Culvert NN-C5 with 54" Culvert	$ 14,000	$ 15,000				
Construct Bypass Channel	$ 293,000	$ 305,000				
54-cfs Outfall Pump Station	$ 1,600,000		$ 346,600	$ 721,200	$ 750,000	
Joe Leary Slough Basin						
Peth Property Bypass Channel	$ 820,000		$ 177,600	$ 739,200		
Joe Leary Slough Channel Widening	$ 204,000	$ 106,000	$ 110,500			
4 Bridge Replacements	$ 480,000	$ 100,000	$ 416,000			
South Spur Ditch Channel Widening	$ 101,000	$ 52,500	$ 54,500			
Little Indian Slough Basin						
Increase Channel Capacity Upstream of Farm-to-Market Rd	$ 30,000				$ 35,000	
48" Culvert Replacement on Farm-to-Market Road [LI-C-1]	$ 78,000				$ 91,000	
48" Culvert Replacement on Farm-to-Market Road [LI-C-2]	$ 35,000				$ 41,000	
Big Indian Slough Basin						
Outfall Detention Pond	$ 3,100,000				$ 726,400	$ 3,021,600
Big Indian Slough Channel Widening	$ 310,000	$ 161,500	$ 168,000			
Replace 3 Culverts with Bridges	$ 360,000	$ 75,000	$ 312,000			
Totals	$ 7,530,000	$ 924,000	$ 1,585,200	$ 1,460,400	$ 1,643,400	$ 3,021,600

Note 1: Escalation is assumed to be 4% per year.

c. Bypass Channel

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

d. Increased Pumping Capacity

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

Regional detention at subbasin A-11a at Marihugh Road 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, flooding in Joe Leary Slough is largely driven by the tidal cycle. Since ground elevations of adjacent agricultural fields are often in the range of 5 to 10 feet, stormwater drainage alternatives that can reduce flooding are limited. The following drainage improvements 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 slough bypass channel would be located primarily on low lying properties owned by J. Peth, W. Paulus and others. The bypass would circumvent the culvert and channel restrictions along D'Arcy Road where the channel is confined by the road. This bypass channel would lower water surface elevations in the lower section of the slough up to Farm-to-Market Road.

b. Joe Leary Slough Widening

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

c. Bridge Replacement

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

d. South Spur Ditch Widening

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

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

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 upstream detention. Therefore the result may be conservative.

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

The following drainage improvement is recommended:

a. Culvert Replacement and Channel Widening

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

Detention is not recommended at this time. Detention 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

Big Indian Slough has the greatest impact from development within the Bayview UGA. Considerable development, including residential, commercial, and light industrial development, has already occurred, but more development is planned within the Bayview UGA. The following drainage improvements are recommended:

a. Outfall Detention Pond

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

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

b. Big Indian Channel Widening

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

c. Replace Culverts with Bridges

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

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

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

C. Stormwater Management Strategies

There are several stormwater management strategies that are recommended to be instituted in the Bay View Watershed. **Table 8-2** provides a list and cost estimate of proposed stormwater management strategies that 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.

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. Historical, Drainage Districts are not aware of proposed developments permit approval. By this time it is too later for Drainage Districts to recommend stormwater drainage mitigation. 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, 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 Watershed 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.

Table 8-2: Recommended Stormwater Management Strategies for the Bay View Watershed

Stormwater Management Program Items	5-Year Program Estimate (FY 2007)	Projected Stormwater Management Program Costs with Escalation				
		FY 2007	FY 2008	FY 2009	FY 2010	FY 2011
Entire Bay View Watershed						
Negotiate Interlocal Agreements with Drainage Districts	$ 60,000	$ 20,000	$ 21,000	$ 22,000		
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	$ 90,000	$ 50,000	$ 10,000	$ 11,000	$ 11,000	$ 12,000
Revise, Expand and Update Hydraulic Model	$ 80,000		$ 21,000	$ 22,000	$ 22,000	$ 23,000
No Name Slough Basin						
Analyze and Optimize the Paccar and Port Detention Ponds	$ 30,000		$ 16,000	$ 17,000		
Slough and Channel Cleaning and Maintenance	$ 45,000	$ 9,000	$ 9,000	$ 10,000	$ 10,000	$ 11,000
Pump Station Operation and Maintenance	$ 25,000	$ 5,000	$ 5,000	$ 5,000	$ 6,000	$ 6,000
Joe Leary Slough Basin						
Slough and Channel Cleaning and Maintenance	$ 200,000	$ 40,000	$ 42,000	$ 43,000	$ 45,000	$ 47,000
Little Indian Slough Basin						
Slough and Channel Cleaning and Maintenance	$ 25,000	$ 5,000	$ 5,000	$ 5,000	$ 6,000	$ 6,000
Big Indian Slough Basin						
Analyze and Optimize the Boslog and Port Detention Ponds	$ 20,000		$ 10,000	$ 11,000	$ -	$ -
Slough and Channel Cleaning and Maintenance	$ 50,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000	$ 12,000
Pump Station Operation and Maintenance	$ 50,000	$ 10,000	$ 10,000	$ 11,000	$ 11,000	$ 12,000
Totals	$ 725,000	$ 199,000	$ 159,000	$ 168,000	$ 122,000	$ 129,000

Note 1: Escalation is assumed to be 4% per year.

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. Existing Detention Pond Analysis and Optimization

There are several large detention ponds within the Bay View Watershed. It may be worthwhile to analyze the existing operation of these detention ponds. There may be modifications, such as resized orifices or adjusted overflow weirs, that can be made to improve the operation of the detention facilities. Properties such as Paccar, Port of Skagit County, and Boslog, have existing detention ponds that could potentially be evaluated and optimized.

7. Slough and Channel Cleaning and Maintenance

Slough and channel cleaning and maintenance are 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.

8. 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.

9. 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.

Appendix A:

Stormwater Facility Inventory

Basin A: No Name Slough

Basin B: Joe Leary Slough

Basin C: Indian Slough

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN A: No Name Slough Basin

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
A-OUT	24"	74 LF	Coated Metal FM with Tide Gate		0.4			33, 34	1164	
	24"	74 LF	Coated Metal FM with Tide Gate			0.6		33, 34	1165	
	18"	73 LF	Coated Metal FM with Tide Gate		0.3			33, 34	1163	
	18"	73 LF	Coated Metal FM with Tide Gate			0.0		33, 34	1166	
A-C1	10 FT.	30 LF	CMP	4.4	-5.6	-5.6		33, 34	1160, 1161	
A-C2	13' W X 10' H	50 LF	Arched CMP		-9.5	-7.5	-0.040	33, 34	1159	
A-C64	36"									
A-C88	18"	40 LF	Concrete							
A-C87	24"	31 LF	Concrete							
A-C85	24"	54 LF	Concrete							
A-C86	24"	30 LF	CMP							
A-C72										
A-C84										
A-C71										
A-C83	10"		Ductile Iron							
A-C82	12"	40 LF	CPE							
A-C81										
A-C3	48"	30 LF	CMP							
A-X1	Cross Section									
			Top =	10' to 12'						
			Width at Water Surface =	5'						
			Freeboard =	2.5'-3'						
			Depth =	2.5'						
A-C4	12"	18 LF	Ductile Iron							
A-X2	Cross Section									

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN A: No Name Slough Basin										
Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
			Top =	14' to 15'						
			Width at Water Surface =	7'						
			Freeboard =	2.5'-3'						
			Depth =	2.5'						
A-X3	Cross Section									
			Top =	10' to 12'						
			Width at Water Surface =	5'						
			Freeboard =	2'						
			Depth =	2'						
A-C5	30"	30 LF	CPE							
A-C6	3.0' W X 2.5' H	6 LF	Box Culvert							
A-C7	30"	31 LF	CPE with 5' Wide Wingwalls							
A-C8	24"	80 LF	CPE							
	18"	75 LF	Concrete							
A-C9	24"	37 LF	Concrete							
	18"	35 LF	Concrete							
A-C10	24"	16 LF	CMP							
A-C11	24"	22 LF	CMP							
A-C12	24"	22 LF	CMP							
A-CB1			Type 2							
A-CB2			Type 2							
A-C13	24"	39 LF	CMP							
A-C14	4.0' W X 4.0' H	31 LF	Box Culvert / 24" CMP							
A-C15	12"	31 LF	Concrete							
A-C16	12"	30 LF	Concrete							
A-C17	18"	93 LF	Concrete							
A-C18	18"	63 LF	Concrete							
A-C19	12"	43 LF	Concrete							
A-CB3										
A-C20										
A-C21	36"	94 LF	Concrete							

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN A: No Name Slough Basin

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
	36"	94 LF	Concrete							
A-C22	36"	41 LF	Concrete							
A-C23	30"	53 LF	Concrete							
A-C24	18"	37 LF	Concrete							
A-C25	12"	31 LF	CPE							
A-C26	18" / 12"		Concrete							
A-C27	12"	63 LF	CPE							
A-C28	24"	69 LF	CPE / Concrete							
A-C29	12"		CPE							
A-C30										
A-CB4										
A-CB5										
A-C31										
A-CB6										
A-CB7										
A-CB8										
A-CB9										
A-C32										
A-C33	18"	43 LF	Concrete							
A-C34	18"	53 LF	Concrete							
A-C80	12"									
A-C35	12"	115 LF	Concrete							
A-C36	12"	71 LF	Concrete							
A-CB9										
A-C37	12"	116 LF	CMP							
A-C38	18"	87 LF	Concrete							
A-C39	18"	35 LF	CMP							
A-C40	12"	39 LF	Concrete							
A-C41	18"	31 LF	Concrete							
A-C41	10"	32 LF	Clay Tile							
A-C42	18"	40 LF	CPE							
A-C43	18"	40 LF	CPE							
A-C44	18"	48 LF	CPE							

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN A: No Name Slough Basin

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
A-C45	12"	34 LF	Concrete							
A-C46	18"	45 LF	Concrete							
A-C47	18"	24 LF	Concrete							
A-C48	12"	43 LF	Concrete							
A-C49	24"	45 LF	Concrete							
A-C50	18"	40 LF	CMP							
A-C51	18"	120 LF	Concrete							
A-C52	12"	45 LF	Concrete/Wood							
A-C53	18"	63 LF	Concrete							
A-CB10			Type 1							
A-C54	14"		CMP							
A-C55	12"	63 LF	Concrete							
A-C56	18"	48 LF	Concrete							
A-C57	30"	48 LF	CPE							
A-C88	36"		CPE							
A-C59	18"	49 LF	Concrete							
A-C81	30"		CPE							
A-C82	30"		CPE							
A-C83	30"									
A-C87	30"		CPE							
A-C64	36"									
A-C71	18"	40 LF	Concrete							
A-C84	24"	31 LF	Concrete (submerged)							
A-C72	24"	54 LF	Concrete							
A-C85	24"	30 LF	Buried CMP							
A-C86	18"	40 LF	Concrete							

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
Joe Leary Slough										
B-C1	(12) 4' dia. with Tide Gates									0+00
B-B1	Bridge	100 LF	Concrete							
B-X1	Cross Section									10+00
			GS =	3.8						44.9' LT
			Top =	2.8						27.8' LT
			Grade Break =	-1.3						18.0' LT
			Toe =		-3.9					9.0' LT
										Channel Width = 17.9'
			Toe =		-4.0					8.9' RT
			Grade Break =	-1.5						16.9' RT
			Top =	3.2						23.0' RT
			GS =	3.9						43.0' RT
B-X2	Cross Section									52+60
			GS =	6.9						33.1' LT
			Top =	6.5						26.9' LT
			Grade Break =	0.0						13.0' LT
			Toe =		-4.6					9.9' LT
										Channel Width = 17.0'
			Toe =		-4.6					7.1' RT
			Grade Break =	-1.6						13.1' RT
			Top =	3.4						23.6' RT
			GS =	4.2						64.7' RT
B-C2	15' w.X 11.5'h.	30 LF	Arched CMP		-6.7	-7.3	0.0200			In = 54+40, Out = 54+10
B-X3	Cross Section									55+60
			GS =	2.5						63.9' LT
			Top =	3.2						25.8' LT
			Grade Break =	-0.5						10.6' LT
			Toe =		-5.2					6.6' LT

BASIN B: Joe Leary Slough										
Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
										Channel Width = 17.1'
			Toe =		-4.3					10.5' RT
			Grade Break =	-0.4						13.5' RT
			Top =	3.0						19.5' RT
			GS =	4.0						46.7' RT
Branch from the Southwest - Basin B-2 (Persons Road)										
B-C14	18"		Concrete							
Return to Joe Leary Slough										
B-X4	Cross Section									115+00
			GS =	6.8						60.3' LT
			Top =	6.3						24.8' LT
			Grade Break =	2.2						18.8' LT
			Grade Break =	0.6						9.2' LT
			Toe =		-3.8					6.4' LT
										Channel Width = 12.4'
			Toe =		-3.8					5.8' RT
			Grade Break =	0.2						8.8' RT
			Grade Break =	2.8						19.0' RT
			Top =	9.5						29.9' RT
			GS =	10.0						36.4' RT
B-B2	Bridge	74 LF	Concrete	10.5						117+30
B-X5	Cross Section									121+40
			GS =	4.7						49.5' LT
			Top =	6.1						19.3' LT
			Grade Break =	2.0						10.0' LT
			Toe =		-2.8					7.0' LT
										Channel Width = 14.3'
			Toe =		-3.0					7.3' RT
			Grade Break =	0.0						11.3' RT

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
			Grade Break =	1.0						16.4' RT
			Top =	6.2						26.2' RT
			GS =	6.7						56.4' RT
B-X6	Cross Section									155+00
			Top =	6.8						25.1' LT
			Grade Break =	1.2						10.5' LT
			Toe =		-2.8					7.0' LT
										Channel Width = 15.7'
			Toe =		-1.5					8.7' RT
			Top =	4.9						21.5' RT
B-C3	15' w.X 11.5'h.	40 LF	Arched CMP with Concrete Footing		-2.7	-2.9	0.0050			In = 158+30, Out = 157+90
										158+10 @ CL Allen West Road
B-X7	Cross Section									160+60
			GS =	5.9						69.2' LT
			Top =	5.4						21.3' LT
			Grade Break =	1.3						7.9' LT
			Toe =		-1.7					2.9' LT
										Channel Width = 8.7'
			Toe =		-3.2					5.8' RT
			Grade Break =	0.8						10.3' RT
			Top =	6.1						23.6' RT
			GS =	5.7						76.3' RT
			GS =	7.9						101.1' RT
B-B3	Bridge		Wood	9.9						203+66
B-X8	Cross Section									203+66
	Wood Bridge		GS =	11.5						60.0' LT
			Top N. Side Bridge =	9.9						7.5' LT
			Toe =		-2.8					7.0.' LT

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
										Channel Width = 12.7'
			Toe =		-2.8					5.7' RT
			Top S. Side Bridge =	9.9						7.5' LT
			GS =	14.1						60' RT
B-B4			Wood	12.3						211+52
B-X9	Cross Section									216+90
			GS =	11.4						117.5' LT
			Top =	10.9						19.3' LT
			Grade Break =	1.6						10.0' LT
			Toe =		-2.4					5.0' LT
										Channel Width = 11.4'
			Toe =		-2.3					6.4' RT
			Grade Break =	0.7						11.4' RT
			Top =	6.9						27.9' RT
			GS =	12.4						80.4' RT
B-C4	15' W. X 9' H.	24 LF	Arched CMP		-2.4	-2.4	0.0000			In = 219+21, Out 218+97
										219+09 @ CL Benson Heights Place
B-X10	Cross Section									220+20
			GS =	13.4						75.8' LT
			Top =	8.7						28.0' LT
			Grade Break =	1.3						12.4' LT
			Toe =		-1.2					7.4' LT
										Channel Width = 13.9'
			Toe =		-0.6					6.5' RT
			Grade Break =	1.4						9.5' RT
			Top =	8.0						19.5' RT
			GS =	10.4						42.1' RT
B-B5	Bridge		Wood	10.0						225+55

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
B-B6	Bridge		Wood							242+72
B-X11	Cross Section		Top Bridge (N. End) =	13.0						20.5' LT
			Toe =		-0.4					7.1' LT
										Channel Width = 14.3'
			Toe =		-0.4					7.2' RT
			Top Bridge (S. End) =	13.0						20.5' RT
B-X12	Cross Section									247+60
			GS =	13.1						53.9' LT
			Top =	13.4						25.3' LT
			Grade Break =	2.3						9.3' LT
			Toe =		-0.4					6.3' LT
										Channel Width = 11.3'
			Toe =		0.0					5.0' RT
			Grade Break =	2.0						7.0' RT
			Top =	13.7						25.0' RT
			GS =	12.2						63.5' RT
Joe Leary Slough Splits into Maiben Road Ditch and South Spur Ditch										249+25 = 500+00 @ Fork
South Spur Ditch										500+00 @ Fork
B-B7			Wood	17.4						500+97
B-X13	Cross Section									501+10
			GS =	13.7						73.5' LT
			Top =	13.3						24.0' LT
			Grade Break =	4.0						9.7' LT
			Toe =		0.0					6.7' LT
			Water Depth (date) =		1.4					Channel Width = 13.2'
			Toe =		0.0					6.5' RT
			Grade Break =	3.3						10.5' RT
			Top =	16.2						23.0' RT

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
			GS =	15.7						74.0' RT
B-X14	Cross Section									508+60
			GS =	15.7						56.0' LT
			Top =	16.2						23.8' LT
			Grade Break =	13.7						7.4' LT
			Toe =		-0.8					3.7' LT
			WaterDepth (date) =		1.4					Channel Width = 10.7'
			Toe =		0.2					7.0' RT
			Grade Break =	2.7						11.5' RT
			Top =	14.9						31.4' RT
B-X15	Cross Section									515+30
			GS =	9.6						67.8' LT
			Top =	8.6						17.1' LT
			Toe =		0.5					7.6' LT
			Water Depth (date) =		1.2					Channel Width = 13.4'
			Toe =		0.2					5.8' RT
			Top =	12.1						21.5' RT
			GS =	11.5						70.0' RT
B-X16	Cross Section									529+00
			GS =	6.0						61.3' LT
			Top =	6.6						11.8' LT
			Toe =		-0.3					4.3' LT
			Water Depth =		1.2					Channel Width = 10.9'
			Toe =		0.0					6.6' RT
			Top =	7.3						15.5' RT
			GS =	8.9						66.3' RT
B-X17	Cross Section									535+00
			GS =	6.6						65.9' LT
			Top =	7.3						14.3' LT

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
			Toe =		0.5					7.8' LT
			Water Depth (date) =		1.2					Channel Width = 16.5'
			Toe =		0.5					8.7' RT
			Grade Break =	5.5						11.9' RT
			Top =	7.8						17.7' RT
			GS =	7.9						68.3' RT
B-X18	Cross Section									547+00
			GS =	6.1						60.7' LT
			Top =	6.3						10.6' LT
			Toe =		0.3					1.7' LT
			Water Depth (date) =		1.5					Channel Width = 3.9'
			Toe =		0.3					2.2' RT
			Grade Break =	4.5						6.5' RT
			Top =	7.4						13.0' RT
			GS =	8.4						65.3' RT
B-X30	Cross Section							64/65		553+71
			GS =	5.6						35.0' LT
			Top =	5.7						10.0' LT
			Toe =		-1.0					3.5' LT
			Water Depth (13JUL05) =		2.0					Channel Width = 7.0'
			Toe =		-1.0					3.5' RT
			Top =	6.4						12.0' RT
			GS =	7.2						40.0' RT
B-C5	14' w.X 9'h.	85 LF	Arched CMP @ Josh Wilson Road		0.8	1.8	-0.0118			555+14 @ C.L. Josh Wilson Rd
Branch from West (Josh Wilson Road)										
B-C8										
B-C9	12"	39 LF	CMP							
B-C10	18"	43 LF	Concrete							
B-C11	18"	250A LF	CMP							

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
Return to South Spur Ditch										
B-X31	Cross Section									556+38
			Top @ E.P. =	12.3						35.0' LT
			GS =	12.2						32.5' LT
			GS =	5.7						12.0' LT
			Top =	3.9						8.0' LT
			Toe =		0.2					7.5' LT
			Water Depth (13JUL05) =		1.9					Channel Width = 7.5'
			Toe =		0.2					0' LT-RT
			Top =	5.5						3.5' RT
			GS =	7.2						46.0' RT
B-X33	Cross Section									575+15
			Top @ E.P. =	11.7						29.5' LT
			Grade Break =	11.5						24.5' LT
			Top =	5.1						7.5' LT
			Toe =		0.4					5.0' LT
			Water Depth (13JUL05) =		1.8					Channel Width = 9.0'
			Toe =		0.4					4.0' RT
			Top =	7.0						9.5' RT
			GS =	7.9						49.5' RT
B-C6	13.5' w.X 9.5'h.	80 LF	Arched CMP @ Michael Place		-0.5	-0.1	-0.0050			576+00 @ C.L. Michael Road
B-X34	Cross Section									576+79
			Top @ E.P. =	12.0						29.0' LT
			Grade Break =	11.4						25.0' LT
			Top =	4.7						5.0' LT
			Toe =		0.5					3.0' LT
			Water Depth (13JUL05) =		1.7					Channel Width = 8.0'
			Toe =		0.5					5.0' RT
			Top =	7.5						10.0' RT

BASIN B: Joe Leary Slough										
Desc.	**Diameter/Width**	**Length**	**Material**	**Top**	**Inlet I.E.**	**Outlet I.E.**	**Slope**	**LL Page**	**Points**	**Station**
			GS =	8.1						51.0' RT
B-X19	Cross Section									589+40
			E.P. =	12.0						60.8' LT
			Top @ E.P. =	12.4						27.4' LT
			Grade Break =	5.4						4.2' LT
			Toe =		1.2					2.7' LT
			Water Depth =		2.3					Channel Width = 7.5'
			Toe =		1.1					4.8' RT
			Top =	8.9						11.8' RT
			GS =	9.0						61.8' RT
B-X35	Cross Section							58/59		594+87
			Top @ E.P. =	14.0						27.0' LT
			Grade Break =	13.6						23.5' LT
			Grade Break =	9.4						9.7' LT
			Top =	5.3						0.0' LT
			Toe =		2.3					2.0' RT
			Water Depth (13JUL05) =		0.4					Channel Width = 5.5'
			Toe =		2.3					7.5' RT
			Top =	9.3						15.5' RT
			GS =	9.9						55.0' RT
Field Ditch from South										
B-C16	36"	42 LF	CPE		2.30	2.38	-0.0019	58/59		0+08 on South Field Ditch
B-X37	Cross Section							62/63		1+38
			GS =	8.3						34.0' E
			Top =	9.2						9.0' E
			Toe =		3.1					1.0' E
			Water Depth (13JUL05) =		0.0					Channel Width = 2.0'
			Toe =		3.1					1.0' W

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
			Top =	9.2						8.0' W
			GS =	9.1						33.0' W
B-X38	Cross Section							62/63		9+21
			GS =	8.1						35.0' E
			Top =	8.9						11.0' E
			Toe =		3.0					3.0' E
			Water Depth (13JUL05) =		0.0					Channel Width = 6.0'
			Toe =		3.0					3.0' W
			Top =	7.9						8.5' W
			GS =	8.2						34.0' W
B-X39	Cross Section							62/63		12+79
			GS =	8.3						35.0' LT
			Top =	9.0						10.0' LT
			Toe =		2.9					2.0' LT
			Water Depth (13JUL05) =		0.0					Channel Width = 4.0'
			Toe =		2.9					2.0' RT
			Top =	8.6						9.0' RT
			GS =	8.6						34.0' RT
B-C17	18"	38 LF	Concrete		3.41	3.93	-0.0137	64/65		12+89 on South Field Ditch
Return to South Spur Ditch										
B-X36	Cross Section							58/59		596+56
			Top @ E.P. =	13.8						26.5' LT
			Grade Break =	13.3						23.0' LT
			Grade Break =	10.5						11.5' LT
			Top =	5.4						2.0' LT
			Toe =		1.9					0.5' RT
			Water Depth (13JUL05) =		0.8					Channel Width = 6.0'
			Toe =		1.9					6.5' RT
			Top =	9.5						15.5' RT

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN B: Joe Leary Slough										
Desc.	**Diameter/Width**	**Length**	**Material**	**Top**	**Inlet I.E.**	**Outlet I.E.**	**Slope**	**LL Page**	**Points**	**Station**
			GS =	9.3						53.5' RT
B-C7	14.2' w.X 10'h.	63 LF	Arched CMP @ Avon-Allen Road		3.0	3.6	-0.0095			In = 610+13, Out = 609+50
Maiben Road Ditch										249+25 @ Fork
B-X20	Cross Section									250+40
			GS =	11.9						63.6' LT
			Top =	11.1						12.7' LT
			Grade Break =	2.9						2.3' LT
			Toe =		0.4					0.7' RT
										Channel Width = 9.0'
			Toe =		0.2					9.7' RT
			Grade Break =	5.7						12.2' RT
			Top =	12.9						23.2' RT
			GS =	13.4						56.4' RT
B-B8	Bridge		Wood / Steel Cross Supports	15.3						253+60
B-X21	Cross Section									261+80
			GS =	11.7						70.0' LT
			Top =	12.4						19.7' LT
			Toe =		1.4					2.3' LT
			Water Depth (Date) =		2.5					Channel Width = 5.3'
			Toe =		1.1					3.0' RT
			Grade Break =	4.3						5.0' RT
			Top =	13.0						16.0' RT
			GS =	12.0						65.5' RT
B-X22	Cross Section									276+70
			GS =	12.8						68.2' LT
			Top =	12.2						15.5' LT
			Toe =		2.5					3.8' LT
			Water Depth =		3.5					Channel Width = 5.1'

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN B: Joe Leary Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
			Toe =		1.6					1.3' RT
			Top =	13.3						12.8' RT
			GS =	12.4						47.8' RT
B-X23	Cross Section									292+25
			GS =	13.8						61.7' LT
			Top =	12.6						17.8' LT
			Grade Break =	6.0						4.1' LT
			Toe =		3.0					2.4' LT
			Water Depth (Date) =		4.7					Channel Width = 6.3'
			Toe =		3.7					3.9' RT
			Grade Break =	5.3						4.5' RT
			Top =	13.5						13.5' RT
			GS =	12.8						65.2' RT
B-X24	Cross Section									299+55
			GS =	14.4						41.3' LT
			Top =	14.0						16.7' LT
			Grade Break =	7.1						5.2' LT
			Toe =		3.7					1.0' LT
			Water Depth (Date) =		4.8					Channel Width = 6.2'
			Toe =		3.7					5.2' RT
			Top =	13.7						14.8' RT
			GS =	13.2						61.0' RT
B-B9	Bridge		Wood	15.9						306+30
B-X25	Cross Section									314+40
			GS =	17.2						60.9' LT
			Top =	17.0						22.4' LT
			Grade Break =	11.9						16.2' LT
			Grade Break =	7.0						4.0' LT
			Toe =		4.6					1.6' LT

BASIN B: Joe Leary Slough										
Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
			Water Depth (Date) =		5.2					Channel Width = 6.6'
			Toe =		4.1					5.0' RT
			Top =	16.7						16.7' RT
			GS =	16.6						60.3' RT
B-X26	Cross Section									324+60
			GS =	14.9						62.0' LT
			Top =	14.9						18.9' LT
			Grade Break =	10.5						11.1' LT
			Toe =		5.4					3.9' LT
			Water Depth =		5.9					Channel Width = 9.5'
			Toe =		5.4					5.6' RT
			Top =	16.8						20.0' RT
			GS =	16.3						58.7' RT
B-X40	Cross Section		GS =	16.0						68.9' LT
			Top =	16.7						19.7'LT
			Grade Break =	11.4						8.1' LT
			Toe =		6.2					0.3
			Water Depth =		7.4					Channel Width = 6.4'
			Toe =		6.5					6.1' RT
			Top =	15.4						15.4'RT
			GS =	15.5						66.4' RT
B-C15	14' W. X 9' H. CMP	50 LF	CMP		7.3	6.8	0.0100			In = 341+38, Out = 340+88
Basin B-3 (Persons Road)										
B-C12	18"		CMP							
B-C13	18"		Concrete							

BASIN C: Indian Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
Indian Slough										
Bay View-Edison Road										
		71 LF	Conc Bridge @ Bay View-Edison Rd	10.5				23	1000-1001	E. Side = Start 0+00
C-OUT	Dam	41 LF	Concrete Dam with (7) 48" Steel Tide Gates	8.1		-3.5		21	1003-1004	1+40.8
	Lift Station	47 LF	24" Ductile Iron with Exterior Glaze with Tide Gate			2.8		21/25	1006	Out = 1+19.4, 35.6' LT
		46 LF	24" CMP with Tide Gate			3.0		21/25	1005	Out = 1+20.7, 32.8' LT
C-X1	Cross Section							36		3+12
			Top Dike =	9.0						
			Toe =		-5.6					22' South of Top
			Water Level (5-14-03) =		-1.6					Channel Width = 96'A
			Toe =		-5.6					26' North of Top
			Top Dike =	8.5						
C-C56	30"	49 LF	CMP with Tide Gate		-3.0	-3.4	0.0082	25	11,O=1012	Out = 17+93.5, 32.9' RT
C-C71	30"	41 LF	Concrete		-2.4	-3.7	0.0321	68	126-127	
C-C72	36"	264 LF	CMP		-2.5	-3.6	0.0042	68	125-128	
C-B1	10.2'	49 LF	Wood - Railroad Crossing					2	1013-1014	N. Side @ C.L. = 29+02.4
	Cross Section		Top Bulkhead (West) =	7.4						20.5' LT
			Top Ditch Channel =	-1.5						6.5' LT
			C.L. Channel =		-5.4					Channel Width = 13'
			Top Ditch Channel =	-1.5						6.5' RT
			Top Bulkhead (East) =	7.5						28.8' RT
C-C55	36"	30 LF	CMP Tide Gate (Outlet to Ctr CB)	1.4		-1.6		24/25	1015	29+96.9, 27.1' RT
	36"	30 LF	CMP Tide Gate (Outlet to Ctr CB)	1.4		-1.6		24/25	1016	29+98.0, 25.6' RT
			CB Type 2	5.8				24/25	1017	29+99.9, 55.6' RT

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN C: Indian Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
	36"	256 LF	CMP (End Pipe to Ctr CB)		-3.2			24/25	1018	
	36"	256 LF	CMP (End Pipe to Ctr CB)		-3.2			24/25	1019	
C-X40	Cross Section		GS =							37+43
			Top RR Grade =	17.8						75.3' LT
			Grade Break =	6.0						67.8' LT
			Toe R/R =	0.8						55.5' LT
			Top Canal Ditch =	-0.8						11.2' LT
			Toe Canal Ditch =	-3.2						7.1' LT
										Channel Width = 15.0'
			Toe Canal Ditch =	-3.6						7.9'RT
			Top Canal Ditch =	-0.9						10.2' RT
			Toe Concrete Bulkhead =	-0.8						23.0' RT
			Top Concrete Bulkhead =	4.9						25.0'RT
			Top =	5.9						49.5' RT
			Toe =	3.5						69.1' RT
			EP =	4.3						75.7' RT
C-X2	Cross Section							36		53+92
			Top Railroad Grade =	7.2						63' LT
			Grade Break =	1.9						56' LT
			Grade Break =	0.1						13' LT
			Toe =		-2.8					9' LT
			Centerline							Channel Width = 18'
			Toe =		-3.1					9' RT
			TOP =	6.7						34' RT
			GS =	8.2						49' RT
C-B2	49.4'	64 LF	Concrete Bridge @ Farm-to-Market Road					2	1020-1021	W. Side @ C.L. = 63+92.3
	Cross Section		Top Road Grade (NW) =	9.6						25' LT
			Top Ditch =	0.8						9' LT
			Top Ditch =		-2.8					6' LT
			Centerline Ditch =		-4.1					Channel Width = 11.5'

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN C: Indian Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
			Water Level (2-11-03) =		-1.5					
			Top Ditch =		-3.5					5.5' RT
			Top Ditch =	7.0						24.4' RT
			Top Road Grade (SW) =	10.1				2	1021	24.4' RT
C-X3	Cross Section							38		77+24
			GS =	11.1						50.5' LT
			Top =	10.0						25.5' LT
			Toe =		-3.1					5.5' LT
			Water level (5-12-03)		-0.5					Channel Width = 11'
			Toe =		-2.8					5.5' RT
			Top =	9						21.5' RT
			GS =	10.1						41.5' RT
C-C73	36"	111 LF	LCPE		3.1	2.9	0.0018	70	134-135	84+77.3
C-C74	36"	47 LF	Concrete		2.8	2.0	0.0169	70	132-133	84+71.3
C-C75	30"	49 LF	LCPE/Concrete		0.3	0.1	0.0041	70	130-131	84+72.6
C-B3	63.7'	62 LF	Concrete Bridge @ SR 20	N = 11.1				3/25	1022-1023	95+45.4
	Cross Section		Top Bulkhead (N) =	11.1						30' LT
			Top Ditch =	7.6						19' RT
			Grade Break =	2.9						9' RT
			Top Ditch =		-1.3					6' RT
			Centerline =		-3.2					Channel Width = 12'
			Top Ditch =		-1.4					6' LT
			Top Ditch =	8.2						18' LT
			Top Road Grade =	10.7				3	1024-1025	33.7' LT
Branch from the Southwest										
C-B10			Concrete Bridge @ SR 536							

BASIN C: Indian Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
C-C54	36"	50 LF	(2) 36" CMP							
			West Pipe =		0.7	0.7	0.0000	37		
			East Pipe =		0.9	0.9	0.0000	37		
Return to Main Channel										
C-X4	Cross Section							38		118+63
			Top =	7.1						15' LT
			Toe =		-1.7					4.0' LT
			Water Level (5-12-03)		0.2					Channel Width = 8'
			Toe =		-1.5					4.0' RT
			Top =	7.8						12.5' RT
			GS =	10.4						37.5' RT
C-C1	16.3' W X 9.6' H	63 LF	Arched CMP @ Bradshaw Road	10.3	0.7	0.7	0.0000	3	1032-1033	Out = 125+28.8
			Top Road	12.4						In = 125+91.4
C-B4		62 LF	Concrete Bridge @ SR 20	N = 14.8				4	1034-1035	Out = 132+26.8
										In = 132+67.3
C-B5		69 LF	Wood - Railroad Crossing	N = 15.7				4	1036-1037	Out = 133+24.2
										In = 133+34.4
C-C2	48"	20 LF	CMP		2.0	1.4	0.0300	5	1038-1039	Out = 140+92.4
	36"	20 LF	CMP		3.3			5		In = 141+12.5
	48"	24 LF	Concrete		1.0			5		
C-C3	48"	21 LF	Concrete		2.4	2.6	-0.0094	5	1040-1041	Out = 144+38.2
										In = 144+59.6
C-X5	Cross Section							40		150+57
			GS =	8.5						38' LT
			Top =	8.3						13' LT
			Toe =		-0.2					4.5' LT
			Water Level (5-12-03)		2.7					Channel Width = 9'
			Toe =		-0.3					4.5' RT
			Top =	8.8						13.5' RT
			GS =	9.0						38.5' RT

BASIN C: Indian Slough										
Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
C-C4	10.0' W X 17.0' H	80 LF	Arched CMP @ Higgins Airport Way		3.0	3.0	0.0000	5	1042-1043	Out = 152+57.3
										In = 153+36.9
C-X6	Cross Section							40		155+38
			GS =	10.1						41' LT
			Top =	8.9						16' LT
			Toe =		1.1					5.0' LT
			Water Level (5-12-03)		3.6					Channel Width = 10'
			Toe =		0.8					5.0' RT
			Top =	8.8						16' RT
			GS =	9.9						41' RT
Branch from the North										
C-C15	48"	30 LF	CMP							
C-C16	36"	34 LF	CMP							
C-C17	25"	39 LF	Ductile Iron							
C-C17	25"	40 LF	Ductile Iron							
C-C18	36"	41 LF	Concrete							
C-C19	30"	24 LF	Concrete							
C-C20	18"	30 LF	CMP							
C-C21	18"	60 LF	CMP							
C-C22	12"	20 LF	CPE							
C-C23	12"	60 LF	CPE							
C-C24	15"	43 LF	Ductile Iron							
C-C25	18"	30 LF	CPE					10		
C-C26	18"	36 LF	CPE							
C-C27	18"	36 LF	CMP					13		
Return to Main Channel Indian Slough										
C-C5	48"	21 LF	CMP		2.2	2.8	-0.0291	11	1044-1045	Out = 158+73.8, 2.7' LT
										In = 158+94.3, 3.7' LT
	36"	22 LF	CMP		2.3	2.4	-0.0046	11	1046-1047	Out = 158+73.1, 2.4' RT

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN C: Indian Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
										In = 158+94.8, 3.5' RT
C-X7	Cross Section							40		160+51
			GS =	8.6						42.5' LT
			Top =	8.3						17.5' LT
			Toe =		0.4					5.5' LT
			Water Level (5-12-03)		3.9					Channel Width = 11'
			Toe =		0.4					5.5' RT
			Top =	9.9						17' RT
			GS =	10.8						42' RT
C-C28	25"		DI			4.9		12	1048	166+74.8, 30.7' LT
C-B6		69 LF	Wood - Railroad Crossing	16.7				5	1049-1050	N. Side @ C.L. = 167+32.1
C-B7	49.7'	74 LF	Concrete Bridge @ SR 20	NE = 15.3				6	1052	N. Side @ C.L. = 168+00.8
				NW = 15.5				6	1051	S. Side @ C.L. = 168+60.3
				SW = 15.5				6	1054	
				SE = 15.1				6	1053	
C-CB7	TYPE 1 CB			8.22	7.18	6.81		77	1228	
C-C29	12"		CPE			5.0			1055	174+11.4, 54.3' RT
C-C6	48"	34 LF	CPE		3.9	3.8	0.0029	12	1056-1057	Out = 176+18.3
										In = 176+52.5
C-CB6			Type 1	9.53	8.28	7.9		77	1224	
C-C7	48"	27 LF	Concrete		4.5	3.5	0.0373	12	1058-1059	Out = 189+55.6
										In = 189+82.5

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN C: Indian Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
C-C8	48"	24 LF	CMP		5.2	5.0	0.0082	13	1060-1061	Out = 196+38.9
										In = 196+63.4
C-CB3			Type 1	14.63	12.96	12.78		76	1203	
C-CB4			Type 1	12.98	10.6	10.57		76	1205	
C-CB5			Type 1	12.9	10.09	10		76	1211	
C-C9	36"	32 LF	Concrete		5.7	5.2	0.0158	13	1062-1063	Out = 220+27.8
										In = 220+59.4
C-B8	39.2'	72 LF	Concrete Bridge @ SR 20	NE = 18.5				7/29	1067	S. Side @ C.L. = 232+81.8
				NW = 18.2					1066	N. Side @ C.L. = 233+25.8
				SW = 19.2					1065	
				SE = 19.5					1064	
C-B9		79 LF	Wood - Railroad Crossing	20.0				7/29	1068-1069	N. Side @ C.L. = 234+02.9
C-C58	Arched CMP	44 LF	Arch with Concrete Channel Walls	N = 14.4				7/29	1072	In = 240+77.2
			@ Ovenell Road	S = 14.9					1071	Out = 240+33.3
C-C30	60"	58 LF	Concrete		9.4	9.6	-0.0035	7/29	1074-1075	
C-C10	42"	19 LF	Steel		8.1	7.7	0.0216	7/29	1076-1077	Out = 251+98.6
										In = 252+17.1
C-C11	36"	36 LF	Concrete		8.8	8.6	0.0056	7/29	1078-1079	Out = 257+58.5
										In = 257+93.4
C-C12	4.0' W X 5.5' H	35 LF	Box Culvert @ Avon-Allen Road	E = 18.5				8/29	1081	258+84.7
				W = 18.3				8/29	1080	258+49.8
C-C13	48"	46 LF	Concrete / CMP	E = 12.9	8.9			30	1084	273+73.0
				W = 12.4		8.4	#DIV/0!	30	1083	273+26.7
C-C31	36"	32 LF	CMP							
C-C14	36"	33 LF	Concrete	E = 12.3	9.3			27/30	1086	282+53.4
C-C52	48"	31 LF	CMP	W = 12.6		9.6	-0.3097	27/30	1085	282+20.2
C-C51	12"	106 LF	Concrete							

BASIN C: Indian Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
C-C50	48" W X 36" H	160 LF	CMP							
C-CB2			Type 2							
C-CB1		245 LF	Type 2							
C-C49	72" W X 36" H	54 LF	Arched CMP							
C-C48			Concrete, Broked End, Buried							
C-C47										
Little Indian Slough										
Bayview-Edison Road										
C-OUT2	48"	59 LF	CMP - Concrete Headwall (North)		-4.1	-4.0	-0.0017	16/32	1101-1103	Out = 0-58.6
	48"	59 LF	CMP - Concrete Headwall (South)		-4.3	-4.1	-0.0034	16/32	1100-1102	In = 0+00
C-C33	30"	25 LF	Concrete		-3.6	-3.4	-0.0079	16/32	1106-1107	Out = 27+23.4
										In = 27+48.6
C-C34	24"	20 LF	Concrete		-2.5	-2.7	0.0099	16/32	1121-1122	Out = 35+10.4
										In = 35+30.6
C-C44	18"	22 LF	Concrete			-2.1		16/32	1123	44+94.0, 8.8' LT
C-C35	14"	28 LF	Metal		-1.4			17/32	1124	
										In = 51+96.6
C-C36	18"	34 LF	Concrete		0.5	0.0	0.0148	7/32	1125-1126	Out = 58+50.0
										In = 58+83.8
C-C37	24"	200 LF	CMP		4.7	3.9	0.0040	9/32	1127-1128	Out = 65+56.3
										In = 67+56.3
C-C38	24"	44 LF	Concrete		5.2	5.2	0.0000	9/32	1136-1137	Out = 73+06.7
										In = 73+51.1
C-C39	18"	18 LF	Concrete		5.3	5.1	0.0109	33	1139-1140	Out = 79+42.8
										In = 79+61.1
C-C40	36"	220 LF	Concrete		4.4	4.2	0.0009	33	1147-1148	Out = 85+96.1
										In = 88+16.0
C-C41			Did Not Find Culvert							
C-C42	18"	16 LF								
C-C43	24"	22 LF								
C-C62	24"		PVC (From Pond)							

Bay View Watershed Stormwater Management Plan Phase 1
Appendix A: Stormwater Facility Inventory

BASIN C: Indian Slough

Desc.	Diameter/Width	Length	Material	Top	Inlet I.E.	Outlet I.E.	Slope	LL Page	Points	Station
C-C63	24"		PVC (From Pond)							
C-C64	12"		CPE							
C-C45	12"	46 LF	Concrete							
C-C46			Submerged							
C-C53	24"	33 LF	Concrete							