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Airport Communities Coalition, Appellant, v. Department of Ecology and The Port of Seattle, Respondents.	PCHB No. 01-160 DECLARATION OF ROGER A. PEARCE SUPPORTING PORT OF SEATTLE'S MOTION FOR PARTIAL SUMMARY JUDGMENT ON SEPA ISSUE

Roger A. Pearce declares as follows:

1. Identity of Declarant. I am one of the attorneys representing respondent Port of Seattle in this action. I am over the age of eighteen, have personal knowledge of the facts stated in this declaration, and am competent to testify to those facts.

2. Identification of Exhibits. Attached as exhibits to this declaration are true and correct copies of the following appendices from the Final Environmental Impact Statement for Proposed Master Plan Update Development Actions at Seattle-Tacoma International Airport ("FEIS") which was issued by the Federal Aviation Authority and Port of Seattle:

- **Exhibit A.** Appendix P (National Resource Mitigation Plan) to the FEIS.
- **Exhibit B.** Portions of Appendix Q (Water Studies) to the FEIS.
- **Exhibit C.** Portions of Appendix G (Hydrologic Modeling Study) to the FEIS.

ORIGINAL

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I declare under penalty of perjury under the laws of the state of Washington that the foregoing is true and correct.

Executed at Seattle, Washington, this 25th day of February 2002.

Roger A. Pearce

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A

APPENDIX P

NATURAL RESOURCE MITIGATION PLAN

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NATURAL RESOURCE MITIGATION PLAN

FOR THE PROPOSED MASTER PLAN UPDATE IMPROVEMENTS AT SEATTLE-TACOMA INTERNATIONAL AIRPORT

Prepared for

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January 15, 1996

TABLE OF CONTENTS

Page

TABLE OF CONTENTS (continued)

AR 003221

TABLE OF CONTENTS (continued)

AR 003222

LIST OF FIGURES

LIST OF TABLES

LIST OF TABLES (continued)

1. INTRODUCTION

1.1 PROJECT DESCRIPTION

The Port of Seattle (Port) is proposing to update the Master Plan of Seattle-Tacoma International Airport (Sea-Tac Airport). Implementation of the proposed updated Master Plan would result in development that could cause significant, unavoidable adverse impacts to natural resources in the project vicinity, most notably to Miller and Des Moines creeks, and 10.35 acres of wetlands (Figure 1.1-1). This report describes the mitigation to compensate for these natural resource impacts.

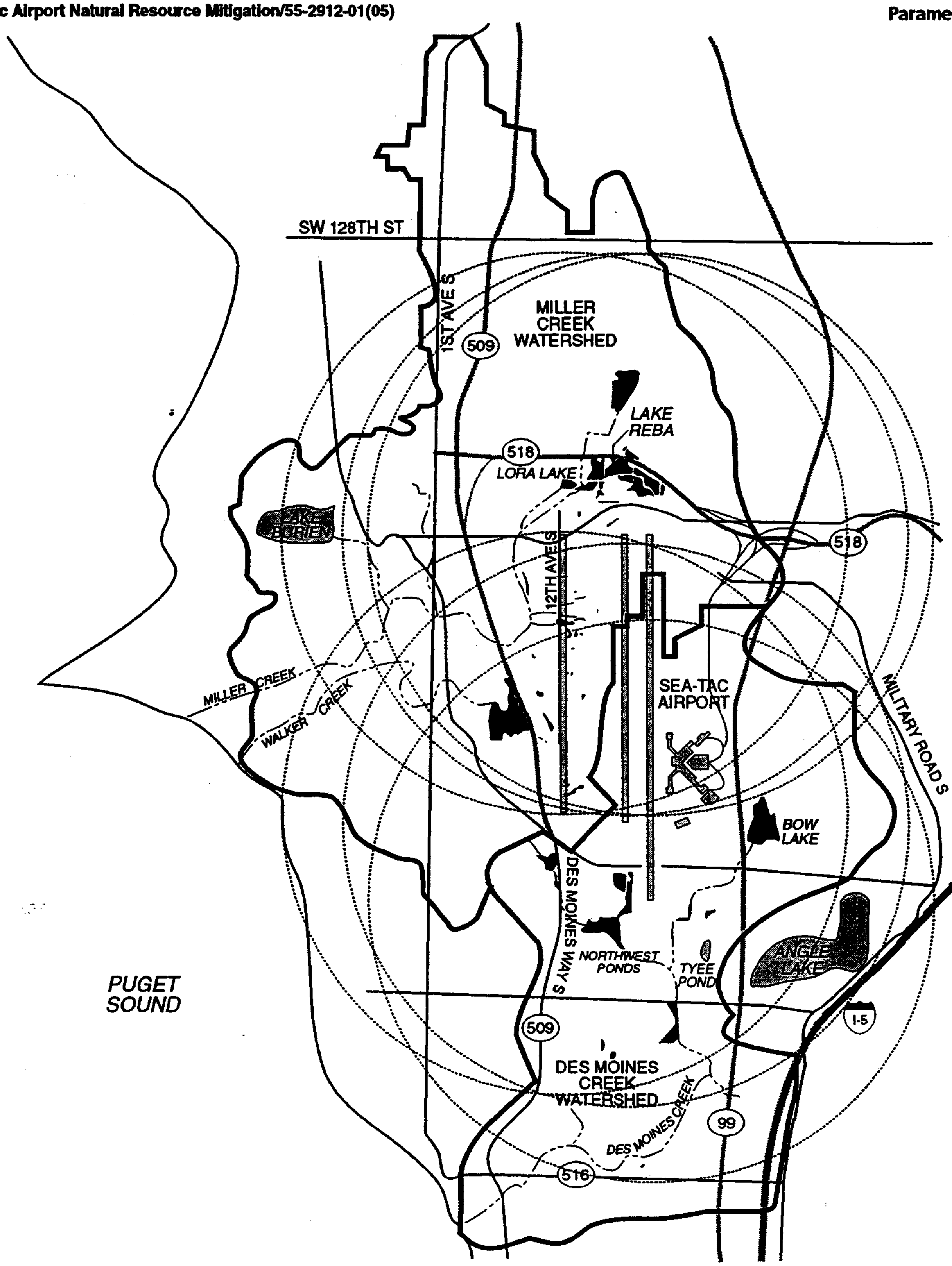
1.1.1 Purpose and Need

As currently configured, Sea-Tac Airport is unable to efficiently meet existing and future regional air travel demands. The airfield operates inefficiently during poor weather because it can accommodate only a single arrival stream. As a result, significant arrival delay occurs during poor weather. Aircraft are either held on the ground in their originating city, slowed en route, or they are placed in holding patterns to await clearance to land at Sea-Tac Airport. These conditions result in the inefficient operation of the existing airfield, as described in Chapter I of the Final EIS.

With or without airport development, airport activity is expected to increase as a consequence of regional population growth. As aviation demands grow, aircraft operating delay would increase exponentially. The increased passenger, cargo, and aircraft operations demands would place increasing burdens on the existing terminal and support facilities. Without improvements, the roadway system, terminal space, gates, cargo and freight processing space would become more inefficient and congested, and the quality of service would be reduced.

Before and during preparation of the proposed Master Plan Update, regional officials identified the following needs:

- Improve the poor weather airfield operating capability to accommodate aircraft activity with an acceptable level of aircraft delay;
- Provide sufficient runway length to accommodate either warm weather operations or payloads for aircraft types operating to the Pacific Rim;
- Provide Runway Safety Areas (RSAs) that meet current Federal Aviation Administration (FAA) standards; and
- Provide efficient and flexible landside facilities to accommodate future aviation demand.



Source: Shapiro 1995e

SCALE IN MILES

0 1

N

10,000 feet from end of runway

Watershed Boundary

Figure 1.1-1
Proposed Master Plan Update Impact Area

1.1.2 Key Project Elements

The proposed Master Plan Update includes the following major components:

- adding a third parallel runway (16X/34X) with a length of up to 8,500 ft and associated taxiway and navigational aids
- extending Runway 34R by 600 ft
- establishing standard RSAs for existing Runways 16R/34L and 16L/34R
- adding a new air traffic control tower
- improving and expanding the main terminal
- improving and expanding parking and access
- developing the South Aviation Support Area (SASA) for cargo and/or maintenance facilities
- relocating, redeveloping, and expanding support facilities.

Those proposed airport improvements that would have the greatest effect on wetlands and streams are the new runway, the Runway 34R extension, and the development of SASA. The analysis in this Natural Resource Mitigation Plan assumes the maximum buildout described in the proposed Master Plan Update Final EIS, including a new 8,500-ft runway. If the Port were to choose to build a shorter runway (less than 8,500 ft), impacts to natural resources, particularly Miller Creek, would be reduced.

1.1.3 Unavoidable Impacts to Wetlands and Streams

1.1.3.1 Wetlands

Some 55 individual wetlands totalling nearly 144 acres occur within the detailed study area used for analysis in the Master Plan Update EIS (EIS). Thirty-four individual wetlands could be directly affected by development at the Airport. The EIS identified 10.35 acres that would be directly affected by proposal implementation. The 21 wetlands that would not be affected include some of the larger wetlands on the airport site.

Significant unavoidable adverse impacts would occur to wetlands in the study area. The impacts include filling, grading, changing hydrology, and removing vegetation.

To mitigate for the unavoidable impacts to wetlands, the Port proposes to create new wetlands on a 47-acre site of an approximately 69-acre parcel located within the city limits of Auburn, Washington. Wetland mitigation at the Airport, within the watersheds where the impacts may occur, is not feasible for three reasons: (1) most of the area surrounding the Airport is developed, and not enough available land exists in the watershed to create compensatory mitigation wetlands without additional business and residential relocation; (2) the FAA is currently finalizing a Draft Circular that states that airports with "wildlife attractions" within 10,000 ft of the edge of any active runway is not recommended by the FAA; and (3) wildlife control activities in wetlands near the airport would conflict with wetland habitat mitigation goals. Because of wildlife attraction issues, the Port cannot commit to maintaining sites on or near the Airport as wetland habitat mitigation in perpetuity. If a site were to become a safety concern because of its attraction to wildlife, particularly birds, and jeopardize aircraft safety, the Port would be compelled to remove it. Safe airport operations are the Port's and the FAA's primary concern. However, the hydrologic functions the wetlands perform would be replaced at the airport site with the proposed storm water management facilities.

1.1.3.2 Streams

Proposed Master Plan Update improvements would affect two streams: Miller Creek, at the northwest corner of the airport property, and Des Moines Creek, at the southern end. Both would require relocation.

<u>Miller Creek</u>

The Airport Master Plan's proposed fill activities would directly affect three areas in the Miller Creek watershed (Shapiro 1995e) due to the proposed new parallel runway embankment. Area 1 includes approximately 980 ft of Miller Creek. The affected portions extend approximately 1,000 ft south of Lora Lake. Area 2 includes Class III tributaries, totaling 2,080 ft, that originate as seeps in the Airport Operations Area (AOA) then flow west to Miller Creek. Area 3 includes 200 ft of the Class III headwaters of Walker Creek. These waters, which originate from seepage and storm water runoff at the corner of 12th Avenue South and South 176th Street, flow northwest to State Route 509 (SR 509).

<u>Des Moines Creek</u>

The relocation of Des Moines Creek was first proposed in the *South Aviation Support Area (SASA) EIS*, a joint NEPA/SEPA document prepared by the Port of Seattle and the FAA. The Final EIS was released in March 1994; the FAA's Record of Decision was made final in September 1994. The proposed Master Plan Update has further refined the layout and contents of SASA, which would require a new realignment plan for Des Moines Creek. This new alignment is assessed in this mitigation plan. Because the proposed Master Plan Update would be implemented in phases, with development at the SASA location occurring relatively late in the process, the final SASA layout cannot be established until that time. If the layout is substantially different than outlined in the SASA Final EIS or the Master Plan Update Final EIS, supplemental environmental review could be required. At that time, the Port would apply for the necessary permits, including those required for the Des Moines Creek relocation, and a detailed mitigation plan would be prepared.

Regardless of the final layout, it is likely that 2.23 acres of wetlands would be filled for the SASA development. Building the compensatory wetlands when implementing the first phases of Master Plan would help ensure that they are functional by the time the actual impacts occur to the existing wetlands at the SASA site.

1.2 PROJECT LOCATION

The proposed project is located at Seattle-Tacoma International Airport, SeaTac, King County, Washington.

The impacted wetlands and streams are located in Sections 20, 21, 28, 29, 32, and 33, Township 23N, Range 4E; and Sections 4 and 5, Township 22N, Range 4E, Willamette Meridian in the Des Moines and Miller Creek watersheds.

The wetland mitigation site lies within the city limits of Auburn, King County, Washington in Section 31, Township 22N, Range 5E, Willamette Meridian in the Green River watershed.

1.3 RESPONSIBLE PARTIES

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1.4 REPORT ORGANIZATION

The report is modeled after *Guidelines for Developing Freshwater Wetlands Mitigation Plans and Proposals* (Ecology 1994). The Natural Resource Mitigation Plan is divided into three major chapters; Wetlands, Miller Creek, and Des Moines Creek. Each chapter is intended to be a separate report that fully describes the mitigation proposal for that particular resource. The chapters discuss (1) the current resource conditions, (2) the goals, objectives, and performance standards of the proposed mitigation, (3) the proposed mitigation site, (4) the proposed mitigation site plan, and (5) provisions for monitoring, protecting, and maintaining the mitigation. Each chapter also discusses proposed contingency measures to be implemented if the established performance standards are not achieved.

2. MITIGATION APPROACH

Federal, state, and local natural resources regulatory programs share a common policy objective; that is, protecting and conserving the biological and physical integrity of our natural resource systems. The agencies implementing these programs have widely overlapping regulatory mandates, thereby requiring an integrated approach to project planning. The National Environmental Policy Act (NEPA) (40 CFR §1508.20) specifically defines the following sequential process for project planning to reduce adverse impacts:

- avoid the impact altogether by not taking a certain action or parts of an action;
- minimize impacts by limiting the degree of magnitude of the action and its implementation;
- rectify the impact by repairing, rehabilitating, or restoring the affected environment;
- reduce or eliminate the impact by preservation and maintenance operations during the life of the action (including monitoring and appropriate corrective measures;) and
- compensate for the impact by replacing or providing substitute resources or environments.

The Port of Seattle used this approach to develop the proposed Master Plan Update. This chapter documents that process. Section 2.1 discusses airport siting, operation, and design alternatives considered to avoid and minimize impacts to natural resources. Section 2.2 identifies the overall intent of the Master Plan Update to mitigate for unavoidable impacts to regulated natural resources. The broad Master Plan Update mitigation goals in Section 2.2 are further defined in separate sections for wetlands and streams (see Sections 3.3, 4.3 and 5.3), with specific goals, objectives, and performance standards.

2.1 MITIGATION SEQUENCING

The planning process that led to the proposal analyzed in the Master Plan Update EIS began in the mid 1980s. During this process, several alternatives that would avoid or minimize the impacts to the wetlands and streams at Sea-Tac Airport were considered. This section describes those alternatives.

2.1.1 Alternatives Considered to Avoid or Minimize Natural Resource Impacts

Several siting, operational, and design alternatives that would avoid or minimize natural resource impacts were analyzed during the Flight Plan Study, the New Major Supplemental Airport Siting Study, and the Master Plan Update. Chapter II and Appendix B of the Master Plan Update FEIS describe the alternatives in further detail.

2.1.1.1 Siting Alternatives

Flight Plan Study

Studies in the late 1980s concluded that the existing two runways at Sea-Tac would not meet regional air travel needs beyond the year 2000. As a result, the Port of Seattle and the regional planning council (now called the Puget Sound Regional Council or PSRC) co-sponsored a process, called the Flight Plan Study, to identify a long-term solution to the region's air transportation needs. The study analyzed alternatives to accommodate demand by replacing or supplementing Sea-Tac Airport. Based on this 2½-year effort, the 1992 Flight Plan Study recommended a multiple airport system that included a new runway at Sea-Tac Airport.

New Major Supplemental Airport Siting Study

In response to the Flight Plan Study and additional study by PSRC, the PSRC General Assembly adopted Resolution A-93-03 in April 1993 to amend the Regional Aviation Systems Plan. The PSRC resolution states " . . .that the region should pursue vigorously, as the preferred alternative, a major supplemental airport and a third runway at Sea-Tac."

The PSRC then studied the feasibility of a major supplemental airport in response to the recommendations of the Flight Plan Study and the subsequent Resolution A-93-03. The Major Supplemental Airport (MSA) Study was to be conducted in two phases. Phase I identified feasible sites and Phase II was to prepare a preliminary site plan for each of the feasible sites.

The Phase I studies resulted in three recommended sites; Arlington, Marysville, and Tanwax Lake. Due, in part, to significant public opposition, Phase II was not implemented. Executive Board Resolution EB-94-01 (October 27, 1994) states that ". . .the Executive Board concludes that there are no feasible sites for a major supplemental airport within the four-county region and that continued examination of any local site will prolong community anxiety while eroding the credibility of regional governance."

2.1.1.2 Operation Alternatives

As stated in Section 1.1.1, the following four operational needs have been identified:

- Improve the poor weather airfield operating capability to accommodate aircraft activity with an acceptable level of aircraft delay;
- Provide sufficient runway length to accommodate either warm weather operations or payloads for aircraft types operating to the Pacific Rim;
- Provide RSAs that meet current FAA standards; and
- Provide efficient and flexible landside facilities to accommodate future aviation demand.

The EIS for the proposed Master Plan Update Improvements, as required by NEPA and the State Environmental Policy Act (SEPA), analyzed a reasonable range of alternatives to meet these four needs. Table 2.1-1 addresses the various alternatives.

As the table suggests, several alternatives were considered that would avoid impacts to natural resources at the airport. The alternatives were rejected, however, because they did not meet the four operational needs identified in the proposed Master Plan Update.

2.1.1.3 Design Alternatives

The Master Plan Update EIS lists several design measures that would be implemented to minimize the natural resource impacts. These measures include:

- using off-site fill to avoid approximately 19 acres of wetlands in an otherwise feasible on-site borrow area
- using retaining walls rather than sloped fill to avoid direct impacts to portions of Miller Creek

2.1.2 <u>Proposed Compensation</u>

The remainder of this Natural Resource Mitigation Plan outlines the Port's proposal to compensate for the unavoidable impacts to wetlands and streams that full implementation of the proposed Master Plan Update would cause.

2.2 GOAL DEVELOPMENT FOR NATURAL RESOURCE MITIGATION PLAN

The federal wetlands "no net loss" standard aims to achieve no overall net loss of wetland acreage and function and to increase the nation's quality and quantity of wetlands resources through restoration and creation. This policy objective is central to the mitigation approach for unavoidable impacts to stream and wetland resources resulting from implementation of the proposed Master Plan Update. The goals for this program broadly define the intent to compensate for unavoidable wetland and stream impacts, by providing appropriate replacement resources both on- and off-site. The potential impacts to biological and physical functions (discussed in Sections 3.2, 4.2, and 5.2) are emphasized in the mitigation goals to ensure that objectives and performance standards are appropriate, measurable, and achievable. The overall goals identified below are further defined in separate sections for wetlands and Miller Creek (Sections 3.3 and 4.3, respectively); they are accompanied by design objectives and performance standards appropriate to each resource.

Goal 1. Achieve no overall net loss of wetland acreage and stream length.

Goal 2. Create diverse native wetland and riparian plant communities and streambed habitat with equal or greater functional value for wildlife and fish.

Goal 3. Enhance airport operations safety, consistent with FAA guidelines, by providing off-site replacement habitats for wildlife species that create a potential hazard for aircraft.

Goal 4. Achieve no net loss of 100-year floodplain storage.

AR 003235

Table 2.1-1. Summary of alternatives considered.

	Alternative	Evaluation
1.	**Improve the poor weather airfield operating capability to accommodate aircraft activity with an acceptable level of aircraft delay**	
A.	Use of other modes of transportation	Not considered further, as this alternative will not address the poor weather operating issues at Sea-Tac. Less than 5% of passengers using Sea-Tac are traveling to distances where surface transportation is efficient and cost effective.
B.	Use of other airports or construction of a new airport.	Not considered further. Regional consensus has been established through PSRC EB-94-01 as: (1) there is no sponsor or funding for a new airport; (2) extensive studies of these alternatives indicate that there are no feasible sites; (3) if a site could be identified, market forces and planning/development requirements would prevent the airport from successfully serving regional demand until 2010 or later. The FAA and Port have independently concluded that a new airport would not satisfy the needs addressed by the EIS.
C.	Activity/demand management	Not considered further, as these actions will not eliminate the poor weather operating need.
D.	Runway development at Sea-Tac	To be considered further: Runway lengths from 7,000 ft to 8,500 ft (Alternatives 2, 3, and 4 in the EIS).
E.	Use of Technology	Not considered further. No technologies currently exist, or are planned, to address the poor weather operating constraint at Sea-Tac.
F.	Delayed or Blended Alternative (Combination of other modes, use of existing airports, and activity/demand management)	The net result of this alternative would be a delay in the implementation of the Master Plan Update alternatives. Because there is no commitment to any individual or combination of elements and because aviation activity levels are currently growing at a rate slightly higher than forecast, this alternative was not considered further.
G.	Do-nothing/no-build	To be considered further (Alternative 1 in the EIS)
2.	**Provide sufficient runway length to accommodate warm weather operations without restricting passenger load factors or payloads for aircraft types operating to the Pacific Rim.**	
A.	Extension of Runway 16L/34R to 12,500 ft	To be considered further, as this is presently the longest runway.
B.	Extension of Runway 16R/34L to 12,500 ft	Not considered further due to the cost of addressing impacts to South 188th Street.
C.	Development of a new 12,500 ft long runway	Not considered further due to substantial community disruption and unnecessary cost that would result.
D.	Delayed Alternative	Not considered further, as it would not address the needs of Sea-Tac.
E.	Do-nothing/no-build	To be considered further (Alternative 1 in the EIS)

Table 2.1-1. **Summary of alternatives considered (continued).**

	Alternative	Evaluation
3.	**Provide RSAs that meet current FAA standards.**	
A.	Displaced threshold/declared distance procedures	Considered as the do-nothing/no-build
B.	Clearing, grading, and development of areas for 1,000 ft beyond the existing pavement	Considered further.
C.	Clearing, grading for 1,000 ft including the 600-ft extension to 34R	Considered further.
D.	Delayed Alternative	Not considered further, as it would not address the RSA requirements. However, this would be the same as do-nothing.
E.	Do nothing/no-build	To be considered further for declared distances (Alternative 1).
4.	**Provide efficient and flexible landside facilities to accommodate future aviation demand.**	
A.	Use of other modes of transportation	Not considered further, as less than 5% of the future passengers using Sea-Tac are traveling to distances where surface transportation is efficient and cost effective and likely to be used.
B.	Use of other airports or construction of a new airport	Not considered further. Regional consensus has been established through PSRC EB-94-01 as: (1) There is no sponsor or funding for a new airport; (2) Extensive studies of these alternatives indicate that there are no feasible sites; (3) If a site could be identified, market forces and planning/development requirements would prevent the airport from successfully serving regional demand until 2010 or later.
C.	Activity/demand management	Not considered further, as these actions will not reduce demand.
D.	Landside development at Sea-Tac	To be considered further: Three primary alternatives to be considered further: Central Terminal Development, North Unit Terminal Development and South Unit Terminal Development (Alternatives 2, 3, and 4, respectively in the EIS).
E.	Delayed or Blended alternative (combination of other modes, use of existing airports, and activity/demand management)	The net result of this alternative would be a delay in the implementation of the Master Plan Update alternatives. Because there is no commitment to any individual or combination of elements and because aviation activity levels are currently growing at a rate slightly higher than forecast, this alternative was not considered further.
F.	Do-nothing/no-build	To be considered further (Alternative 1 in the EIS).

Source: Landrum & Brown 1995

3. WETLANDS

The mitigation plan for wetland impacts associated with proposed Master Plan Update improvements is presented in this chapter. Sections of the proposed plan generally correspond to *Guidelines for Developing Freshwater Wetlands Mitigation Plans and Proposals* (Ecology 1994). Affected wetlands were delineated and characterized for their biological and physical functions, which provided the basis for selecting a mitigation site and developing this plan. Goals, design objectives, and performance standards are identified to guide construction of the mitigation wetland and to provide long-term standards for measuring mitigation success.

3.1 WETLAND DELINEATION OF IMPACT AREA

Shapiro and Associates, Inc. conducted a detailed wetland investigation of the Sea-Tac Airport Master Plan Update study area from August to December 1994 (Shapiro 1995b). By reviewing existing literature, conducting a field reconnaissance, and using air photo interpretation, 55 wetlands were identified on both Port-owned and adjacent private land. Of these, 32 wetlands ranging in size from approximately 0.02 to 18.10 acres were delineated and surveyed as part of the Shapiro study. Wetland 27 is subject to fill under authority of an approved Section 404 Nationwide 26 permit. Wetlands were delineated using the criteria described in the *Federal Manual for Identifying and Delineating Jurisdictional Wetlands* (FICWD 1989). The Federal Manual's Intermediate-Level On-site Determination Method was chosen to determine wetland boundaries. Delineated wetland boundaries do not differ from those that would be identified using the criteria described in the *U.S. Army Corps of Engineers Wetlands Delineation Manual* (Environmental Laboratory 1987).

Of the remaining 23 wetlands not delineated by Shapiro, 10 had been inventoried during previous studies (CH2M Hill 1995; Parametrix 1994; Sea-Tac 1993; Sheldon 1992) and 13 wetlands were not delineated because permission to access private properties containing these wetlands could not be obtained. Figure 3.1-1 shows the locations of wetlands in the Sea-Tac Master Plan Update study area. Table 3.1-1 lists the size, Cowardin (1979) classification, and dominant vegetation communities for each wetland. The complete jurisdictional wetland determination for the proposed Sea-Tac Airport Master Plan Update improvements is included in the EIS as Appendix H-A (Shapiro 1995b).

Thirty-four individual wetlands would be impacted by implementation of the proposed Sea-Tac Airport Master Plan Update improvements. The total area of wetland impact is 10.35 acres. These impacts would occur mostly during Phase I of plan implementation, which includes

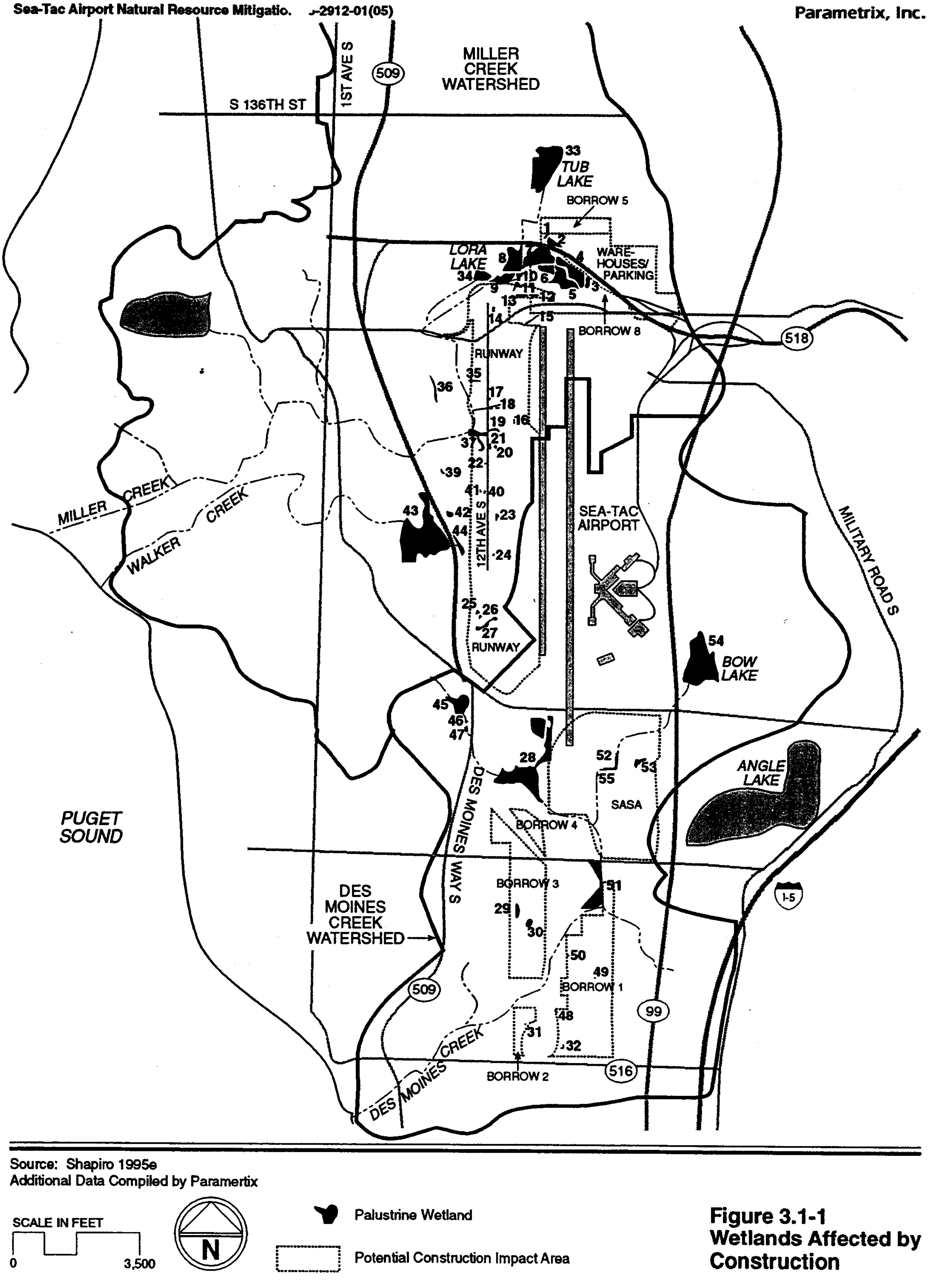

Figure 3.1-1
Wetlands Affected by Construction

Table 3.1-1. Classification, size, and impacts to wetlands in the proposed Sea-Tac Airport Master Plan Update study area.

Wetland Number	Classification [1]	Wetland Size (Acres)	Total Impact (Acres)	Vegetation Cover Type Impact (Acres)		
				FO	SS	EM
1	PFO	0.07	0.07	0.07		
2	PFO/EM (60/40)	0.74	0.74	0.44		0.29
3	PFO	0.56	0.56	0.56		
4	PFO	5.02	0.00			
5	PFO/SS	4.58	0.00			
6	PSS	0.87	0.00			
7	PFO/OW/EM	6.70	0.00			
8	PSS/EM	4.95	0.00			
9	PEM/FO (60/40)	2.85	0.13	0.05		0.08
10	PSS	0.31	0.00			
11	PFO/EM (80/20)	0.50	0.47	0.37		0.09
12	PEM/FO (80/20)	0.21	0.21	0.04		0.16
13	PEM	0.05	0.05			0.05
14	PFO	0.19	0.19	0.19		
15	PEM	0.28	0.28			0.28
16	PEM	0.06	0.06			0.06
17	PEM	0.03	0.03			0.03
18	PFO	0.12	0.12	0.12		
19	PFO	0.57	0.57	0.57		
20	PSS/EM (90/10)	0.06	0.06		0.06	0.01
21	PFO	0.22	0.22	0.22		
22	PEM/SS (90/10)	0.06	0.06		0.01	0.05
23	PEM	0.78	0.78			0.78
24	PEM	0.14	0.14			0.14
25	PFO	0.06	0.06	0.06		
26	PEM	0.02	0.02			0.02
27	PEM[2]	0.00	0.00			
28	POW/SS (O/100)	18.10	0.06		0.06	
29	PFO	0.74	0.74	0.74		
30	FO/PSS (80/20)	0.50	0.50	0.40	0.10	
31	PEM	0.05	0.00			
32	PEM	0.05	0.05			0.05
33	PFO/SS/EM/OW	17.60	0.00			
34	POW	1.40	0.00			
35	PEM	0.21	0.18			0.18
36	PFO/EM	0.30	0.00			
37	PFO/SS (70/30)	2.41	1.68	1.17		0.50
38	PEM/SS[3]	0.00	0.00			
39	PFO	0.07	0.00			
40	PFO	0.09	0.09	0.09		
41	PEM	0.08	0.08			0.08
42	PEM	0.50	0.00			
43	PEM/SS/FO/OW	30.30	0.00			
44	PFO/SS	0.70	0.00			
45	PEM	5.00	0.00			
46	POW	0.06	0.00			
47	POW	0.20	0.00			
48	PEM	0.02	0.00			
49	PSS	0.02	0.02		0.02	
50	PEM	0.03	0.03			0.03
51	PFO	8.10	0.48	0.48		
52	PFO/SS (90/10)	1.00	1.00	0.90	0.10	
53	PFO	0.60	0.60	0.60		
54	PSS/OW	25.70	0.00			
55	PSS	0.04	0.04		0.04	
TOTAL[4]		**143.86**	**10.35**	**7.08**	**0.39**	**2.88**

Source: Shapiro 1995b; additional data compiled by Parametrix

[1]Based on USFWS classification system (Cowardin et al. 1979). Where impacts would occur to more than one cover type, the percentages used for impact calculations are shown in parenthesis.

[2] Fill of this wetland has been approved by a Section 404 Nationwide 26 permit.

[3]This wetland was determined not to be a regulated wetland by the City of Sea-Tac and the Corps of Engineers.

[4]Values are rounded to two significant figures. Actual values differ slightly due to the effects of rounding.

construction of the new parallel runway, the Runway 34R extension, and RSA upgrades. Wetland mitigation is planned to compensate for all anticipated wetland impacts attributed to full implementation of the proposed Master Plan Update improvements.

3.2 ECOLOGICAL ASSESSMENT OF IMPACT SITE

Study area wetlands occur in two drainage basins (Des Moines Creek and Miller Creek basins) and in many cases are physically separated from other wetlands and upland habitats by urban and industrial development (refer to Figure 3.1-1). In addition to substantial fragmentation of wetland habitat, the small size of most impacted wetlands suggests that they may function independently rather than as an ecological system. However, many of the affected wetlands serve similar physical and biological functions and can be grouped for ecological assessment. The following sections discuss important ecological characteristics of wetlands within proposed impact areas. These characteristics have been incorporated into the mitigation design.

3.2.1 Existing Vegetation

Nineteen vegetation communities were identified in the proposed Master Plan Update study area, including 9 wetland and 10 upland vegetation communities (Landrum & Brown 1995). The 9 wetland vegetation communities may be further grouped into three vegetation cover types: (1) forested wetland; (2) shrub wetland; and (3) emergent wetlands. Vegetation in all wetlands and buffer areas shows characteristics of relatively recently disturbed plant communities, including a predominance of successional species, a young average age of canopy species (estimated from tree diameters), and evidence of past and ongoing human disturbance. These characteristics indicate that most wetlands support vegetation established within the past 25 to 50 years.

Tables 4 and 5 in Appendix H-A of the Sea-Tac Airport Master Plan Update EIS (Shapiro 1995b) list the wetland indicator status of observed plant species in wetlands and uplands in the Sea-Tac Master Plan Update study area. Plant communities in the study area are common to the region; no unique, threatened, or endangered species occur in the study area.

3.2.1.1 Forested Wetlands

Twenty-six wetlands in the Master Plan Update study area support a forested wetland vegetation class. Of these, 18 wetlands with a forested component would be impacted by implementation of the proposed Sea-Tac Airport Master Plan Update improvements. Impacts to forested wetland vegetation at individual sites, ranging in area from 0.04 acres to 1.17 acres, would affect 7.08 acres overall (Table 3.1-1).

Shapiro (1995b) characterized three types of forested wetland vegetation in the impact area: (1) red alder and salmonberry-dominated wetland; (2) willow-dominated wetland; and (3) mixed deciduous wetland.

Red alder- and salmonberry-dominated wetlands are most prevalent in wetlands associated with stream corridors, including wetlands 19 and 37 in the Miller Creek corridor and wetlands 51 and 52 in the Des Moines Creek corridor. Isolated wetlands supporting red alder swamp include wetlands 21 and 40 near the western edge of the proposed airport operations area (AOA), Wetland 29 in the south Borrow Area 3, and Wetland 53 in the SASA area. Big-leaf maple, western red cedar, Sitka willow, and black cottonwood occur as associated species in the overstory. Associated understory plants include Indian plum, blackberry species, and English ivy. The most common herbaceous species observed included horsetail, lady-fern, and reed canarygrass. Other herbaceous plants found in red alder swamps included stinging nettle, tall mannagrass, creeping buttercup, bittersweet nightshade, and Watson's willow-herb. A total of 4.78 acres of red alder-dominated swamp would be affected by the proposed Master Plan improvements.

The greatest willow-dominated wetland concentration occurs in the Lake Reba wetland complex. However, impacts to this vegetation type have been substantially reduced by eliminating use of Borrow Area 8, which encompasses most of the Lake Reba wetland complex. Willow-dominated wetlands that would be impacted include wetlands 3 and 9 in the area proposed for improvements to South 154th Street, Wetland 25 near the western edge of the proposed AOA, and Wetland 30 in the south Borrow Area 3. Sitka and Pacific willow dominate this vegetation community. Red alder, black cottonwood, and Scouler's willow are associated canopy species. The understory is dominated by willow shrubs. Herbaceous species that grow under the relatively thick canopy include tall mannagrass, small-fruited bulrush, common and giant horsetail, lady-fern, creeping buttercup, watercress, American speedwell, and soft rush. The total area affected in willow-dominated wetlands would be 1.07 acres.

Mixed deciduous wetlands occur throughout the study area. Wetlands supporting this vegetation type that would experience impacts include wetlands 1 and 2 in the proposed warehousing and parking area and wetlands 11, 12, 14, and 18 along the northern and western edges of the proposed AOA. The overstory consists of a mixture of hydrophytic trees such as red alder, black cottonwood, Pacific willow, Sitka willow, and western red cedar. The undergrowth varies considerably with the hydroperiod, the amount of sunlight received, and soils. Some of the most commonly observed shrubs include Himalayan blackberry, willow, salmonberry, red elderberry, and Douglas spirea. Herbaceous species found growing below the canopy included creeping buttercup, bentgrass, soft rush, lady-fern, swordfern, reed canarygrass, and common horsetail. the proposed Master Plan Update improvements would affect 1.23 acres of mixed deciduous wetland.

3.2.1.2 Shrub Wetlands

Seventeen wetlands in the Master Plan Update study area support a shrub wetland vegetation class. Of these, seven wetlands with a shrub component would be impacted by build-out of the proposed Sea-Tac Airport Master Plan Update improvements. Acreages of shrub wetland vegetation affected at individual sites would range from 0.01 to 0.10 acres (Table 3.1-1).

Shrub wetland vegetation occurs in the southern and western portions of the AOA. These previously cleared areas are presently revegetating with tree saplings. The dominant vegetation species are red alder, black cottonwood, and willow. Common herbaceous plants include velvet-grass, soft rush, bentgrass, and Watson's willow-herb.

Willow-dominated shrub wetland occurs mainly in the north borrow area where soils are saturated to the surface for most of the year. Pacific willow and Sitka willow share dominance in these areas. Common understory herbaceous species are the same as those described for the willow-dominated forest community.

Salmonberry-dominated wetland occurs in the north borrow area upslope of the willow-dominated depressions. Herbaceous species that occur in this community are similar to those in the red alder- and salmonberry-forested wetland community.

3.2.1.3 Emergent Marsh

Twenty-eight wetlands in the Master Plan Update study area support an emergent wetland vegetation class. Of these, 18 wetlands with an emergent component would be impacted by build-out of the proposed Sea-Tac Airport Master Plan Update improvements. Impacts to emergent wetland vegetation at individual sites would affect areas ranging in size from 0.01 to 0.78 acres (Table 3.1-1).

Monotypic stands of reed canarygrass are located throughout the study area. These wetlands are often bordered by stands of Himalayan blackberry or forested wetland. Species found in association with the reed canarygrass stands include Canadian thistle, black mustard, bentgrass, cattail, and stinging nettle.

Two large stands of cattail occur on the site; one is located between Lake Reba and Lora Lake. The other stand is north of Tyee Valley Golf Course at the south end of the runways. The stand in the north borrow area is bordered on one side by a service road and on the remaining sides by reed canarygrass. Miller Creek provides water to this community year-round. Associated species include reed canarygrass, soft rush, and bittersweet nightshade. The community of cattail in the southern portion of the site has common reedgrass, soft rush, Watson's willow-herb, and reed canarygrass as associated species.

Mixed grass and forb emergent marsh occurs on the airfield in the AOA, in several depressions with compact soils, and in association with several hillside seeps. These areas are characterized by a mixture of hydrophytic forbs such as soft rush, toad rush, cudweed, Watson's willow-herb, common and giant horsetail, common cattail, and an array of hydrophytic grasses that include common velvet-grass, bentgrass, reed canarygrass, and foxtail.

3.2.2 Existing Water Regime

Wetlands in the Master Plan Update study area are associated with a variety of hydrologic features, including Lake Reba, Miller and Des Moines creeks, hillside seeps, roadside ditches, and numerous seasonally saturated to permanently flooded depressions. During field studies in the summer and fall of 1994 (Shapiro 1995b), observed on-site hydrology changed dramatically at the transition to the winter rainy season. Many areas that were dry to 30 inches below the ground surface during late summer had observable wetland hydrology during the latter part of the growing season after fall rains began. During the December 1994 field visits, recent storms had flooded several wetlands.

Wetlands that would be impacted by the proposed Master Plan Update improvements can be divided into two general impact categories, based on site hydrology: (1) wetlands with seasonal hydrology; and (2) wetlands with a year-around source of hydrology. Most wetlands that would be affected are associated with seeps and depressions having a fluctuating hydrologic regime influenced by seasonal rainfall. Acreage impacts to wetlands with seasonal hydrology would total 5.8 acres. Impacted wetlands with a year-around water source are associated with Lora Lake (wetlands 9 and 11), Miller Creek (wetlands 18, 19 and 37) and Des Moines Creek (wetlands 28, 51, 52 and 55). Impacts to these wetlands would total approximately 4.6 acres.

3.2.3 Existing Soils

Soils in the proposed Master Plan Update study area were characterized based on the soil survey of King County (USDA 1973) and field observation (Shapiro 1995b). The Soil Conservation Service (SCS) soil survey identifies soils only in the southernmost portion of the study area south of South 192nd Street. Because SCS typically does not map soils in urban areas, all of the study area north of South 192nd Street is unmapped. Soils in the unmapped area were, however, evaluated for their consistency with SCS-mapped soil series in the general vicinity and for hydric characteristics (Shapiro 1995b). SCS identifies six different soil series in the area, including: (1) Alderwood gravelly sandy loam; (2) Arents, Alderwood material; (3) Bellingham silt loam; (4) Everett gravelly sandy loam; (5) Indianola loamy fine sand; and (6) Norma sandy loam. Only the Bellingham and Norma series soils are identified as hydric (USDA 1991); however, hydric soil inclusions are relatively common in the non-hydric soil series occurring in the project area. An earlier soils survey of the study area (USDA 1952) identified Alderwood series soils as the predominant soil type in the region. This series typically has inclusions of hydric soils (Norma, Bellingham, Seattle, Tukwila, and Shalcar soils).

Shapiro (1995b) distinguished six basic soil types in the study area. Four of these six were determined to be hydric because of soil characteristics indicating saturated conditions, including mottles in a low-chroma matrix and gleyed color formation.

The most common upland soil observed by Shapiro in the study area is generally a brown (10YR 3/3) loam over light brown (10YR 4/3) sandy loam. These soils often are gravelly and appear to be fill material; they most closely match the SCS description of Arents, Alderwood material. Very dark grayish brown (10YR 3/2) loam soils without mottles were also encountered in the northern portions of the study area. These two soil types were not considered to be hydric because soil colors indicate they lack high water tables.

The most common hydric soils observed in the study area are very dark brown and black (10YR 3/2 and 10YR 2/1) loams and sandy loams overlying grayish brown (2.5Y 5/2) sandy loams and gravelly sandy loams. These soils typically have medium and coarse, strong brown (7.5YR 4/6), distinct and prominent mottles in the subsurface horizons. Soils matching this general description were observed in wetlands 2, 7, 11, 12, 16, 20, 22, 23, 31, and 39.

Shapiro (1995b) determined that the very dark grayish brown (10YR 3/2) to black (10YR 2/0) loam soils found throughout the northern portions of the study area were hydric where aquic soil moisture regimes and low-matrix chromas were encountered. Wetlands 6 and 29 contained this soil type.

Saturated, dark greenish gray (5G 4/1) sands were observed in wetlands in the northern borrow area and along the western study area boundary. Because these soils, observed in wetlands 3, 4, and 18, exhibit low-matrix chromas and an aquic moisture regime, they are considered hydric.

Throughout the AOA Shapiro (1995b) found dark brown (10YR 2/2) loams overlying grayish brown and dark grayish brown (10YR 5/1 and 2.5Y 5/2) silt loams, often with prominent mottles. These soils are considered hydric because they exhibit low-matrix chromas and mottles. Wetlands 1, 14, 15, and 21 contain soils matching this general description.

In the study area, Shapiro (1995b) observed two organic soils. The first generally has 6 to 8 inches of black (10YR 2/1) loam over highly decomposed muck. This soil was seen in wetlands 5 and 6. The second is generally a muck or mucky peat soil overlying gleyed mineral soils, as was observed in wetlands 5, 13 and 30. Wetland 5 also included areas of interbedded peat and mineral soil horizons. Soils with high organic contents are considered to be hydric.

3.2.4 Existing Fauna

The impacted wetlands within the study area support forested, palustrine, and emergent vegetation (Cowardin et al. 1979). Wildlife habitat in wetlands within the study area is fragmented by residential and commercial development which limits access to most large mammal, and many waterfowl, species (Landrum & Brown 1995). Water quality in these wetlands may be marginalized by developed buffers and surface contaminants. Amphibians are sensitive to water quality (Richter 1995) and some species that normally use wetlands similar to those being impacted may be absent due to degraded water quality. Faunal diversity in the study area is further limited since most of the impacted wetlands are too small to meet minimum habitat requirements for viable wildlife populations. There are, however, human-tolerant species using the study area.

Impacts to forested wetlands within the study area would generally be small, ranging from 0.04 to 1.17 acres, and most affected wetlands lack significant surface water features. These wetlands may be used by small passerine birds (such as varied thrushes, orange-crowned warblers, black-capped chickadees, and fox sparrows) for nesting and feeding (Ehrlich et al. 1988), small mammals (including mountain beaver, raccoon, opossum, Douglas squirrel, and deer mouse) for breeding and cover, and some amphibians (including northwestern salamander, Pacific chorus frog, and rough-skinned newt) for resting, foraging, and breeding (Nussbaum et al. 1983).

Shrub wetlands offer nesting and cover habitat for songbirds (such as Swainson's thrush, Bewick's wren, and kinglets) and small mammals (including the water shrew and Norway rat). Ponded areas in shrub wetlands are valuable for amphibian breeding, because they offer small vegetative stem structure, emerging from surface waters, that is suitable for egg mass attachment. However, without an associated forested component, shrub wetlands lack the woody debris which is desirable to terrestrial amphibians such as ensatina. The potentially impacted shrub wetlands in the study area are small (<1.0 acre) and isolated from other natural areas, and this limits their habitat value.

Emergent wetlands provide habitat to songbird species, which use the vegetation for nesting and foraging (such as red-winged blackbirds, marsh wrens), small mammals (such as muskrat and water shrew) that forage on the vegetation and invertebrates, and amphibian species (including long-toed salamander, Western toad, and Pacific treefrog) that need vertical stems in standing water for egg mass attachment. Many of the potentially impacted emergent wetlands in the study area are small, isolated, and highly disturbed. Wetlands located within the current AOA, are maintained in a disturbed state which limits their value as wildlife habitat (Landrum & Brown 1995). Most emergent wetlands have intermittent surface flows or seasonal standing water which also limits their overall habitat value.

3.2.5 Wetland Functions and Values

The biological and physical functions of wetlands within the study area were assessed to identify important qualities that should be replicated by the mitigation design.

Functional assessment methodologies for wetlands typically identify and evaluate physical attributes that provide predictive rather than direct measurements of specific ecological functions of interest (Reimold 1994). The limitations of many of the available functional analysis methods make expert opinion critical when assessing wetland functions and values.

Several assessment methodologies are available to determine wetland functions; these include the *Wetland Evaluation Technique* (WET) (Adamus 1991) and the *Wetland Assessment Techniques* (Reppert et al. 1979). These assessment procedures are, however, frequently used to predict wetland functions over broad geographical areas such as entire drainage basins. These methodologies typically do not recognize local variations in small wetlands on a scale such as the proposed Master Plan Update study area. The methods emphasize the importance of waterfowl and flood control functions of wetlands, but they typically do not differentiate the functional value of smaller wetlands that lack aquatic habitat similar to many wetlands within the Master Plan Update study area. Because of the diversity of wetland systems nationwide, general functional assessment procedures often do not recognize regional variations in wetlands with similar physical attributes. To address this gap in assessment methodologies, Hruby et al. (1995) developed a numeric assessment methodology (Indicator Value Assessment or IVA) that establishes relative functional values for wetlands within a limited geographic region. This system is based on professional opinion and the numeric evaluation of physical attributes observed in the field.

For this project, a combined approach was used to assess wetland functions by determining the presence of recognized indicators of biological and physical functions (Hruby 1995; Adamus 1991; and Reppert 1979) and by using professional judgement to evaluate the overall significance of these indicators to the function being considered. This assessment evaluated field indicators of habitat quality for fish and wildlife (Table 3.2-1) and indicators of hydrologic and water quality functions (Table 3.2-2). Field evaluations of wetlands were completed during August and September 1995. Wetlands in the study area were evaluated by recording the presence or absence of these field indicators within and adjacent to each wetland.

Because of the small size of most wetlands and their frequent lack of hydrologic connectivity, and because of the relatively similar vegetation types within wetland classes, most wetlands functions were evaluated for groups of wetlands with similar vegetation cover. Tables 3.2-3 and 3.2-4 summarize the physical and biological functions of forested and emergent wetlands in the study area. Total acreage impacts to shrub wetlands would be low, and most shrub wetlands are associated with other vegetation types. Functional impacts to shrub wetlands would be similar to the assessment for forested and emergent wetlands.

Table 3.2-1. Wetland attributes considered in evaluating biological functions of wetlands impacted by the Master Plan Update improvements.

	Function					
Wetland Attribute	Anadromous Fish	Resident Fish	Passerine Birds	Herptiles	Small Mammals	Waterfowl
Wetland cultivated or drained	X	X	X	X	X	X
Wetland ditched or drained				X		
Amount of impervious surface in wetland or watershed	X	X		X	X	
Amount of buffer in crops or pasture				X	X	
Amount of buffer in forest or shrub vegetation			X	X	X	X
Connection of wetland to other natural areas				X	X	X
Size of wetland			X	X	X	X
Number of Cowardin vegetation classes: 3			X		X	X
Vegetation interspersion			X	X	X	X
Amount of forested wetland			X	X	X	X
Evidence of seasonal ponding in forest vegetation classes			X	X		X
Areas of aquatic bed vegetation			X	X		X
Areas of emergent vegetation			X	X	X	X
Presence of invasive emergent vegetation			X	X	X	X
Amount and diversity of shrub wetland			X		X	
Presence of seeps and springs	X	X				
Wetland contains a seasonal/permanent channel/stream/ditch	X	X	X	X	X	X
Documented evidence of use by fish (within 3 yrs.)	X	X	X	X		
Channel/stream sinuous	X	X		X		
Stream velocities and indicators of erosion	X	X				
Pools and riffles present	X	X	X		X	X
Spawning gravels are present	X	X				
Presence of undercut banks	X	X				
Interspersion of water and emergent vegetation			X	X	X	X
Stream channel shores or OW overhanging vegetation	X	X	X		X	
Adjacent vegetation is deciduous	X	X			X	
Frequency and amount of flooding in wetland	X	X	X			X
Part of wetland is flooded at least once per year			X			
Depth and area of seasonal open water			X	X	X	X
Depth and area of permanent open water		X	X	X	X	X
Areas seasonally ponded, emergent vegetation			X	X	X	X
Perch sites adjacent to or above water			X			X
Conifers forest present with large woody debris	X	X	X	X	X	
Log, stump, or snag is >35" diameter within wetland			X		X	
Nature and amount of woody debris in stream or flooded portions of wetlands	X	X		X	X	
Hummocks/islands present in wetland			X	X	X	X
Upland/wetland edge irregular (W:L ratio >2:1)			X	X	X	X
Evidence of impacts from excess nutrients, toxic materials, or sediments	X	X		X		

Data compiled by Parametrix

Table 3.2-2. **Wetland attributes considered in evaluating physical functions of wetlands impacted by the proposed Master Plan Update improvements.**

	Function							
Wetland Attribute	Baseflow Support	Export of Production	Floodflow Desynchronization	Flood Storage	Surface Runoff Storage	Nutrient Retention/Transformation	Retention of Toxics	Sediment Trapping
Wetland ditched or drained	X	X	X	X	X	X	X	X
Wetland in pasture or cultivation		X				X	X	X
Wetland contains a seasonal/permanent channel/stream/ditch		X		X	X			
Multiple channels within wetland		X				X	X	X
Wetland on slope discharging to stream	X							
Amount and type of human activities in upstream watershed						X	X	
Manmade structures hold back water					X			
Evidence of beaver dams		X		X	X	X	X	X
Wetland has no inlet and no outlet					X	X	X	X
Wetland has outlet but no inlet	X							
Outflow present during summer but no inlet	X							
Topog[illegible]y of wetland relative to outlet			X	X	X	X	X	X
Amou[illegible]looding within the wetland		X	X	X	X			
Wet[illegible] fluctuating water levels throughout year			X	X	X	X	X	X
Am[illegible]egetation present in flooded portions of wet[illegible]						X	X	X
Dir[illegible]ence of sediment trapping						X	X	X
Presen[illegible] of organic soils	X				X	X	X	
Underlying soil a clay, till, or hardpan	X							
Interspersion of vegetation and open water areas		X				X		
Water depths			X	X	X	X	X	X

Data compiled by Parametrix

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Table 3.2-3. Functional assessment of emergent wetlands impacted by the proposed Master Plan Update improvements.

Function	Rating	Rationale
Resident and Anadromous Fish	Low Habitat Value	Most emergent wetlands (including wetlands 15, 16, 17, 23, 24, 32) are isolated from streams and other fish habitat; they also have only small amounts of standing water during winter and spring months, and are thus unable to provide this habitat function. Two emergent wetlands (wetlands 12 and 35) have intermittent connections to Miller Creek by artificial ditches; however, these connections provide no significant habitat to fish nor do they allow fish to access suitable habitat within emergent wetlands.
Passerine Bird Habitat Birds	Low to Moderate Habitat Value	Emergent wetlands are small and range in size from less than 0.1 acre to about 0.5 acre. Due to their small size and location in shallow, seasonally wet depressions, these wetlands lack many of the habitat attributes associated with high wildlife value for breeding birds. Emergent wetlands lack significant open water or standing water during the breeding season. They are often vegetated by reed canarygrass, velvet grass, or other emergent plant species with limited value to birds such as the marsh wren, red-winged blackbird, song sparrows, and common yellowthroat. Emergent wetlands that have associated forest or shrub wetland classes and buffers (wetlands 2, 12, 20, 22) provide habitat for a greater variety of breeding birds, including thrushes and flycatchers. In these wetlands, red alder and cottonwood provide nest and perch sites for birds, although trees are often too young to provide cavity nesting habitat.
Herptiles	Low to Moderate Habitat Value	Soil in emergent wetlands are typically saturated and lack significant amounts of standing water during the late spring and summer months. This condition limits the diversity of amphibians that can breed in the wetlands. Where extended seasonal ponding in ditches (wetlands 12, 35) is present, Pacific chorus frog, red-legged frog, and long-toed salamander may be present.
Small Mammals	Low to Moderate Habitat Value	Emergent wetlands, especially when bordered by forest or shrub communities (wetlands 36, 37, 39) are likely to provide habitat to small mammals. However, the small size of most of these wetlands, and the lack of diverse habitat structure within them, limits the value of these wetlands to small mammals.
Waterfowl	Low Habitat Value	Emergent wetlands affected by the proposed airport improvements lack significant open water or meadow areas that serve as foraging, nesting or resting areas for overwintering waterfowl.
Export of Production	Low	Due to the small size of the wetlands and the lack of permanent hydrologic connections to downstream systems, the wetlands do no provide significant export functions. Minor export could occur from wetlands during periods when flow in ditches (wetlands 12, 35) is present.
Baseflow Support	Low	Emergent wetlands generally occur in small, shallow depressions that seasonally collect storm runoff or intercept a seasonally high groundwater table. For most emergent wetlands, except during storms, there is little water movement through them and little evidence that they contribute to baseflow. Emergent wetlands 12, 17, and 35 provide some baseflow support to Miller Creek since they occur where the groundwater surfaces through seeps and springs. This flow is eventually conveyed to the creek.

Table 3.2-3. Functional assessment of emergent wetlands impacted by the proposed Master Plan Update (continued).

Function	Rating	Rationale
Flood Control	Low	Since the impacted emergent wetlands are generally not associated with floodplains, they do not provide flood control functions. Wetland 1 is associated with Miller Creek and may be within the 100-year floodplain of this creek. The wetland could provide flood control functions.
Surface Runoff Storage	Moderate	Emergent wetlands occurring in shallow depressions that seasonally collect storm runoff may reduce runoff rates during storm events. Wetland 1 is associated with Miller Creek and may provide storm water management functions.
Nutrient Retention/ Transformation	Moderate	Emergent wetlands located in shallow depressions, along ditches or other seasonal drainageways (wetlands 12, 35) provide biofiltration functions that likely result in water quality improvement, including nutrient retention, nutrient transformations, sedimentation, and removal of some chemical pollutants.

Data compiled by Parametrix

Table 3.2-4. **Functional assessment of forested wetlands impacted by the proposed Master Plan Update.**

Function	Rating	Rationale
Resident and Anadromous Fish	Low to Moderate Habitat Value	Most forested wetlands (wetlands 11, 14, 21, 25, 39, 40, 53) are isolated from streams and other fish habitat. They also contain only small amounts of standing water during winter and spring months and are thus unable to provide this function. Some forested wetlands (wetlands 18, 19, 52) have intermittent connections or border Miller or Des Moines creeks. These wetlands provide shade, buffer, and food resources to resident or migratory fish species.
Breeding Birds	Moderate to High Habitat Value	Several larger forested wetlands (wetlands 3, 19, 29, 37, 51, 52, 53) contain many attributes associated with high habitat value for breeding birds. These attributes include association with more than one wetland class, the presence of snags or logs, understory shrub and herbaceous vegetation, and forested buffers connecting to other habitat types. Forested wetlands generally lack significant open water or standing water during the breeding season. Forested wetlands are often dominated by willow, alder, and cottonwood (wetlands 11, 19, 37, 39, 53) that are too young to provide cavity nesting habitat. The lack of dense coniferous forest habitat in all forested wetlands also reduces their habitat value for some bird species.
Herptiles	Low to Moderate Habitat Value	Soil in forested wetlands is typically saturated and lacks significant amounts of standing water during the late spring and summer months. This condition limits the diversity of amphibians that can breed in the wetlands. Where extended seasonal ponding is present (wetlands 3, 37, 40), Pacific chorus frog, red-Legged frog, and long-toed salamander may be present.
Small Mammals	Moderate Habitat Value	Forested wetlands provide habitat to small mammals such as raccoon, opossum, squirrels, mice, and rats. The wetlands typically do not support burrowing animals due to seasonally saturated soils. Large mammals are generally absent from the project area due to urban development.
Waterfowl	Low Habitat Value	Forested wetlands lack significant open water or open areas to allow foraging, nesting, or resting by waterfowl.
Export of Production	Low Value	Forested wetlands located along Miller and Des Moines creeks (wetlands 31, 37, 51, 52) provide export functions, as detritus and insect production may be transported from the system stream flow. Other forested wetlands not hydrologically connected to stream systems do not provide export functions.
Baseflow Support	Low to Moderate Value	Forested wetlands along Miller and Des Moines creeks (wetlands 3, 21, 18, 19, 37, 39, 52) occur in areas of groundwater discharge and contribute to the base flow. Other forested wetlands occur in shallow depressions that seasonally collect storm runoff or that intercept a seasonally high groundwater table. There is little water movement through these wetlands during storms, and they do not appear to contribute to baseflow.
Flood Control	Low to Moderate Value	Since the impacted forested wetlands are not generally associated with floodplains, they do not provide flood control functions. Wetlands associated with Miller Creek (wetlands 3, 11) are within the 100-year floodplain and provide flood control functions.

Table 3.2-4. Functional assessment of forested wetlands impacted by the proposed Master Plan Update (continued).

Function	Rating	Rationale
Surface Runoff Storage	Low Value	No forested wetlands occur in shallow depressions that seasonally collect storm runoff; thus, the surface water storage function of these wetlands is low. Wetlands 3 and 11 are associated with Miller Creek and may provide limited storm water management functions.
Nutrient Retention/Transformation	Low to Moderate Value	Forested wetlands 3 and 11 may provide biofiltration functions that likely result in water quality improvement (including contaminant retention and transformations of nutrients and chemicals). Other forested wetlands are not likely to provide significant water quality functions due to their lack of connections to surface water.

Data compiled by Parametrix

The geohydrology of the Master Plan area is discussed in Chapter IV-4 of the FEIS. The groundwater recharge/discharge functions of wetlands was not specifically addressed in this analysis. However, based on interpretation of the landscape position of wetlands, the function of wetlands relative to groundwater movement can be inferred.

Groundwater discharge/recharge functions of wetlands appear to be variable throughout the site. For wetlands occurring on till soils above the Miller Creek and Des Moines Creek ravines, wetlands appear to form in localized depression where perched soils develop on low permeability till. Due to the low permeability of the till layer, it is unlikely these small wetlands contribute to recharge of groundwater. Wetlands located in the ravines associated with Miller and Des Moines Creek typically have intermittent or perennial seeps and springs that indicate groundwater discharge.

Since many of the wetlands that would be impacted by the proposed Master Plan improvements are small (< 0.5 acre); isolated from other significant aquatic or semi-aquatic habitat; and occur in a landscape fragmented by streets, commercial, residential, or airport development; their wildlife habitat functions are generally significant only to the local vicinity (rather than to a larger landscape or watershed) (Brinson 1993). However, hydrologic functions (such as flood storage, groundwater discharge, and storm water detention) are potentially important at the watershed level, because, when present, they help maintain fish habitat in Miller and Des Moines creeks.

3.3 WETLAND MITIGATION GOALS, OBJECTIVES AND PERFORMANCE STANDARDS

3.3.1 Goals and Objectives

Specific mitigation goals for unavoidable wetland impacts were developed to meet the federal standard of no net loss of wetland functions and area, while still recognizing the unique mitigation siting requirements imposed by FAA airport operation safety guidelines. An FAA draft Advisory Circular 150/5200, which is expected to be in effect in the near future, would require that careful consideration be given to preventing wildlife attractions. While it is preferable for wetland mitigation to be sited near the impact (to provide replacement habitat for displaced wildlife) and within the same drainage basin (to replace lost physical functions), the FAA draft Advisory Circular strongly contradicts this option. Due to topography and extensive development, there are no appropriate lands of sufficient size for wetland habitat mitigation within the Miller Creek and Des Moines Creek drainage basins and outside the 10,000-ft safety radius. Siting wetland mitigation within the Miller Creek or Des Moines Creek basins would require acquisition of additional land currently developed for residential and business uses. Therefore, mitigation for impacts to wildlife habitat would be located outside the basins. Mitigation for impacts to physical functions of wetlands, such as storm water storage and floodwater attenuation could be achieved without creating attractive habitat for wildlife and will be located within the Miller Creek and Des Moines Creek basins. Specific mitigation measures for these impacts are discussed in Sections 4 and 5 of this mitigation plan.

The rationale for selecting an off-site and out-of-basin wetland mitigation area is included in Section 4.3—Rationale for Site Selection. Based on federal policies regarding no net loss and airport safety, the following specific wetland mitigation goals have been developed for the proposed Sea-Tac Airport Master Plan Update improvements:

Wetland Goal 1: Achieve no overall net loss of wetland acreage by establishing a diverse, in-kind replacement habitat with forested, shrub, and emergent wetland classes.

Wetland Goal 2: Provide in-kind wildlife habitat replacement outside the 10,000-ft aircraft operations safety radius.

Wetland Goal 3: Facilitate an increase in overall habitat functions.

3.3.2 Design Objectives, Design Criteria, and Final Performance Standards

Impacts of the proposed Master Plan Update improvements to wetland functions, design objectives to compensate for these impacts, and the compensation ratios that would be achieved by the wetland mitigation plan described in this chapter are summarized in Table 3.3-1. To achieve the wetland mitigation goals, specific design features will be implemented on a 47-acre site located 7.2 miles from Sea-Tac Airport in the city of Auburn. The off-site mitigation location would satisfy the unique wildlife habitat siting requirements associated with airport development by providing replacement habitats outside the recommended 10,000-ft safety radius identified in FAA draft Advisory Circular 150/5200. The size of the mitigation site would allow for development of an aggregate of habitat types that would provide greater overall habitat values than the collection of small, discontinuous wetlands that would be filled. Long-term site protection would be enhanced by allowing for consolidated management, monitoring, and contingency planning. Specific design objectives, design criteria, and final performance standards are identified for each wetland mitigation goal in Table 3.3-2. These performance standards are the basis for the monitoring program discussed in Section 3.6.

3.4 PROPOSED MITIGATION SITE

3.4.1 Site Description

The 47-acre mitigation site is part of a 69-acre parcel located within the City of Auburn immediately west of the Green River (Figure 3.4-1). The undeveloped parcel has been farmed in the recent past and currently supports a mix of upland pasture grasses and forbs that are common to abandoned agricultural land in the Puget Sound basin. Approximately 4.3 acres of reed canarygrass-dominated wetland was delineated during previous site investigations (David Evans and Associates 1995) and is included in the 47-acre portion of the site proposed for mitigation. The remaining 22 acres would be designated as a reserve area for future

Table 3.3-1. **Summary of wetland impacts and compensatory design objectives for the proposed Seattle-Tacoma International Airport Master Plan Update.**

Project Impact	Compensatory Design Objectives	Acreage Provided	Compensation Ratio
Fill of 7.08 acres of forested wetland and loss of associated wildlife habitat.	Provide in-kind replacement of forested wetland vegetation cover and increase overall wildlife habitat value.	20.87 acres of forested wetland	minimum 2:1 maximum 2.95:1
Fill of 0.39 acre of shrub wetland and loss of associated wildlife habitat.	Provide in-kind replacement of shrub wetland vegetation cover and increase overall wildlife habitat value.	1.02 acres of shrub wetland	minimum 2:1 maximum 2.62:1
Fill of 2.88 acres of emergent wetland and loss of associated wildlife habitat.	Provide in-kind replacement of emergent wetland vegetation cover and increase wildlife habitat value.	5.43 acres of emergent wetland	minimum 1.5:1 maximum 1.89:1
Loss of some surface water treatment.	On-site replacement of surface water functions will be addressed in the final design of the proposed Master Plan Update components.	NA	NA
	Additional mitigation to provide flood storage capacity in the Green River drainage basin.	Approximately 30 to 60 acre-ft of flood storage capacity	NA
Loss of degraded wetland buffers.	In-kind replacement for upland buffer impacts and additional mitigation for wildlife using both wetland and non-wetland habitats.	Approximately 3 acres of forested upland buffer	NA

Data compiled by Parametrix
NA - Not applicable

Table 3.3-2. Mitigation goals with associated design objectives and final performance standards (continued).

Design Objectives	Design Criteria[1]	Final Performance Standards[2]
Wetland Mitigation Goal 2: Provide in-kind wildlife habitat replacement outside the 10,000-ft aircraft operations safety radius.		
Provide out-of-kind flooded emergent wetland habitat suitable for waterfowl feeding and resting during the winter and spring months.	Emergent wetlands will satisfy the design criteria for Wetland Mitigation Goal 1. Additional design criteria for waterfowl habitat include:	Emergent wetlands will satisfy the final performance standards identified for Wetland Mitigation Goal 1. Additional performance standards for waterfowl habitat include:
	Provide year-round shallow water (7-12 inches deep) with patches of emergent vegetation as feeding habitat for dabbling species.	Permanently flooded emergent wetlands will have shallow-water habitat (<12 inches deep) near the edges, with emergent vegetation and bottom detritus interspersed throughout.
	Provide ponded water with an emergent edge for water resting habitat.	Ponded water at least 6 inches deep will occur in open areas of at least 1 acre with low surrounding vegetation (<24 inches tall and covering an area at least 35 ft wide) between mid-September and April.
	Provide adequate protection for waterfowl habitat by minimizing adjacent cover for predators.	Evidence of waterfowl use (nesting, breeding, staging, or foraging activities) will be present.
Provide in-kind emergent, shrub, and forested wetland habitat with feeding and breeding for songbirds.	Forested, shrub, and emergent wetlands will satisfy the design criteria for Wetland Mitigation Goal 1. Additional design criteria for songbird habitat include:	Forested, shrub, and emergent wetlands will satisfy the final performance standards identified for Wetland Mitigation Goal 1. Additional final performance standards for songbird habitat include:
	Plant forested wetland adjacent to shrub, emergent, and standing-water habitats.	Perch sites in the forested canopy will overhang emergent wetland areas.
	Plant portions of the forested wetland with shrub understory species to provide a multiple-layered canopy adjacent to the shrub portion of the wetland.	Forested wetlands will have a shrub understory of 400 stems per acre over 25% of the area.
		Evidence of songbird nesting (nest, breeding territories, or observations of breeding behavior) will be present.
Provide in-kind forested, shrub, and emergent wetland feeding and breeding habitat for small mammals.	Forested, shrub, and emergent wetlands will satisfy the design criteria identified for Wetland Mitigation Goal 1. Additional design criteria for small mammal habitat include:	Forested, shrub, and emergent wetlands will satisfy the final performance standards identified for Wetland Mitigation Goal 1. Additional final performance standards for small mammal habitat include:
	Large woody debris (stumps and logs of native species) placed throughout the forested wetland at densities of 70 pieces per acre (approximately 25 ft on-center) to provide year-round cover for small mammals.	Evidence of small mammal use (nests, feeding signs, observations) will be present.
	Low hummocks (with a minimum area of 150 ft^2 at elevation 43 ft) constructed in the shrub wetland areas to provide non-saturated soils for burrowing small mammals.	Shrub hummocks will have a minimum of 12 inches of non-saturated soil above the 42-ft winter ponding elevation and cover at least 10% of the shrub zone.
Provide in-kind breeding habitat for amphibians.	Forested, shrub, and emergent wetlands will satisfy the design criteria for Wetland Mitigation Goal 1. Additional design criteria for amphibian habitat include:	Forested, shrub, and emergent wetlands will satisfy the final performance standards for Wetland Mitigation Goal 1. Additional final performance standards for amphibian habitat include:
	Provide soil saturation in forested wetlands within approximately 12 inches of the soil surface from late December to April.	Soils in forested wetland areas will be saturated within 12 inches of the soil surface from late December to April.
	Provide attachment substrate for breeding amphibian species consisting of emergent vegetation with stem diameters <0.25 inches in ponded water.	Leaf litter and vegetative debris will be present to provide habitat for invertebrates.
		Invertebrates will be present in the ground litter.
		At least 50% of live and dead stems in ponded emergent areas will be species with stem diameters less than 0.25 inches.
		Evidence of amphibian breeding (egg masses; larval stages) will be present.
Wetland Mitigation Goal 3: Facilitate an increase in overall habitat functions.		
Consolidate mitigation for impacts to many small, discontinuous wetlands into a single, larger wetland to provide a more diverse aggregate of habitat types.	Construct a contiguous wetland system with forested, shrub, and emergent wetland types and wildlife habitat features that provide in-kind and out-of-kind habitat replacement.	The mitigation wetland will satisfy the final performance standards identified for Wetland Mitigation Goals 1 and 2.
Assure long-term protection of the mitigation site(s).	Screen the north and south perimeters of the wetland from off-site development with a 50-ft-wide forested and shrub buffer.	Forested buffers will satisfy the final performance standards identified for forested wetlands for Wetland Mitigation Goal 1.
	Locate trails a minimum of 50 ft from emergent wetlands and provide shrub and forest vegetation as screening between trails and emergent wetlands.	All permanently and seasonally flooded wetlands will be screened from on-site trails by a minimum 50-ft-wide buffer of forest and shrub vegetation.
	Provide permanent interpretive and notice signs along the perimeter of the mitigation area describing natural features and restrictions related to use of the wetland mitigation area.	Interpretive signs will be located at 500-ft intervals around the perimeter of the mitigation wetland.

Data compiled by Parametrix

1 The rationale for design criteria is explained in Section 3.5, Mitigation Site Plan.

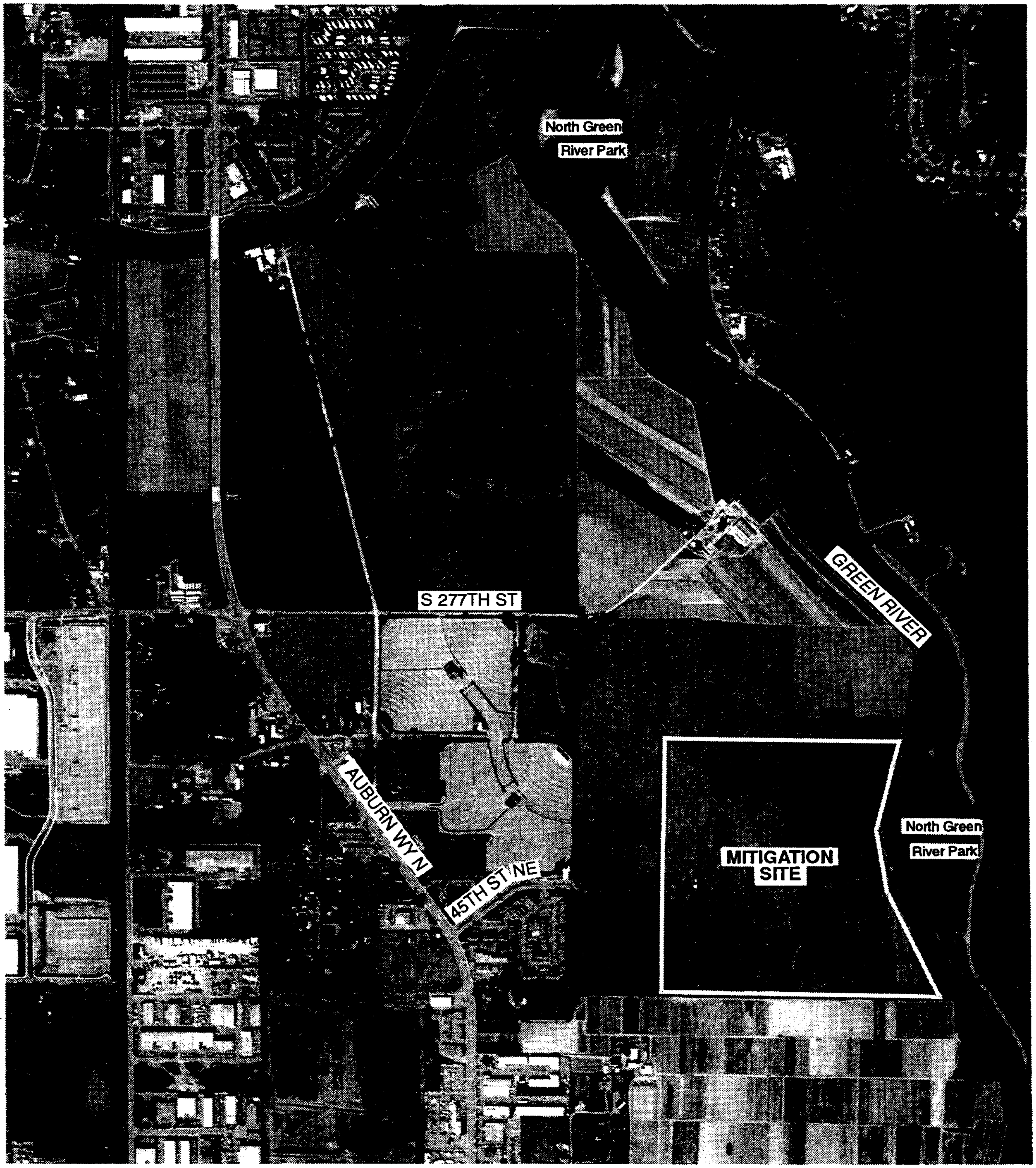
2 Condition required at the end of the 10-yr monitoring program. Interim performance standards are included in the Contingency Plan, Section 3.7.

3 Saturated soil is defined as the zone below the water surface in a hole or monitoring well.

4 All references to depths of flooding or depth to saturated soil refer to depths anticipated during years of normal rainfall (rainfall statistically similar [$p > .10$] to the long-term average).

Table 3.3-2. Mitigation goals with associated design objectives, design criteria, and final performance standards.

Design Objectives	Design Criteria[1]	Final Performance Standards[2]
Wetland Mitigation Goal 1: Achieve no overall net loss of wetland acreage by establishing a diverse, in-kind replacement habitat with forested, shrub, and emergent wetland classes.		
Provide seasonal to permanent wetland hydrology appropriate for each wetland vegetation cover type.	Create a perched water table by constructing a low-permeability soil layer overlain by topsoils with final grades of: 40.5 ft - 41.5 ft in emergent wetlands 41.5 ft - 42.0 ft in shrub wetlands 42.0 ft - 43.0 ft in forested wetlands	In forested areas, soils will be saturated[3] to approximately 12 inches of the surface (or less[4]) from late December through April in years of normal rainfall. In shrub areas, soils will be saturated at approximately the 6- to 12-inch depth year-round in normal rainfall years. Soils will be flooded with approximately 6 inches of water between December and late May. In emergent zones, soils will be saturated near the soil surface during normal rainfall years, and they will be flooded permanently where soil elevations are below 41 ft. Above 41 ft, flooding up to 24 inches deep will occur from late November through June.
Provide in-kind replacement for impacts to 7.08 acres of native forested wetland.	Plant five forested wetland plant associations that are similar in composition to naturally occurring plant associations. Use native deciduous and evergreen species such as black cottonwood, Oregon ash, red alder, western red cedar, and Sitka spruce. Plant a native shrub understory in 70% of the forested wetland area. Use native species such as salmonberry, twinberry, red-osier dogwood, red elderberry, willows, and vine maple. Plant native tree species at densities of at least 250 trees per acre (approximately 13 ft on-center). Plant native shrub species at densities of at least 500 plants per acre (approximately 9 ft on-center).	Forested wetlands will achieve a minimum in-kind replacement ratio of 2:1 by covering at least 14.6 acres of the mitigation site. Native wetland tree species will contribute at least 80% of tree density in each forested wetland plant association. Tree species density will exceed 200 trees per acre in forested wetland areas. Native wetland shrub species will contribute at least 80% of the shrub density in areas of the forested wetland that are planted with shrubs. Shrub density will exceed 400 stems per acre in areas of the forested wetland that are planted with shrubs. Native tree and shrub species will be in a healthy, vigorous growing condition, with average annual stem elongation of at least 2 inches during years 1-5 of the monitoring program.
Provide in-kind replacement for impacts to 0.39 acre of native shrub wetland.	Plant an association of native shrub wetland species that is similar in composition to naturally occurring shrub wetlands, including species such as Pacific willow, red-osier dogwood, bearberry honeysuckle, Douglas hawthorne, and Pacific ninebark. Plant native shrub species at densities of at least 500 plants per acre (approximately 9 ft on-center).	Shrub wetlands will achieve a minimum in-kind replacement ratio of 2:1 by covering at least 0.78 acre of the mitigation site. Native shrub wetland species will contribute at least 80% of shrub density in the shrub wetland association. Species composition in the shrub wetland will include at least a 5% component of each native species planted. Shrub density will exceed 400 stems per acre in shrub wetland areas. Native shrub species will be in a healthy, vigorous growing condition, with average annual stem elongation of at least 2 inches during years 1-5 of the monitoring program.
Provide in-kind replacement for impacts to 2.88 acres of native emergent wetland.	Plant an association of native emergent wetland species similar in composition to naturally occurring emergent wetlands. Use native species that are suited to seasonally and/or permanently flooded conditions, such as water parsley, slough sedge, hardstem bulrush, and common spike rush. Plant native emergent species in approximately 2,500-ft^2 monotypic patches at densities of 450 plants per 1,000 ft^2 (approximately 18 inches on-center).	Emergent wetlands will achieve a minimum in-kind replacement ratio of 1.5:1 by covering at least 4.32 acres of the mitigation site. Native emergent wetland species will contribute at least 70% of plant cover in areas planted with emergent species. Species composition (stem density or percent composition) in the emergent wetland will include at least a 5% component of each native species planted. Plant densities will exceed 1 stem per 1.5 ft^2 in areas planted with emergent species. Upland emergent species will colonize with 80% cover by native species.
Increase flood storage capacity in the Green River drainage basin.	Grade 29 acres of the mitigation area to an elevation of 45 ft or less. Provide a topographic connection between the site and the 100-year floodplain of the Green River backwater area.	A minimum of 29 acres of the mitigation site will be below the 45-ft elevation and directly connected to the 100-year floodplain.



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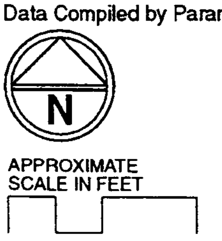


Figure 3.4-1
Aerial Photograph of Proposed Wetland Mitigation Site

SEATAC AIRPORT
55-2912-01 (05)

1/96 K. STEPHANICK

development of regional storm water facilities or other city-designated uses. The site is bounded by a variety of land uses including agriculture to the north and south; undeveloped land, multi-family housing, and a drive-in theater to the west; and the Green River, patches of riparian forest, and undeveloped, forested slopes to the east. A narrow strip of land along the western banks of the Green River is held by King County. The site is currently zoned R2 (single-family residential) by the City of Auburn and the 1995 Comprehensive Plan designation is single-family (Auburn 1995). The site is nearly level but gently slopes to the northwest, with elevations ranging from 45 ft in the northwest corner to 52 ft along the eastern property boundary. Detailed descriptions of on-site hydrology, soils, and vegetation are included in Section 3.4.4, Ecological Assessment of the Mitigation Site.

3.4.2 Ownership

The Port of Seattle owns the 69-acre site.

3.4.3 Rationale for Choice

Implementation of the proposed Sea-Tac Airport Master Plan Update improvements would result in impacts to wetland resources totaling 10.35 acres at 34 individual wetlands (Table 3.1-1). Because most of these wetlands are small and separated from other natural areas by large expanses of urban development, they provide limited ecological functions at the local and landscape scale. The overall intent of this proposed mitigation plan is to offset wetland impacts at a single site, thereby providing a regionally meaningful expanse of habitat with enhanced assurances for successful implementation and long-term protection. Because of draft FAA guidelines for airport operation safety, mitigation planning for impacts to wildlife habitat must seek opportunities for habitat replacement outside a 10,000-ft radius of Sea-Tac's runways.

The search for the mitigation site, which began in February 1995, was constrained by certain limiting parameters including:

- nonwetland sites with evidence of seasonally high water tables,
- vacant or substantially vacant parcels,
- parcels in excess of 10 acres,
- under single ownership (preferably),
- close to surface water features (preferably),
- within the Green River valley from South 180th Street south to the Pierce County border,
- available for purchase by the Port of Seattle.

The properties could not be within 5,000 ft of an existing general aviation airport (because of FAA considerations) in addition to the 10,000-ft guideline for Sea-Tac, or include land to which King County owns the development rights under the farmland preservation program. Also, the conversion of property to wetlands had to be economically feasible.

Although over 100 parcels were initially identified, the search focused to sites larger than 50 acres because of the acreage needed to mitigate impacts at the airport and the ecological and logistical advantages of developing mitigation on a single site. Of eleven sites larger than 50 acres, five sites were rejected because they were unsuitable because of the large amount of wetlands present. These sites offered little or no opportunity for improvement of habitat. Of the six remaining sites, two were not available for purchase, the development rights of two were owned by King County for farmland preservation, and one site had been recently purchased by the City of Kent for its own mitigation purposes.

The remaining site is the one analyzed in this mitigation plan. The site has several attributes that make it favorable for wetland mitigation. It is large enough to accommodate the entire wetland mitigation project and it has excellent physical features that would successfully support the proposed mitigation approach, including proximity to the Green River and to a 100-year floodplain.

In addition, the city of Auburn is planning a regional storm water detention facility to be located in the general vicinity of the proposed mitigation parcel, and the Port of Seattle is exploring options with Auburn to integrate these projects. The proposed wetland mitigation project could receive treated supplemental water from the detention facility, which would be beneficial to the wetland during summer months. Refer to Section 3.5.1 for complete discussion of wetland hydrology.

3.4.4 Ecological Assessment of the Mitigation Site

3.4.4.1 Existing Site Conditions

Vegetation

The mitigation site consists of abandoned agricultural land that is dominated by a mix of native and non-native herbaceous species, including thick stands of Canadian thistle. Grass species intermixed with the thistle include quackgrass, orchardgrass, colonial bentgrass, and a few small patches of reed canarygrass. Table 3.4-1 lists species observed on the mitigation site during site investigations in October 1995. Invasive and noxious species scattered throughout these areas include cocklebur, common dandelion, and climbing nightshade.

A narrow wetland swale bisects the parcel from north to south along the western boundary of the 47-acre mitigation site (Figure 3.4-2). This existing wetland is dominated by grasses that include red top, colonial bentgrass, quackgrass, tall fescue, velvet grass, and patches of reed canarygrass. Other herbaceous species in the wetland include soft rush and creeping buttercup. The mitigation would impact less than 0.1 acre of this existing wetland. Within the mitigation

Table 3.4-1. Plant species observed on the mitigation site during October 1995.

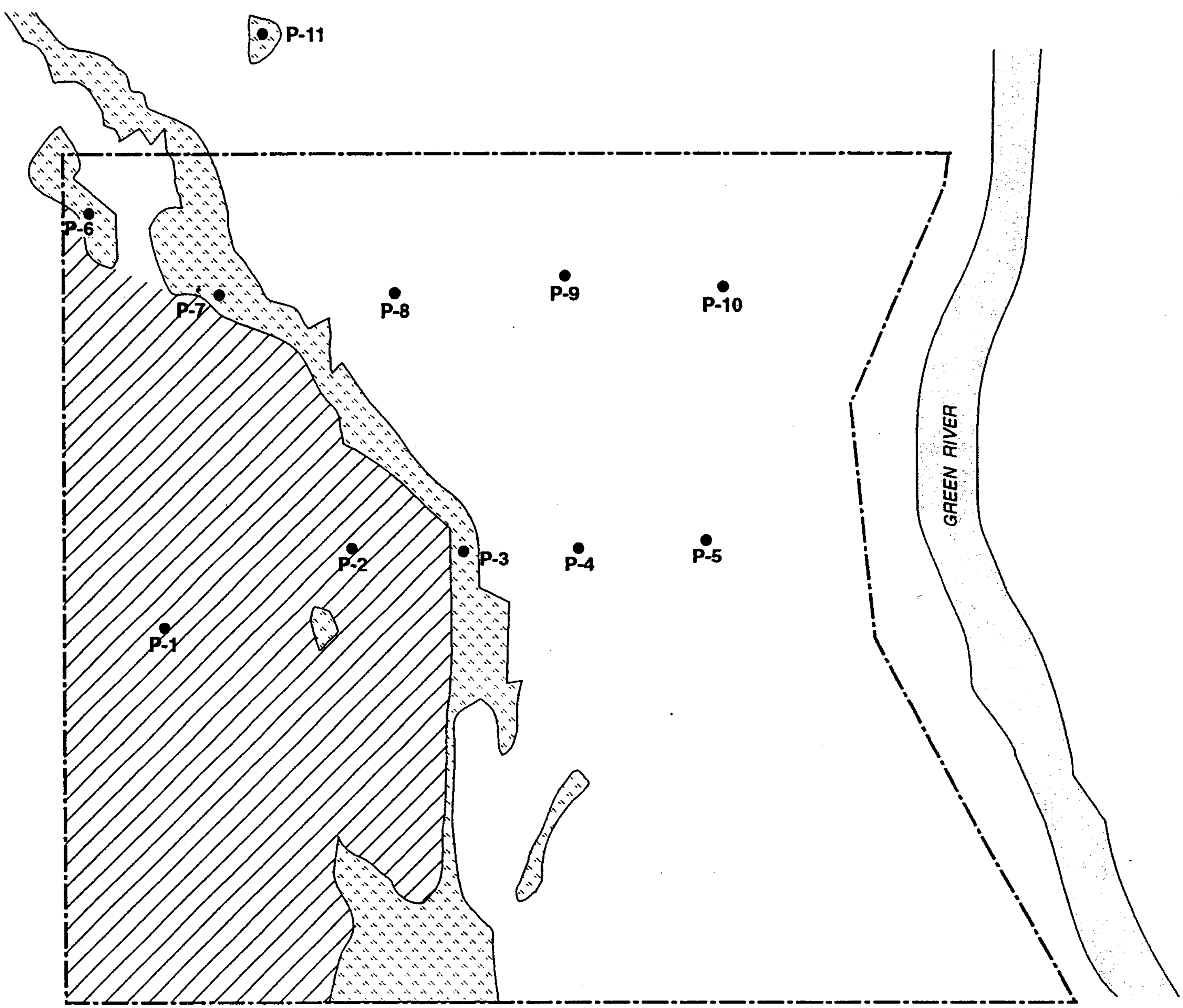
Scientific Name	Common Name	WIS[1]
Trees		
Alnus rubra	Red alder	FAC
Crataegus spp.	Hawthorn	FAC
Fraxinus latifolia	Oregon ash	FACW
Populus trichocarpa	Black cottonwood	FAC
Prunus emarginata	Bitter cherry	FACU
Shrubs		
Acer circinatum	Vine maple	FACU
Cornus stolonifera	Red-osier dogwood	FACW
Corylus cornuta	Beaked hazel-nut	FACU
Cytisus scoparius	Scot's broom	NI
Populus trichocarpa (saplings)	Black cottonwood	FAC
Rosa nutkana	Wood's rose	FAC
Rosa pisocarpa	Pearfruit rose	FAC
Rubus discolor	Himalayan blackberry	FACU
Rubus laciniatus	Evergreen blackberry	FACU
Rubus ursinus	Pacific blackberry	FACU
Salix spp.	Willow	FACW
Salix scoulerana	Scouler willow	FAC
Symphoricarpos albus	Snowberry	FACU
Herbs		
Agropyron repens	Quackgrass	FAC
Agrostis alba	Redtop	FACW
Agrostis tenuis	Colonial bentgrass	FAC
Alopecurus geniculatus	Water foxtail	OBL
Alopecurus pratensis	Meadow foxtail	FACW
Cirsium arvense	Creeping thistle	FACU
Cirsium vulgare	Bull thistle	FACU
Dactylis glomerata	Orchard grass	FACU
Dipsacus sylvestris	Teasel	FAC
Eleocharis palustris	Creeping spikerush	OBL
Epilobium ciliatum	Willow-herb	FACW
Equisetum arvense	Field horsetail	FAC
Festuca arundinacea	Tall fescue	FAC
Geranium spp.	Crane's-bill	FACU
Holcus lanatus	Common velvet grass	FAC
Juncus effusus	Soft rush	FACW
Juncus spp.	Rush	FACW
Lotus corniculatus	Birds foot trefoil	
Phalaris arundinacea	Reed canarygrass	FACW
Phleum pratense	Timothy	FAC

Table 3.4-1. Plant species observed on the mitigation site during October 1995 (continued).

Scientific Name	Common Name	WIS[1]
Phragmites communis	Common reed	FACW
Plantago lanceolata	English plantain	FAC
Poa pratensis	Kentucky bluegrass	FAC
Polystichum munitum	Sword fern	FACU
Ranunculus repens	Creeping buttercup	FACW
Rumex crispus	Curly dock	FAC
Scirpus acutus	Hard-stem bulrush	OBL
Solanum dulcamara	Climbing nightshade	FAC
Tanacetum vulgare	Common tansy	UPL
Taraxacum officinale	Common dandelion	FACU
Trifolium pratense	Red clover	FACU
Typha latifolia	Common cattail	OBL
Xanthium strumarium	Cockle-bur	FAC

Data compiled by Parametrix
[1]Wetland Indicator Status (Environmental Laboratory 1987)

Category	Symbol	Definition
Obligate Wetland Plants	OBL	Obligate wetland plants occur almost always (estimated probability >99%) in wetlands under natural conditions, but may also occur rarely (estimated probability <1%) in non-wetlands.
Facultative Wetlands Plants	FACW	Facultative wetland plants usually occur (estimated probability 67 to 99%) in wetlands, but may also occur (estimated probability 1 to 33%) in non-wetlands.
Facultative Plants	FAC	Facultative plants with a similar likelihood (estimated probability 33 to 67%) of occurring in both wetlands or non-wetlands.
Facultative Upland Plants	FACU	Facultative upland plants usually occur (estimated probability 67 to 99%) in non-wetlands, but also occur (estimated probability 1 to 33% of the time) in wetlands.
Obligate Upland Plants	UPL	Upland plants occur almost always (estimated probability >99%) in non-wetlands under natural conditions.



Note: Wetland boundaries approximate pending field verification by U.S. Army Corps of Engineers

Source: David Evans 1995
Additional Data Compiled by Parametrix

N

APPROXIMATE SCALE IN FEET

0 150 300

Existing Wetland

Well Location

Site Boundary (Approximate)

Reserve Area (for regional storm water facilities or future development)

Figure 3.4-2
Wetlands and Groundwater Monitoring Well Locations on the Proposed Wetland Mitigation Site

site, a wetland (about 0.2 acre) is present. This wetland would be replaced by the mitigation project. The wetland is dominated by reed canarygrass, but includes other emergent plant species, as listed above.

The southern boundary of the mitigation site is a fence line dominated by shrubs with a few scattered trees. Himalayan blackberry is the dominant shrub, and reed canarygrass is the dominant herb. Other shrubs along the fenceline include vine maple, Woods rose, snowberry, and red-osier dogwood. Tree species scattered throughout the fence line consist of Douglas hawthorn, Oregon ash, and black cottonwood.

<u>**Water Regime**</u>

The mitigation site is very flat and without distinctive on-site drainage features. The Green River is located immediately east of the site. A small area of the 100-year floodplain in the northwest corner of the property drains to the Green River to the north (Figure 3.4-3). There is [illegible] hydraulic connection across the mitigation site between the mapped floodplain and the Green River. The narrow wetland swale (see Figure 3.4-2) that bisects the parcel is shallow (<6 inches topographic change) and displayed no wetland hydrology during site investigations for development of this plan (August-November 1995). However, soils in this area display hydric characteristics and do contain a high percentage of silts; this likely restricts their permeability. Drainage characteristics of on-site soils are discussed in Section 3.4.4.3.

Shallow groundwater monitoring began September 12, 1995 when 11 monitoring sites were established (see Figure 3.4-2) to assess the shallow groundwater gradient across the site and to measure any seasonal variations that may occur in response to rainfall and changes in river elevation. Based on these observations, the groundwater table appears to average 8.0 to 9.0 ft below the ground surface in the early fall months, with a rise in groundwater elevation during the transition to the rainy season beginning in November (Table 3.4-2).

<u>**Soils**</u>

The predominant soil type at the proposed mitigation site, below the 6 inches of organic surface material, is silt (or ML) according to the Uniform Soil Classification System (USCS). The silt varies in color from reddish brown to gray, with clay and clay mottles throughout. The Soil Conservation Service soil survey for King County (USDA 1973) identifies the predominant soil in the mitigation area as Oridia silt loam. Other SCS-mapped soils in the area include Renton silt loam and Briscot silt loam (Figure 3.4-4) and their drainage characteristics are listed in Table 3.4-3. The Oridia and Briscot series are described by SCS (USDA 1973) as somewhat poorly drained soils that formed in alluvium in river valleys. In a representative profile, the surface layer is dark grayish-brown silt loam about 9 inches thick. The subsoil is mottled grayish-brown, dark grayish-brown, and gray silt loam and stratified fine sandy loam that extends to a depth of 60 inches or more.

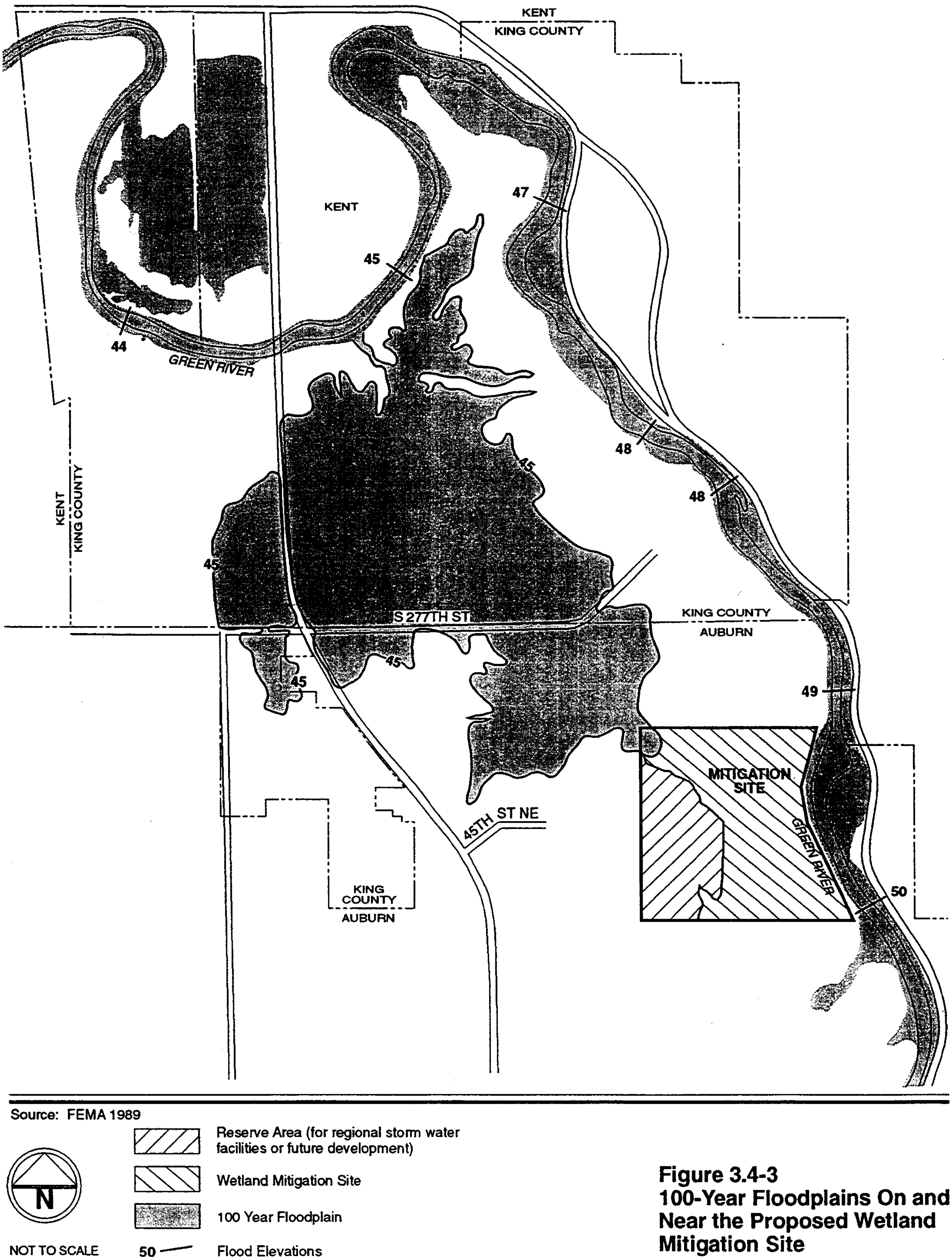
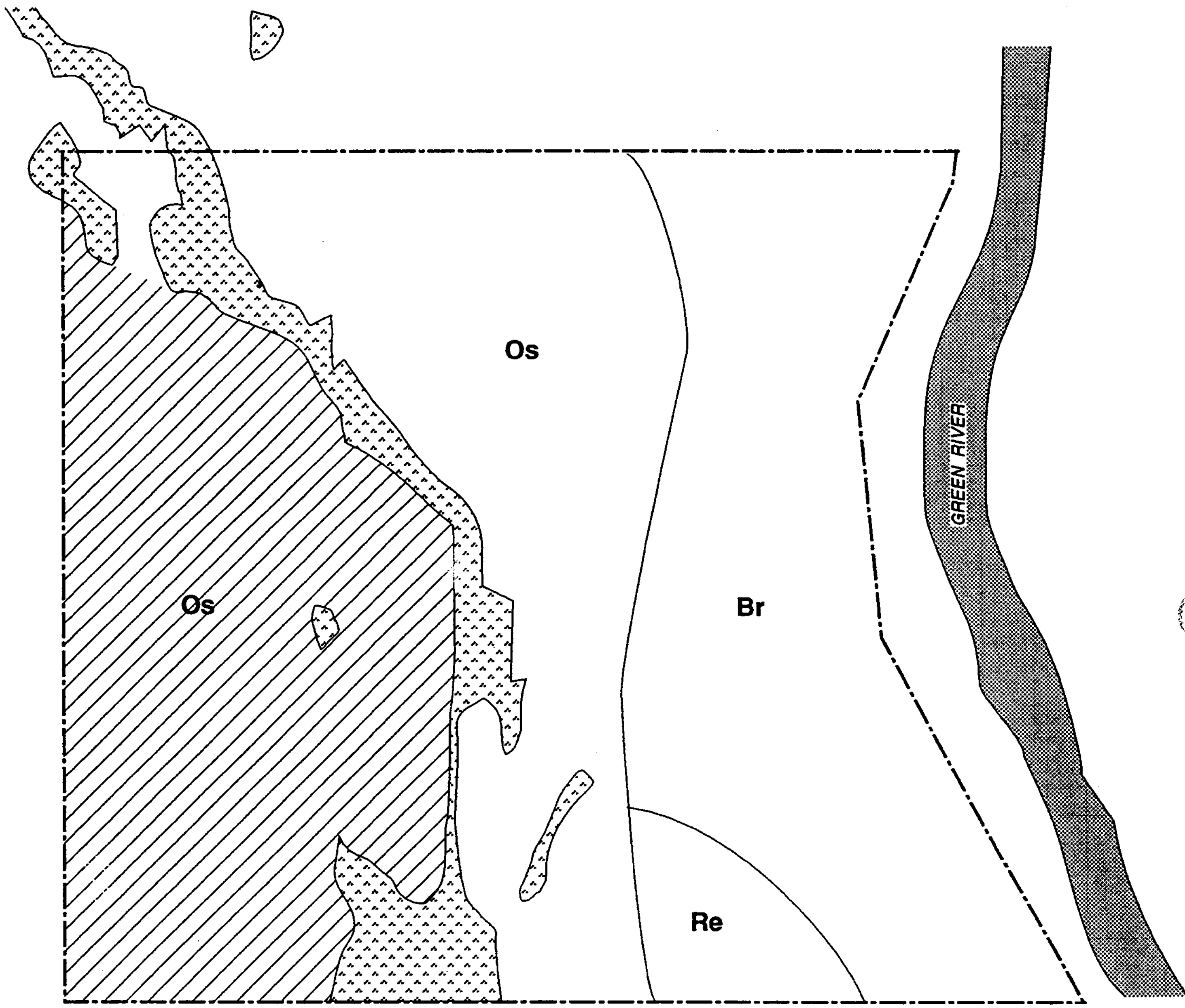


Figure 3.4-3
100-Year Floodplains On and Near the Proposed Wetland Mitigation Site



Note: Wetland boundaries approximate pending field verification by U.S. Army Corps of Engineers

Source: USDA 1973, David Evans 1995
Additional Data Compiled by Parametrix

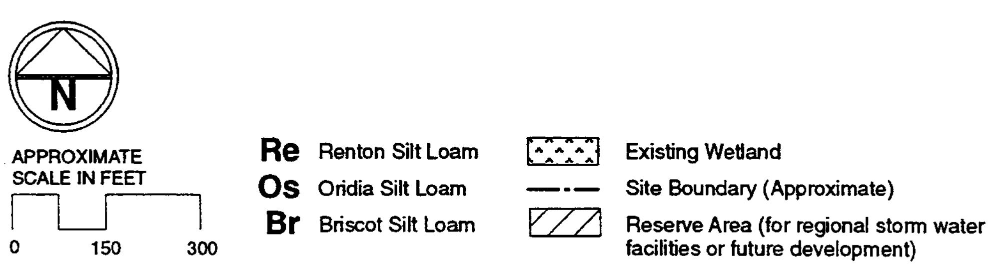


Figure 3.4-4
Soil Types on the Proposed Wetland Mitigation Site

Table 3.4-2. Land surface and depth to groundwater on the proposed wetland mitigation site from September 8, 1995 to present.

Monitoring Well Number	Land Surface Elevation	Depth to Groundwater (ft) Date				
		12 SEP	28 SEP	10 OCT	24 OCT	9 NOV
P-1	47.6	>9.0	>9.0	8.9	7.6	7.0
P-2	46.7	8.0	8.2	8.3	7.6	6.7
P-3	46.7	8.1	8.3	8.4	7.9	6.7
P-4	48.5	>8.8	>8.8	>8.8	>8.8	>8.8
P-5	50.1	>9.0	>9.0	>9.0	>9.0	>9.0
P-6				8.2	7.5	6.7
P-7				8.2	7.5	5.7
P-8				8.8	8.3	5.7
P-9				>7.8	>7.9	>7.9
P-10				>8.3	>8.3	>8.3
P-11				>6.5	>6.5	>6.5

Data compiled by Parametrix

Table 3.4-3. Drainage characteristics of soils on the mitigation site.

Soil Series[a]	Drainage Class	High Water Table			Flooding		
		Permeability[b] (in/hr)	Depth (ft)	Months	Frequency	Duration	Months
Briscot	Poorly	0.63-2.0	1 to -1	Nov-Apr	Occasional	Brief	Dec-Feb
Oridia (drained)	Poorly	0.20-2.0	1 to 3	Nov-Apr	Occasional	Brief	Nov-Apr
Renton	Somewhat poorly	2.0-6.3	1 to 1.5	Nov-Apr	Common	Brief	Nov-Apr

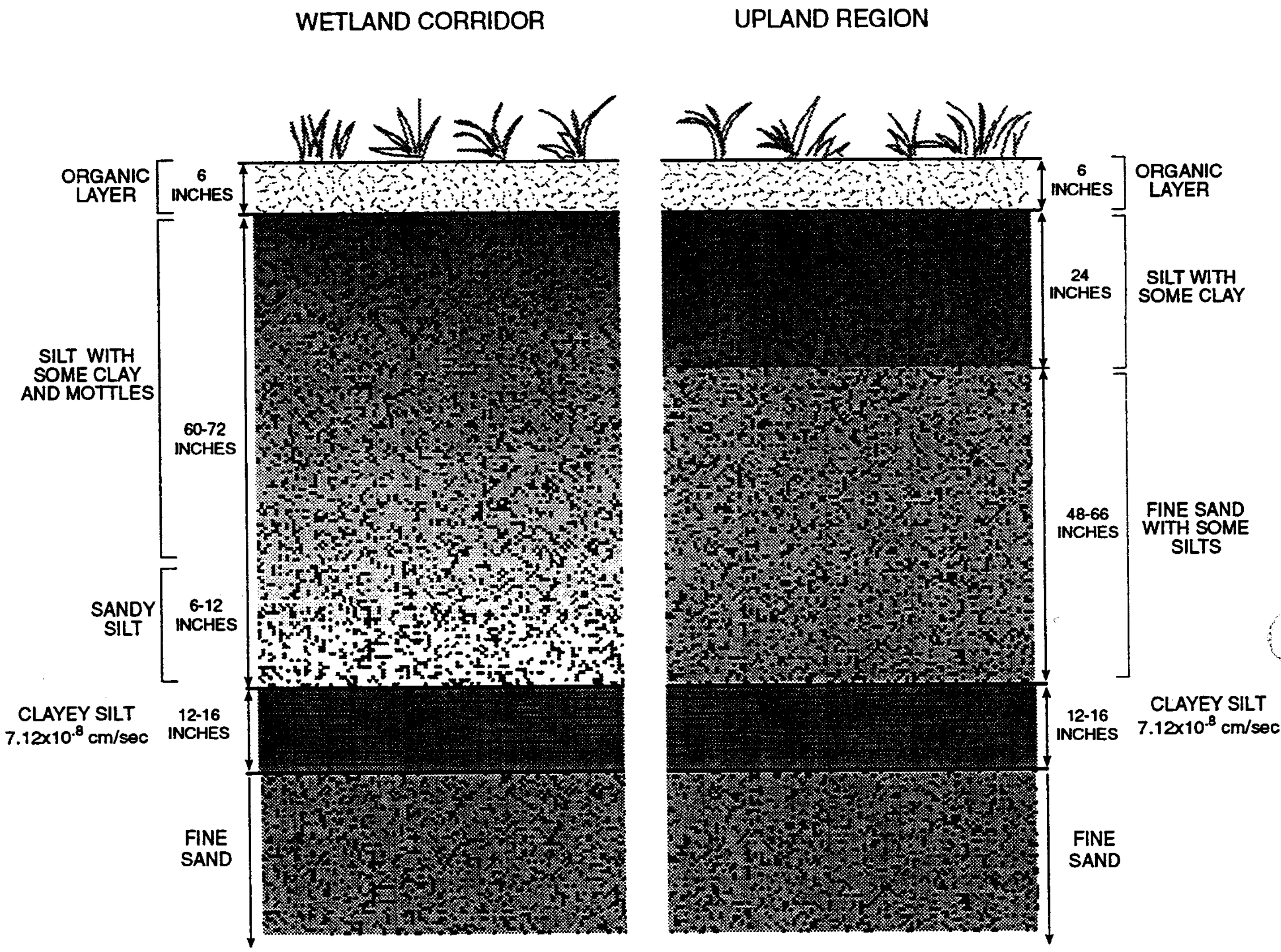
Source: USDA 1973

[a] All soils are classified as hydric (wetland); however, on-site conditions indicate only limited areas of hydric soils are present.

[b] Within the top 20 inches of soil.

This description is consistent with the findings of the soils laboratory testing and the field investigations performed by Parametrix in October 1995. Based on field observations and analytical test results, two distinct soil profiles occur at the proposed mitigation site. For this investigation, the two soil profiles are designated as the wetland corridor and the upland regions. The two soil profiles are presented for comparison in Figure 3.4-5.

The wetland corridor was delineated during previous site investigations and would not be substantially modified during site grading. The wetland corridor soil profile generally consists of a 6-inch acidic organic layer (pH=5.49; organic content = 6.77%) that covers a layer of



Data Compiled by Parametrix

Figure 3.4-5
Typical Soil Profiles at Wetland Mitigation Site

clayey silt. The first 24 inches of the clayey silt is uniform, with clay mottles dispersed throughout. This uniformity is possibly a result of agricultural tilling and cultivating at the site. Below the uniformly mixed silt, the soil is stratified to gray layers of silt and sandy silt that grades to a wet, sandy silt layer at a depth of about 72 inches. Below the wet sandy silt are 12 to 16 inches of very compact clayey silt with an average permeability that varies between 7.12×10^{-8} cm/sec and 2.36×10^{-7} cm/sec (determined at two locations). Below the clayey silt layer, the soil grades to a uniformly fine sand layer. Because of the thickness of the clayey silt layer and the absence of an underlying fine sand layer (similar to the upland soils described below), these soils drain slowly allowing hydric soil characteristics to develop.

The upland portion of the site includes those areas outside the wetland corridor that are planned to be modified as part of the grading plan. The typical soil profile of the upland region is similar to the existing wetland corridor for the first 30 inches, with a 6-inch acidic organic layer followed by a 24-inch, uniformly mixed layer of clayey silt with dispersed mottles. Below the clayey silt layer, the soil is predominantly fine sand, with some silt. The sand is uniform gray up to depths of 96 inches below the surface. A 6- to 8-inch-thick clayey silt layer was again encountered between the 72- and 96-inch depth. Below the clayey silt, the soil returns to a uniform fine sand. The 36- to 48-inch fine sand layer located near the soil surface allows surface soils to drain more quickly than wetland soils, and hydric characteristics have not developed.

Environmental Site Assessment

A Phase I Site Assessment of the mitigation property was conducted in December 1995 (Parametrix 1995) and is incorporated into this document by reference. The report was prepared according to guidelines described in American Society for Testing and Materials (ASTM) Standard Practice for Environmental Site Assessments: Phase I Environmental Site Assessment Process (ASTM E 1527). The assessment did not indicate environmental conditions of concern associated with past or current use of the site and adjacent properties.

Wildlife Habitat

The mitigation site is mostly abandoned agricultural land, which is dominated by grasses and forbs, and a non-flooded, emergent wetland swale. Adjacent areas to the north are still in agricultural use. The habitat south of the site is also disturbed by agricultural use and, to the west of the site, wildlife habitat has been mostly eliminated by residential development. No permanent surface water features occur on the site, and there is no evidence of seasonal surface flow. The most prominent associated habitat feature is the Green River on the eastern site boundary and the steep, forested slopes along the opposite bank of the Green River, which provide habitat connectivity to other natural areas.

The Washington Department of Fish and Wildlife Priority Habitats and Species database (WDFW 1995) identifies the palustrine emergent wetland that bisects the site as a priority habitat (all wetlands are identified by WDFW as priority habitat). The wetland is dominated by reed canarygrass, velvet grass, and soft rush. The habitat quality of the wetland and the adjacent grassy uplands is compromised by invasive species, low vegetative diversity, and lack of habitat structure. Small mammals may use the area for feeding and breeding, but lack cover from predation. The site may provide foraging habitat for raptors, such as Northern harriers—which may use nearby fenceposts as low perches—and red-tailed hawks. Grassy areas on the site lack habitat structure for nesting cover, protection from predation, thermal cover, or perching for passerine species.

A narrow band of shrub vegetation along the south fenceline consists of invasive blackberry species with reed canarygrass undergrowth. Himalayan and evergreen blackberry are non-native species that dominate disturbed habitats. The blackberries provide the only shrub habitat on the site. Although they bear fruit and provide habitat structure that would otherwise not be present, they are considered a nuisance species.

During field investigations in November 1995, tracks or scat of coyote, mink, deer, raccoon, and kingfisher were observed on the mitigation site. Species observed include common snipe, red-tailed hawk, common yellowthroat, and mallard duck.

3.4.4.2 Functions and Values of Mitigation Wetland

The off-site wetland mitigation site is designed to provide in-kind replacement of wetland biological functions affected by implementation of the proposed Sea-Tac Airport Master Plan Update improvements. The proposed design of the mitigation site would also provide additional mitigation for species using wetland buffer areas and other upland habitats at the airport. Although not related to impacts of the proposed Master Plan Update improvements, additional flood storage capacity would be considered as part of the design process. Vegetation cover and site hydrology following construction of the mitigation wetland are discussed in Sections 3.5.1 and 3.5.3.

Wildlife Habitat

Construction of the forested, shrub, and emergent wetlands would create conditions that provide habitat for a variety of wildlife species. Table 3.4-4 identifies a broad range of wildlife species that would be expected to occur over time in the mitigation wetland. However, habitat structure and availability would change as vegetation matures over the next several decades, and many listed species would begin using the site in future years. Table 3.4-5 identifies expected trends in wildlife use of the site through several stages of vegetation establishment, up to and beyond 25 years following construction.

Table 3.4-4. Wildlife species expected to occur in the wetland mitigation site after construction.

	Habitat Type					
	Permanently Flooded Emergent Wetland	Seasonally Flooded Emergent Wetland	Shrub Wetland	Forested Wetland	Riparian Forest	Abandoned Agricultural Land
Amphibians						
Northwestern salamander	X	X		X	X	
Long-toed salamander		X	X	X	X	
Pacific giant salamander	X			X	X	
Rough-skinned newt	X	X			X	
Ensatina					X	
Western toad	X	X				
Pacific chorus frog	X	X	X	X	X	
Red-legged frog	X	X	X	X	X	
Bullfrog (1)	X					
Reptiles						
Common garter snake	X	X	X	X	X	
Birds						
Great blue heron	X	X	X	X		
Canada goose	X	X				X
Green-winged teal	X	X				X
Mallard	X	X	X			X
Northern pintail	X	X				X
American wigeon	X	X				X
Osprey					X	
Bald eagle					X	
Northern harrier	X	X				X
Red-tailed hawk				X	X	X
Killdeer	X	X				X
Common snipe	X	X				
Herring gull	X					
Rock dove (1)						X
Western screech-owl				X	X	
Rufous hummingbird					X	X
Belted kingfisher	X					
Downy woodpecker				X	X	
Northern flicker				X	X	
Pileated woodpecker				X		
Willow flycatcher				X	X	
American/northwestern crow	X	X		X	X	X
Black-capped chickadee				X	X	
Bushtit				X	X	
Bewick's wren			X	X	X	
Winter wren					X	
Marsh wren	X		X			
Golden-crowned kinglet					X	
Ruby-crowned kinglet				X	X	
American robin		X		X	X	X

Table 3.4-4. **Wildlife species expected to occur in the wetland mitigation site after construction (continued).**

	Habitat Type					
	Permanently Flooded Emergent Wetland	Seasonally Flooded Emergent Wetland	Shrub Wetland	Forested Wetland	Riparian Forest	Abandoned Agricultural Land
Varied thrush				X	X	
European starling (1)				X	X	X
Yellow warbler				X	X	
Yellow-rumped warbler				X	X	
MacGillivray's warbler			X	X	X	
Common yellowthroat	X		X			
Wilson's warbler				X	X	
Rufous-sided towhee				X	X	
Fox sparrow				X	X	
Song sparrow	X	X	X	X	X	X
Dark-eyed junco				X	X	
Red-winged blackbird	X	X	X			X
Brown-headed cowbird	X	X	X	X	X	X
American goldfinch				X	X	
House sparrow (1)						X
Mammals						
Vagrant shrew		X	X	X	X	
Pacific water shrew	X	X				
Shrew-mole					X	
Pacific mole						X
Pacific jumping mouse				X	X	
Raccoon	X	X	X	X	X	
Mink	X	X	X	X	X	
Striped skunk					X	X
Coyote			X	X	X	
Red fox			X	X	X	

Data compiled by Parametrix.
[1] Non-endemic species.

Post-construction habitat structure in forested areas would be immature, similar to regenerating forest, and would develop mature forest habitat attributes after several decades. The proposed shrub understory would promote the development of habitat structure. Songbird use in early stages of habitat development would include leaf and bark gleaning species (kinglet/chickadee/bushtit/vireo) that forage in the area. Oregon ash, vine maple, willows, red cedar and hemlock produce seeds that are used by many songbird and mammal species. Small mammals would likely forage for seeds and invertebrates, even though optimal habitat conditions would not occur for one or more decades. As the canopy begins to develop, nesting opportunities and predator avoidance cover would increase.

Table 3.4-5. **Potential wildlife use of constructed wetland habitats.**

	Emergent Wetland				Shrub Wetland				Forested Wetland			
	Years[1]				Years[1]				Years[1]			
Wildlife Types	0-2	2-5	5-15	>15	0-2	2-5	5-15	>15	0-2	2-10	10-25	>25
Mammals												
Shrews and Mice		F	F	F	F	B/F	B/F	B/F	F	B/F	B/F	B/F
Squirrels						F	F	F	F	F	B/F	B/F
Raccoons, Mink		F	F	F		F	F	F		F	B/F	F/M
Fox, Coyote		F	F	F		F	F	F		F	F	F
Bats	F	F	F	F	F	F	F					
Birds												
Shrub-Nesting Songbirds		F	F/M	F/M	F/M	F/M	B/F/M	B/F/M	F/M	B/F/M	B/F/M	B/F/M
Swallows and Swifts	F	F	F	F	F							B/F
Forest-Dwelling Songbirds						F/M	F/M	F/M	F/M	F/M	B/F/M	B/F/M
Cavity-Nesting Birds						F	F	F		F	F	B/F
Marsh-Nesting Songbirds	F	F	B/F	B/F	F	F	B/F	B/F	F	F		
Raptors	F	F	F	F	F	F	F	F	F	F	F	B/F
Wading Birds		F/M	F/M	F/M	F/M	F/M						
Dabbling Waterfowl		F/M	F/M	F/M	F/M	F/M						
Diving Waterfowl		F/M	F/M	F/M								
Herpetofauna												
Reptiles		F	F	F	F	B/F	B/F	B/F	F	B/F	B/F	F
Terrestrial-Breeding Amphibians		F	F	F	F	F	F	F	F	B/F	B/F	B/F
Aquatic-Breeding Amphibians		F	B/F	B/F	F	B/F	B/F	B/F	F	F	F	F
Macroinvertebrates												
Aquatic Insects	B/F	B/F	B/F	B/F	B/F	B/F	B/F	B/F	B/F	B/F	B/F	B/F
Gastropods			B/F	B/F		B/F	B/F	B/F		B/F	B/F	B/F

Data compiled by Parametrix
B = Breeding F = Foraging M = Migration
[1]Years after construction.

Tree nesting songbirds (such as thrushes, vireos, and warblers) would use horizontal branches for nesting when the canopy closes enough to provide cover. Leaf litter and forest detritus would begin to accumulate, providing habitat for the invertebrates (Pennak 1989) that amphibians (such as ensatina), small mammals, and ground-foraging birds feed on. Small mammals, in turn, would become food for predators (such as barred owls). Over the course of several decades, forest competition, disease, or climatic conditions would weaken some trees and likely result in mortality. Dead and decaying trees would eventually provide woody debris and snag habitat for flickers, woodpeckers, and small cavity-nesting birds.

The shrub and emergent wetlands should reach stable habitat conditions earlier than the forested wetland community. Shrub species should produce forage and nesting opportunities within two to ten years. Swainson's thrush and Wilson's warblers use moist shrub habitats for nesting and foraging. Berries produced by salmonberry, elderberry, and red-osier dogwood are used by several insectivorous songbird species to supplement fall and winter diets (Ehrlich et al. 1988). Mink, shrews, and other small mammals would readily exploit insect and aquatic invertebrate food sources. Wading birds, such as great blue herons and bitterns, can feed on small mammals and amphibians. Amphibian use, however, would likely be limited by immigration rates because of the lack of existing amphibian habitat in the area. Some species, such as Pacific giant salamander, northwestern salamander, and rough-skinned newt commonly migrate through terrestrial habitats and may be expected to use the mitigation site.

Although flooded emergent wetlands can provide substantial forage opportunities for ducks, habitat use would vary with proximity to upland predator cover. Waterfowl, which are wary of dense shrubs that allow predators to approach undetected, prefer interspersion of emergent vegetation and open water. Slough sedge, spike rush, and scouring rush are all species preferred by dabbling ducks and geese during migration (Payne 1992). Narrow-leaf burreed is preferred by dabblers and migrating wood ducks. As decaying vegetation builds up in flooded areas, shovelers, pintails and other diving species could use growing populations of plankton, algae, aquatic insects, and gastropods.

3.5 MITIGATION SITE PLAN

The mitigation site plan and general construction methods used to achieve mitigation design objectives are presented in this section. Considered in detail are the evaluation methods and justification for establishing the wetland water regime, the grading plan, revegetation plan, and monitoring and contingency plans for wetland development.

The potential impacts associated with developing the site for wetland mitigation were assessed in *Environmental Report: Port of Seattle Master Plan Improvements Wetland Mitigation Site, Auburn, Washington*, (Parametrix 1996) which is incorporated into this document by reference. The report found no significant environmental impacts associated with implementation of the mitigation project that could not be mitigated.

3.5.1 Water Regime

An adequate water regime is the most critical factor required to establish the desired forest, shrub, and emergent wetland vegetation classes on the mitigation site. The duration and amount of standing water, or soil saturation, control the wetland community types present on-site. (Throughout this report saturation depths refer to the level of water in a hole or monitoring well.) Based on the design objectives outlined in Table 3.3-2 and knowledge of typical hydrology in native Puget Sound wetland communities, a proposed hydrologic regime for the mitigation wetland was developed:

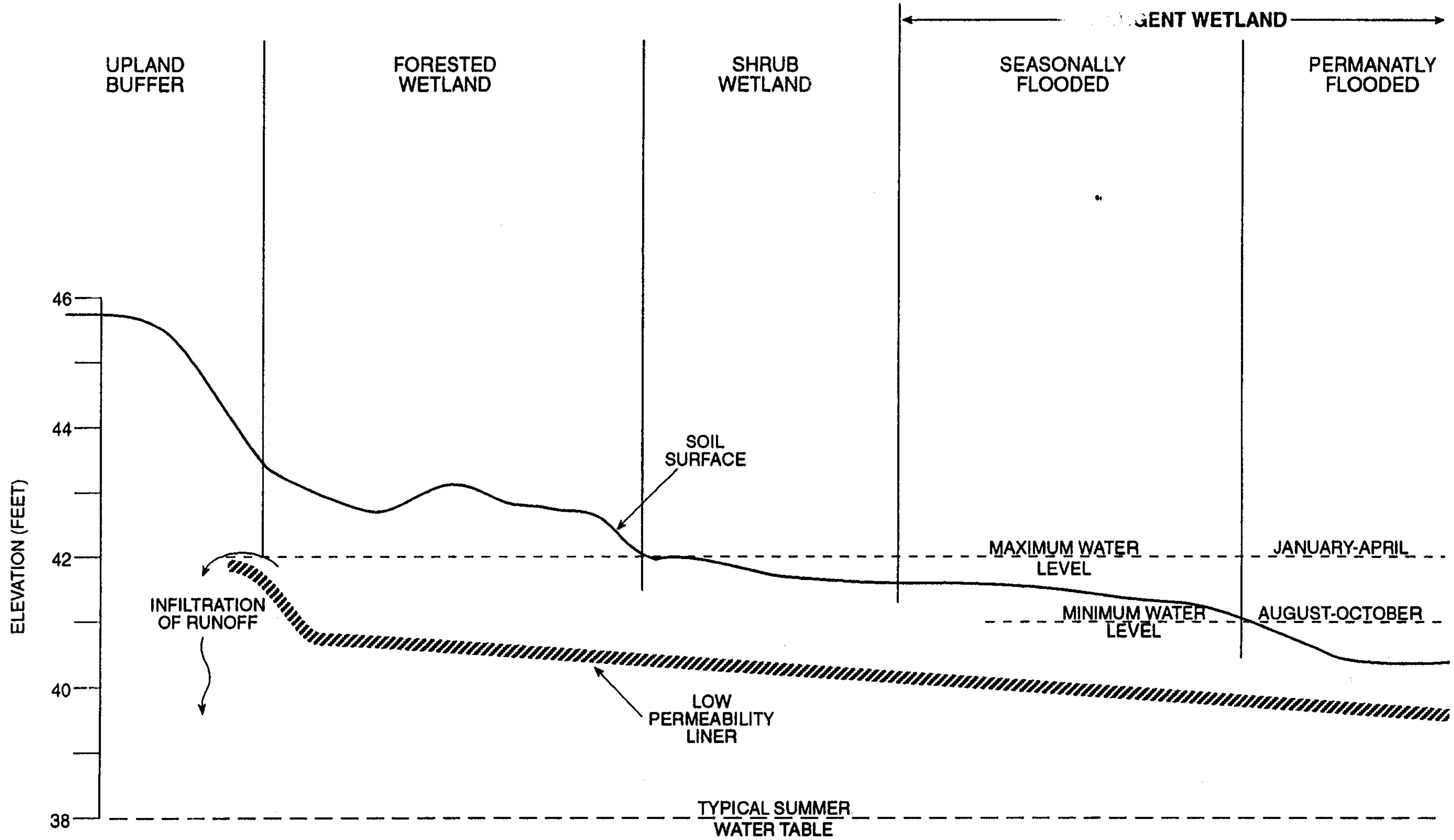
- **Forested Wetlands** would be established where soils are seasonally saturated during the winter and spring period (late December - April). Soil saturation in forested wetlands would be 12 to 18 inches below the soil surface during much of this period.

- **Shrub Wetlands** would be created in areas where soils remain wet throughout the year (saturated to within 6 to 12 inches of the soil surface). Flooding with up to 6 inches of standing water would occur during the December - May period.

- **Emergent Wetlands** would be established where extended periods of flooding (up to 12 inches deep) are present. In areas where flooding is not permanent, soils would remain moist or saturated to within 6 inches of the soil surface throughout the summer months.

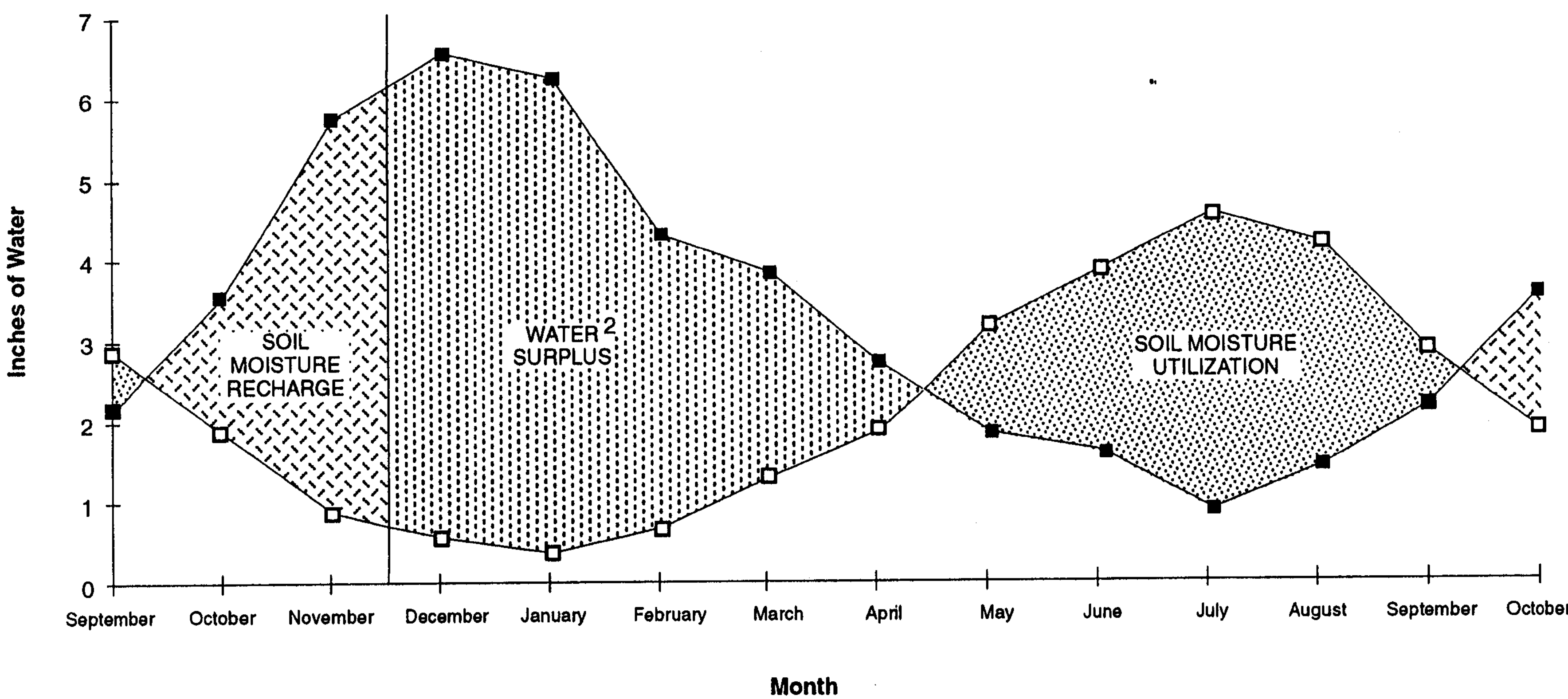
Groundwater monitoring on the mitigation site indicates that it is feasible to create the hydrologic conditions defined above by excavating a basin to intercept the shallow groundwater that occurs at depths of at least 8 to 9 ft below the ground surface during late summer months (see Table 3.4-2). However, to reduce overall earthwork, the above hydrologic regime would be established by creating a perched water table in an excavated basin that has been lined with low-permeability soils (native on-site soils compacted or amended with clay) (refer to Section 3.5.2, Site Grading). Water levels in the excavated basin would be controlled by seasonal patterns of precipitation and evapotranspiration, rather than by seasonal fluctuations in groundwater levels. The relationship of the proposed wetland vegetation zones to water levels and site topography are shown in Figure 3.5-1. The methods used for developing the water level regime are summarized below.

Seasonal precipitation and evapotranspiration patterns for the project area based on 30 years of rainfall data are summarized in Table 3.5-1. Figure 3.5-2 illustrates the annual soil moisture regime in a wetland based on the 30-year rainfall data for the area. The figure shows that over a 14-month period starting in September, there is a period of soil moisture recharge until about 6 inches of total precipitation has accumulated (typically by mid-November). Following this recharge, additional precipitation generates surplus water in the soil profile that results in soil saturation and runoff. Once a soil water surplus develops, water depths in the wetland basin increase throughout the winter months. During early spring, evapotranspiration increases to



Data Compiled by Parametrix

Figure 3.5-1
Relationship of Seasonal Water Level Variations and Surface Elevations to Proposed Wetland Vegetation Classes



1 Based on precipitation data for Kent, Washington (Department of Commerce 1982) and estimated evapotranspiration data for Seattle-Tacoma Airport and the Puyallup Experiment Station (Cooperative Extension Service 1968). Soil moisture based on water-holding capacities of on-site soils (Snyder, et al. 1973).

2 Water surplus is available to fill wetland basin to design capacities.

■ Average Monthly Precipitation

□ Average Monthly Potential Evapotranspiration and Infiltration

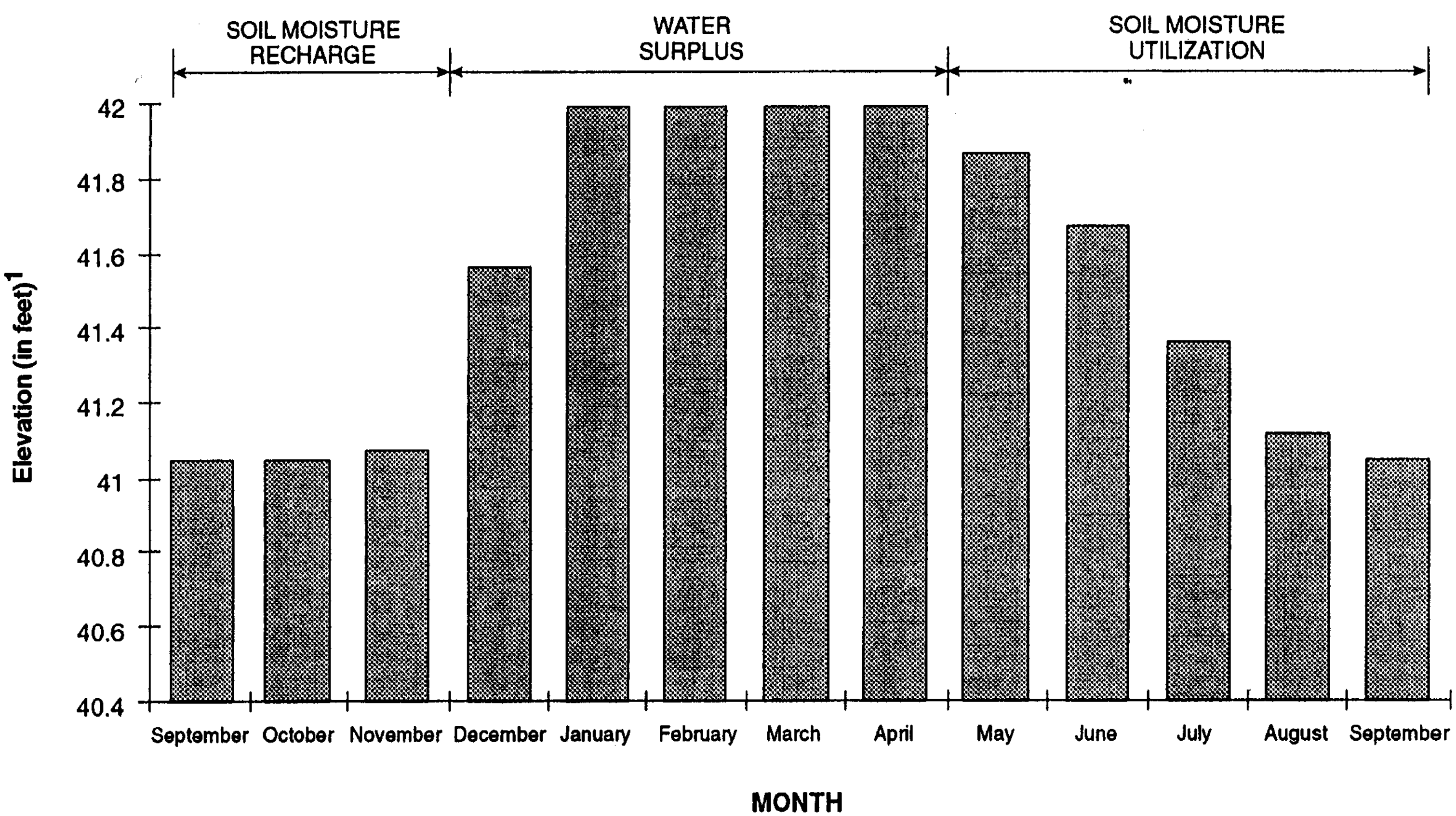
Figure 3.5-2
Moisture Regime for the Proposed Mitigation Wetland[1]

rates exceeding precipitation, resulting in the use of soil moisture and a gradual decline in water depths.

The model described above can be used to predict the hydrologic regime in a closed wetland basin such as the proposed mitigation wetland. The constructed basin would be lined with a low-permeability layer of compacted soil that would limit infiltration of precipitation into the underlying soil to less than 0.1 inch per month. Since Puget Sound is an area where annual precipitation exceeds the combined effects of evapotranspiration and infiltration, the wetland basin would fill with water and generate runoff during the winter months (Figure 3.5-3). Considering site topography and soil conditions, the wetland would be designed to accumulate water up to surface elevations of 42 ft (see Section 3.5-1). Above this elevation, the basin would be unlined, and runoff from the wetland would infiltrate into the subsoil on the site (as is presently the case). During spring and summer months, as soil water is used, water levels would decline to about 41 ft by late September. During October and early November, precipitation would recharge soil moisture and increase water levels to the 42-ft winter elevation.

Table 3.5-1 presents average precipitation values and shows an average annual excess of over 13 inches of water. Based on historical climate data (Department of Commerce 1982), even during record dry periods, sufficient rainfall would occur during the fall and winter months to fill the wetland basin. Thus, the wetland would have the designed water depths by April of each year. During years when below-normal rainfall occurs in the spring and summer months, water levels would decline more rapidly and to lower levels compared to average rates shown in Figure 3.5-3. During abnormally dry summers (when little or no rainfall may occur for 30 to 90 days), the emergent wetland communities could be dry from late June to mid October and shrub wetlands could be dry from late May to late October. Because of the infrequency of extended dry periods, and the fact that many shrub and emergent wetlands in Puget Sound are dry during late summer months, no long-term adverse effects from periodic drought are anticipated.

A regional storm water detention facility may be constructed by the city of Auburn in the general vicinity of the proposed mitigation parcel and could be used to enhance hydrologic conditions in the wetland. The mitigation wetland could receive treated supplemental water from the detention facility (treated and conveyed to the wetland through a biofiltration swale) during summer months when water levels are lowest. This additional water would be particularly beneficial during below-average rainfall years and could ensure that standing water would be present in emergent areas throughout the summer of all years. However, the feasibility of integrating the wetland with a storm water detention facility has not been fully investigated, and the wetland has been designed to function without supplemental water. Integration of the facilities would require only minor modifications of the mitigation plan for the addition of biofiltration swales and an adjustable flow control structure to divert a portion of the storm water into the wetland.



Data Compiled by Parametrix

[1] Water level elevations were calculated from changes in monthy excess of precipitation (Table 3.5-1), assuming maximum elevations of 42 feet and 6-inch water storage capacity of on-site soils

Figure 3.5-3
Predicted Seasonal Water Level Variations in Proposed Mitigation Wetland

Table 3.5-1. **Monthly precipitation, evapotranspiration, and cumulative water balance (in inches) for a lined wetland basin in Kent, Washington (1950-1980).**

		Water Losses			
Month	Average Rainfall	Potential Evapotranspiration[1]	Infiltration[2]	Monthly Excess[3]	Cumulative Excess
October	3.47	1.8	0.1	1.57	1.57
November	5.68	0.8	0.1	4.78	6.35
December	6.48	0.5	0.1	5.88	12.23
January	6.18	0.3	0.1	5.78	18.01
February	4.23	0.6	0.1	3.53	21.54
March	3.77	1.2	0.1	2.47	24.01
April	2.64	1.8	0.1	0.74	24.75
May	1.75	3.1	0.1	-1.45	23.3
June	1.52	3.8	0.1	-2.38	20.92
July	0.81	4.5	0.1	-3.79	17.13
August	1.34	4.1	0.1	-2.86	14.27
September	2.05	2.8	0.1	-0.85	13.42
Annual	**39.92**	**25.3**	**1.2**		**13.42[4]**

Source: Cooperative Extension Service 1968; Department of Commerce 1982

1 Evapotranspiration is the physical loss of water to the atmosphere from soil and water surfaces (evaporation) and the physiological loss of water from plant foliage to the atmosphere (transpiration). Potential evapotranspiration is the amount of water loss that occurs when available soil moisture exceeds that needed for transpiration (associated with plant growth) and evaporation. Since wetland conditions will exist year-round, soil moisture will not be limiting and potential evapotranspiration rates will control water levels in the wetland.

2 Infiltration is the amount of water lost from the proposed wetland through the low-permeability soil liner.

3 Equals rainfall minus water losses.

4 Annual excess is lost from the wetland by discharge to adjacent areas when water levels exceed elevation 42, which corresponds to the top edge of the low-permeability liner.

The proposed wetland would be located within the 100-year floodplain of Green River backwater areas (FEMA 1989). Within this area, the base 100-year flood elevation is 45 ft; thus, during 100-year flood events, forested and shrub wetland areas that are typically non-flooded would be flooded with up to 3 ft of water. Emergent wetland communities would also experience increased flooding of 3 ft (see Figures 3.5-1 and 3.5-5). Flooding of the wetland is not expected to alter wetland plant communities because of its infrequent occurrence and short duration. All wetland plant species included in the mitigation plan (Section 3.5.3) are adapted to saturated soil conditions and have established naturally in areas subject to periodic flooding along many Puget Sound rivers. It is anticipated that excess ponding from periodic floods would recede quickly due to lateral flux over the proposed liner.

3.5.2 Site Grading

The mitigation design objectives would be achieved by grading a basin to a range of depths and creating a perched water table several feet above the natural summer water table at the site (Figures 3.5-4 and 3.5-5). This section discusses the technical considerations, constructability issues, and limitations associated with grading the mitigation site.

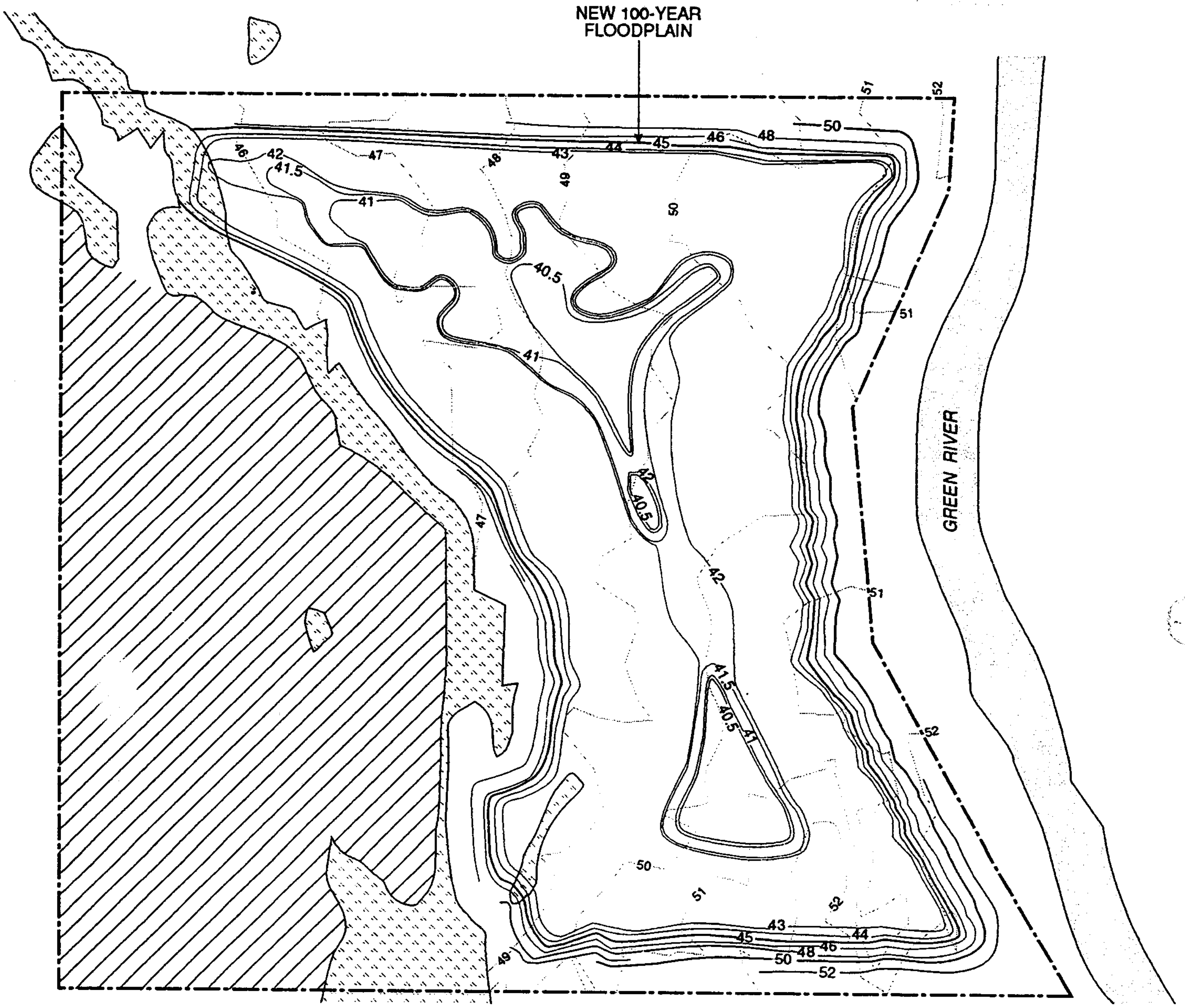
The proposed grading involves four earthwork construction steps. First, the top 6 inches of soil would be excavated and removed from the site. Five to 10 ft of underlying sandy silt-loam soils would be excavated to form a basin, with approximately one-third of the soil stockpiled for reuse on site (two-thirds available for off-site use). The third step is to line the basin with a 9- to 12-inch layer of compacted soil of low permeability, to create an artificial "perched" groundwater condition. The last grading step is to replace the stockpiled soil over the low-permeability layer. This soil would be graded at varying thicknesses to provide the appropriate rooting depth and zones of saturation for each of the desired wetland classes. The construction steps related to technical issues and approximate soil quantities are described below.

3.5.2.1 Surface Soil Removal

Surface soil would be removed because of potential adverse impacts from invasive plants. Excavation of 6 inches of surface soil in most areas would largely eliminate seeds, roots, and rhizomes and reduce colonization by most invasive plants; excavation depths may be slightly greater where reed canarygrass predominates. Based on a site grading area of 29 acres (including the wetland and floodplain areas below elevation 45) and removal of 6 inches of surface topsoil, the quantity of topsoil hauled off-site would be approximately 23,400 yd^3.

3.5.2.2 Basin Excavation

Approximately 400,000 yd^3 of soil would be excavated to create the wetland basin (Figure 3.5-5), with excavation depths ranging between 7 and 12 ft across the site. Approximately one-third of the excavated material would be selectively stockpiled on-site. Fine-grained clayey silt soils would be used to construct the low-permeability liner, and sandy loam soils would serve as backfill and replacement soils. The bottom of the excavation would have a slight slope toward the low point(s) in the basin. The transition slope, between the floor of the basin and the undisturbed grades around the perimeter of the mitigation area, would be approximately 10H:1V (horizontal to vertical) to facilitate planting and to minimize the potential for erosion.



Note: Wetland boundaries approximate pending field verification by U.S. Army Corps of Engineers

Source: David Evans 1995
Additional Data Compiled by Parametrix

N

APPROXIMATE SCALE IN FEET

0 150 300

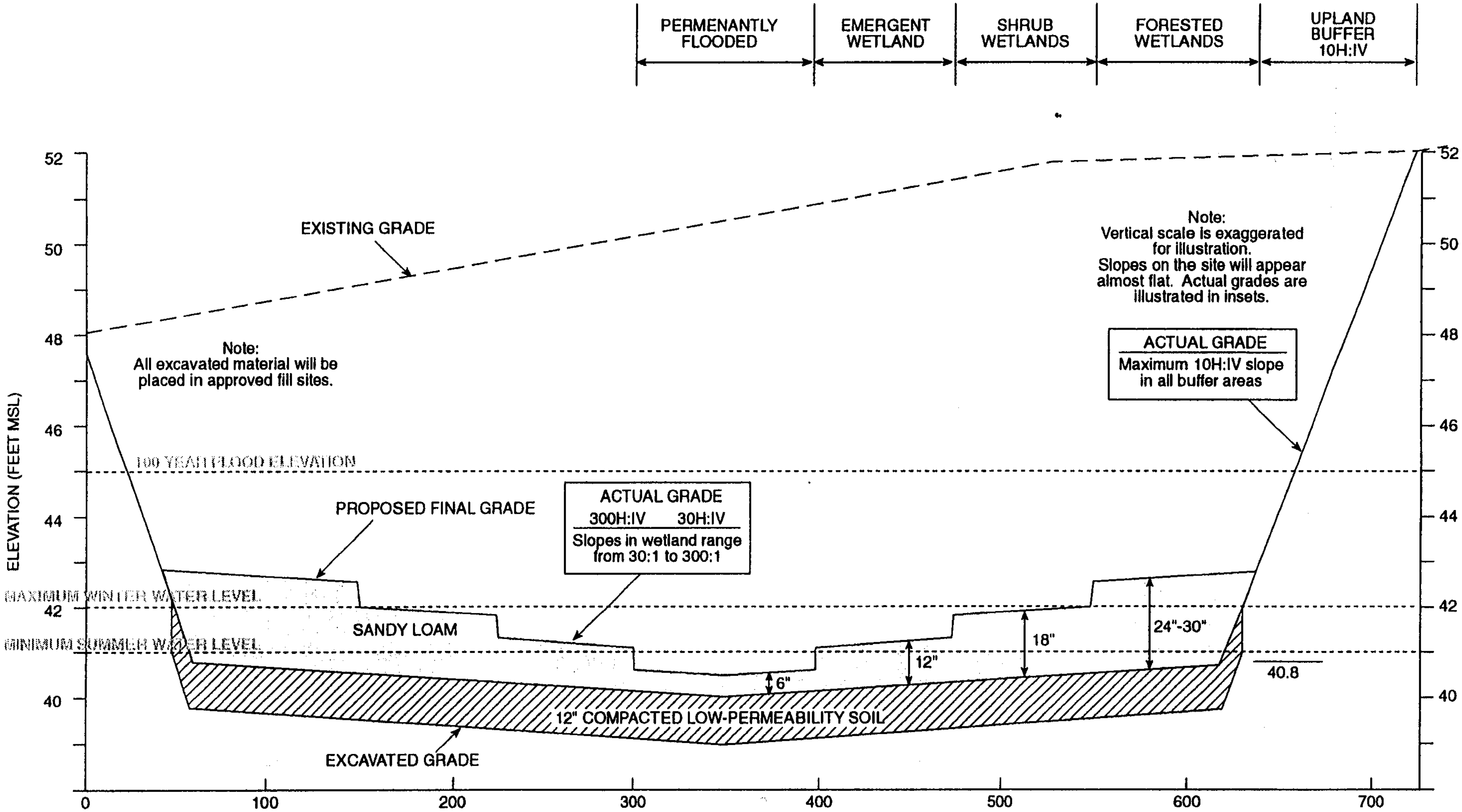
— 50 — Proposed Grade

····· 50 ····· Existing Grade

Existing Wetland

Reserve Area (for regional storm water facilities or future development)

Figure 3.5-4
Site Grading



Data Compiled by Parametrix

Figure 3.5-5
Site Grading
Idealized Cross Section

3.5.2.3 Low-Permeability Layer Construction

The low-permeability layer construction would depend on the quantity and properties of the clayey silt encountered during excavation. It is anticipated that a 9- to 12-inch-thick layer would be constructed using compacted native soils, if sufficient quantities of suitable soils are found. Approximately 44,000 yd^3 of a compactable soil are needed to create a 12-inch-thick low-permeability layer over the 27-acre wetland area.

Preliminary site soils information (collected at monitoring well locations shown in Figure 3.4-2) shows that a clayey silt layer extends from the 6- to 30-inch depth. If the clay layer is continuous and 15 inches of the clayey silt are available after surface soil removal, a sufficient quantity of this material would be stockpiled for construction of the low-permeability layer. Another layer of clayey silt was found at depths ranging from 72 to 96 inches. The measured permeability of this lower clayey silt layer also appears adequate to meet project requirements. However, the extent and continuity of this lower layer is unknown, and the measured thickness of this layer (6 to 8 inches) would make it difficult to segregate during excavation. These soils would be considered for construction of the low-permeability liner only if sufficient quantities of the shallower clayey silt are unavailable. Additional field investigations within the proposed grading area will be conducted to further characterize the available soils.

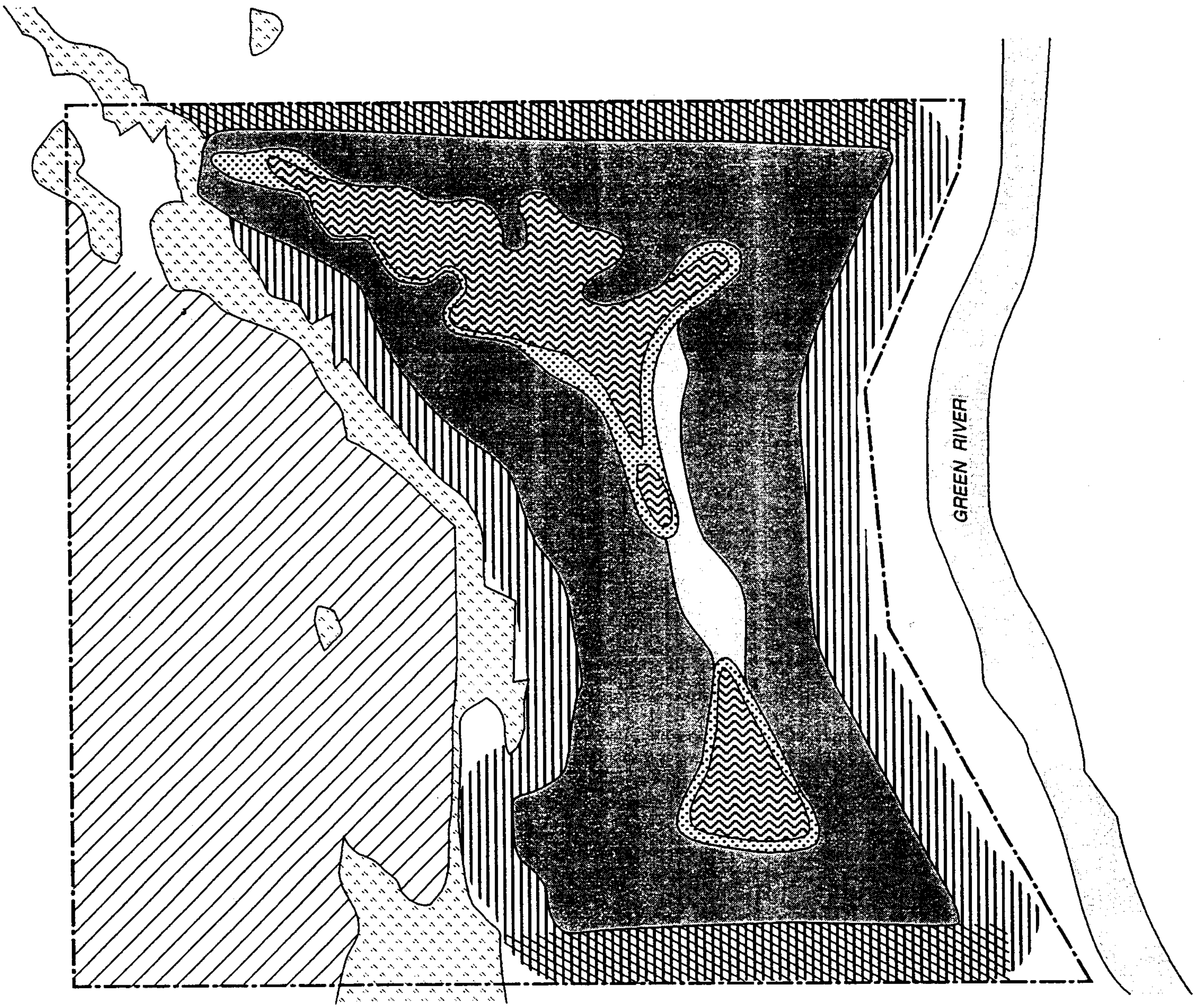
If suitable on-site soils are not found in sufficient quantity, bentonite would be used as a soil amendment to reduce the permeability characteristics of available soils. This method has been used successfully in other large-scale earthwork projects such as landfill and pond liner improvements, and is appropriate for wetland construction.

3.5.2.4 Soil Replacement and Finish Grading

As shown in Figure 3.5-5, soil would be placed and graded to varying thicknesses over the low-permeability layer to provide the proper rooting depth and zone of saturation for the selected vegetation classes. Generally, soil thickness would change in increments of approximately 6 inches between wetland classes, with the thickest soils occurring in forested areas. The proposed grading and wetland class acreages indicate that approximately 100,000 yd^3 of replacement soil are needed. The on-site sandy loam material would be used as a topsoil.

3.5.3 Landscape Plan

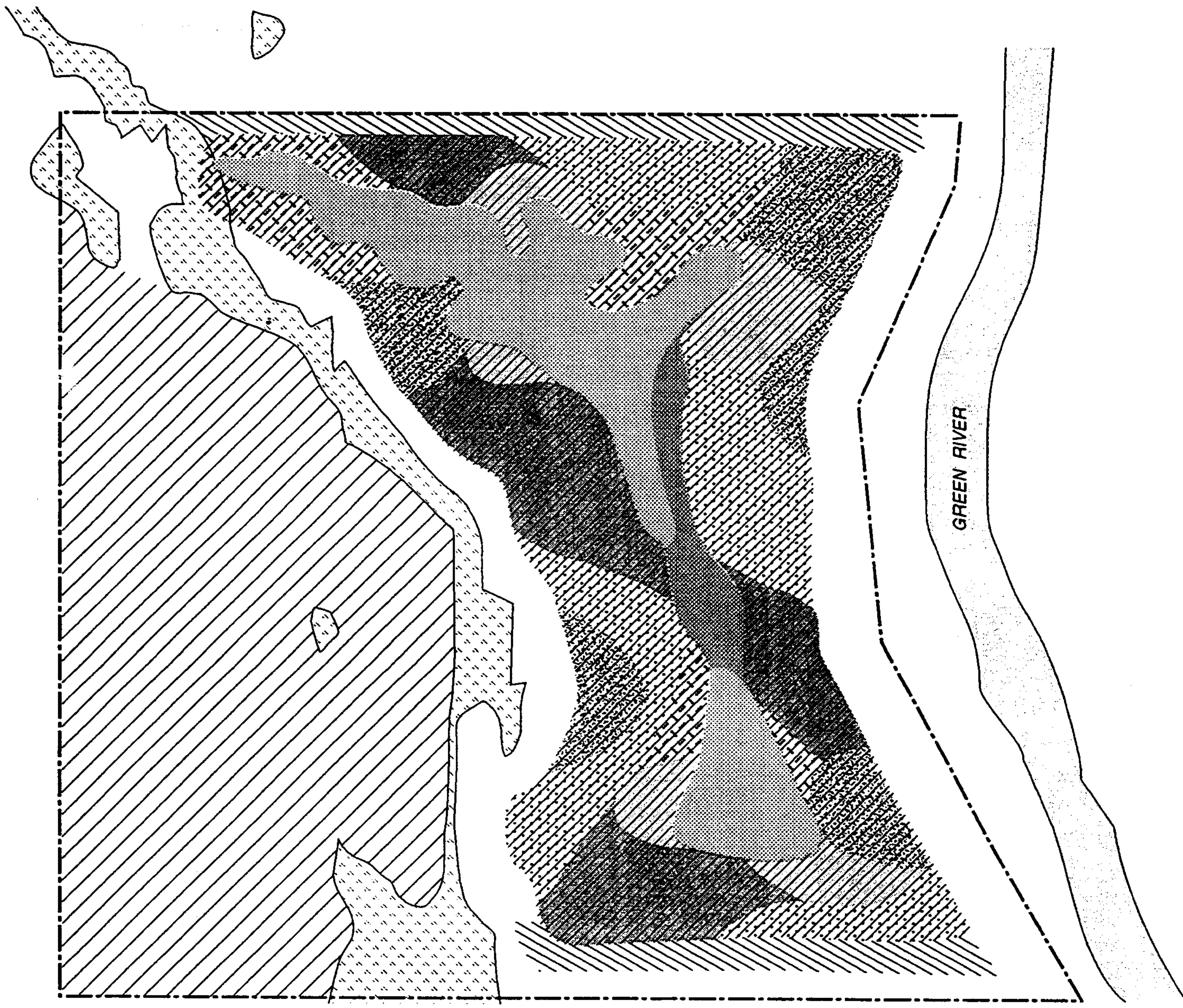
Four wetland vegetation classes would be planted in the mitigation area (Figure 3.5-6). These general classes would include eight wetland plant associations (Figure 3.5-7) typical of freshwater wetlands and forested uplands in the northern Puget Sound basin. These plant associations are groups of plants selected to mimic naturally occurring native plant groups that may be found within a wetland class. The species composition and relative abundance of species in each plant association and their wetland indicator status are listed in Table 3.5-2.



Note: Wetland boundaries approximate pending field verification by U.S. Army Corps of Engineers

Source: David Evans 1995
Additional Data Compiled by Parametrix

Figure 3.5-6
Proposed Wetland Classes and Buffer Vegetation Types for the Wetland Mitigation Site



Note: Wetland boundaries approximate pending field verification by U.S. Army Corps of Engineers

Source: David Evans 1995,
Additional Data Compiled by Parametrix

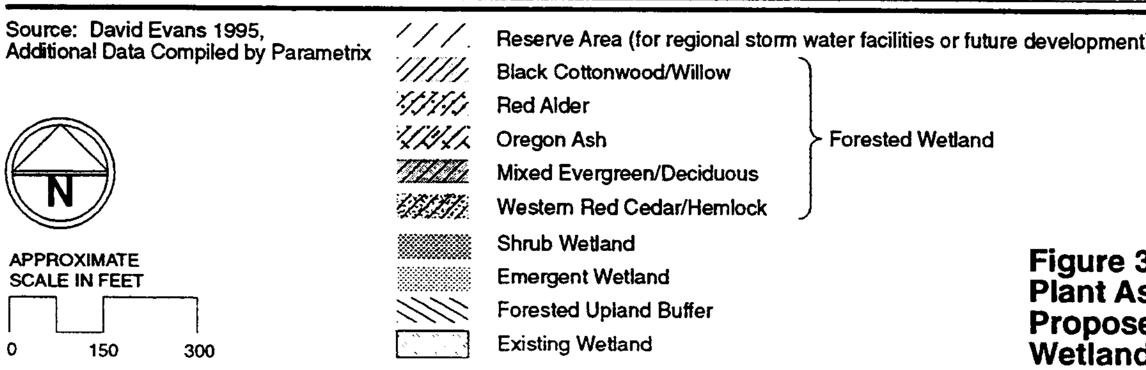


Figure 3.5-7
Plant Associations Proposed for the Wetland Mitigation Site

Table 3.5-2. Plant species proposed for the wetland mitigation area.

			Plant Associations[1]									
Scientific Name	Common Name	Indicator Status	Upland Hydroseed	Wetland Hydroseed	Buffer	Emergent	Shrub	Western Redcedar/Hemlock	Mixed Evergreen/Deciduous	Oregon Ash	Red Alder	Black Cottonwood/Willow
Trees												
Acer macrophyllum	Big-leaf maple	FACU			+							
Alnus rubra	Red alder	FAC							+	-	+	
Fraxinus latifolia	Oregon ash	FACW								+		-
Malus fusca	Western crabapple	FACW						-		-		-
Picea sitchensis	Sitka spruce	FAC						-				
Populus trichocarpa	Black cottonwood	FAC									-	+
Populus trichocarpa X *deltoides*	Hybrid cottonwood	FAC									-	+
Pseudotsuga menziesii	Douglas fir	FACU			+							
Salix sitchensis	Sitka willow	FACW								+		
Salix hookeriana	Hooker's willow	FACW-						+	+	-		
Salix lasiandra	Pacific willow	FACW+										+
Thuja plicata	Western red cedar	FAC						+	+		-	
Tsuga heterophylla	Western hemlock	FACU-			+			+			-	
Shrubs												
Acer circinatum	Vine maple	FAC-			+			+			+	
Cornus stolonifera	Red-osier dogwood	FACW					+				+	-
Corylus cornuta	Hazelnut	FACU			+			+				
Lonicera involucrata	Twinberry	FAC+						+			+	+
Oemleria cerasiformis	Indian plum	FACU			+			-				
Physocarpus purshiana	Pacific ninebark	FACW-					+	-	-		-	
Rosa nutkana	Nootka rose	FAC			-			-	-			
Rubus spectabilis	Salmonberry	FAC+								-	+	-
Salix scouleriana	Scouler's willow	FAC					+					

Table 3.5-2. Plant species proposed for the wetland mitigation area (continued).

Scientific Name	Common Name	Indicator Status	Plant Associations[1]: Upland Hydroseed	Wetland Hydroseed	Buffer	Emergent	Shrub	Western Redcedar/Hemlock	Mixed Evergreen/Deciduous	Oregon Ash	Red Alder	Black Cottonwood/Willow
Sambucus racemosa	Red elderberry	FACU			-						-	
Symphoricarpos albus	Snowberry	FACU			+							
Herbs												
Carex obnupta	Slough sedge	OBL				+	+			+		+
Eleocharis palustris	Common spike-rush	OBL				+						
Lolium multiflorum	Annual ryegrass	UPL	-	-		+						
Oenanthe sarmentosa	Water parsley	OBL				+	+			+		+
Polygonum amphibium	Water smartweed	OBL				+						
Potentilla palustris	Marsh cinquefoil	OBL				+	+					
Scirpus acutus	Hardstem bulrush	OBL				+						
Sparganium emersum	Narrow-leaf burreed	OBL				+	+					
Grasses												
Agrostis alba	Red top	FACW		+								
Alopecurus geniculatus	Water foxtail	OBL		+								
Alopecurus pratensis	Meadow foxtail	FACW		+								
Dactylus glomerata	Orchard grass	FACU	+									
Festuca arundinacea	Tall fescue	FAC	+									
Lotus corniculatus	Birdsfoot trefoil	FAC		-								
Phleum pratense	Timothy	FAC	+									
Trifolium pratense	Red clover	FACU	-									

Data compiled by Parametrix
[1]The symbols "+" and "-" indicate the relative abundance of selected species in each plant association.

These plant associations were selected because they are adapted to the expected soil moisture during normal rainfall years and they provide a range of moisture tolerance during unusually dry or wet years. The general relationship of wetland classes to site hydrology is illustrated in Figure 3.5-8. Plant species were also selected based on their value as food sources for wildlife. The hybrid cottonwood, *Populus trichocarpa x deltoides*, would be planted to provide rapid development of canopy cover and greater structural diversity in the early years of mitigation plan implementation. Plantings of native *Populus trichocarpa* would be intermingled with hybrid poplar, but these native trees would be spaced so as not to be shaded excessively by the hybrid poplar. Since the hybrid cottonwood is sterile, future colonization of cottonwood on the site would primarily be by native cottonwood; some colonization by hybrid cottonwood by fallen branches and suckering may occur.

The five forested wetland plant associations and one shrub wetland plant association used would both correspond to slight hydrologic variations in the wetland and provide habitat diversity. Selected species, including red alder, black cottonwood, Oregon ash, western red cedar, willow, salmonberry, and red-osier dogwood, are typical of lowland Puget Sound wetlands. Each association includes species that tolerate the seasonally saturated soil conditions expected on the site. However, some associations, such as the black cottonwood/willow- and Oregon ash-dominated associations, include a higher proportion of FACW species that are particularly adapted to wet soils. These associations are identified for planting adjacent to seasonally and permanently flooded emergent areas.

Following site grading, a wetland hydroseed mix that includes a small percentage of a sterile hybrid grass would be planted over all areas identified for shrub and forested wetland plantings. The hybrid grass would provide rapid soil stabilization while the slower-growing native grasses establish. Planting of overstory trees and shrubs in forest and shrub plant associations would occur during the first fall or spring season following site grading, when the soil moisture is near the ground surface and temperature conditions are favorable for establishing roots and plant growth. Two- to three-year old branched seedlings at least 24 inches tall would be planted at a density of approximately 250 stems per acre (or 13 ft on center). This density exceeds the final performance standard of 200 trees per acre (refer to Table 3.3-2), allowing for some natural mortality during the early years. Shrub understory species in the forested areas would be planted in patches at densities of 300 plants per acre to mimic their natural occurrence patterns. Shrubs in the shrub wetland area would be planted at a density of 400 plants per acre. Part of the site would be graded to a relatively abrupt shoreline, eliminating the shrub wetland zone between elevation 41.5 and 42 ft, thereby providing forested wetland cover and overhanging vegetation adjacent to permanently flooded emergent areas. Understory development in the forest and shrub wetland areas is expected to occur through colonization from adjacent seed sources.

Emergent wetlands would be planted with native emergent species common in the Green River Valley and the northern Puget Sound region. Since wetland hydrology is designed to create both seasonally and permanently flooded areas, selected plants that are tolerant of extended flooding and soil saturation would be established. Species would include water parsley, slough sedge, narrow-leafed burreed, hardstem bulrush, and common spike rush. The typical growth pattern for emergent marsh plants is in monotypic patches with some interspersion in open, less densely vegetated areas and proposed planting would mimic this pattern. Planting shoots with rhizomes 18 inches on center in monotypic stands of varying size and seeding a mix of emergent species in the areas between patches should achieve that result. Because ponding in emergent areas is expected well into the early summer, planting of emergent species would occur during the fall months when soils are becoming saturated but before water levels reach their winter maximum.

All vegetated upland areas disturbed during mitigation wetland construction would be hydroseeded using native upland grasses that typically occur in open fields in the area (Table 3.5-2). Following hydroseeding, forested buffers would be planted bordering the northern and southern boundaries of the mitigation wetland. These boundaries are most susceptible to outside disturbance from ongoing agricultural activities and from potential future urban development. Trees and shrubs would be planted at densities sufficient to attain the stem density performance standards identified for forested wetland habitat. Buffer plantings are not proposed for the eastern boundary, which is to remain undeveloped. A narrow strip of land to the east of the site, adjacent to the Green River, is owned by King County. Approximately 120 ft of open grassland would remain as an open space between the edge of the constructed mitigation wetland and the King County property boundary. Land along the western edge of the site is delineated as emergent wetland and would remain undeveloped.

3.6 MONITORING PLAN

The mitigation project would be monitored for a 10-year period, with monitoring focusing on the physical and ecological data necessary to determine whether performance standards for the project (Table 3.3-2) are being achieved. Monitoring reports would summarize the ecological condition of the wetland, and the degree of compliance with performance standards; as necessary contingency actions would be recommended. The first phase of preparing the monitoring report would be to complete an as-built report, as described below; Section 3.6.2 describes the activities and schedule during the monitoring period.

3.6.1 As-Built Report

An as-built wetland report that describes the mitigation as constructed and planted would be prepared to define the baseline conditions for measuring progress toward the defined mitigation goals and final performance standards. The as-built report also establishes all sampling locations for future monitoring activity. Any later significant deviations from plan documents would be noted, and the significance of these deviations evaluated and coordinated with the U.S. Army Corps of Engineers.

A detailed wetland map would be prepared from field surveys. Baseline observational data, with which future monitoring can be compared, would also be collected and mapped, as appropriate. These baseline data would include:

- topographic mapping
- locations of major plant community boundaries
- locations of surface water

The topographic survey is used to evaluate the amount of land added to the 100-year floodplain and to determine whether the performance standard for floodplains has been achieved.

For the as-built report, a staff gage assembly would be installed within the pond area, with the lower portion extending to the sediment surface of 40.5 ft. The staff gage would be mounted on a treated 4-inch x 4-inch post, and its location surveyed and mapped.

A visual site inspection to describe the types, condition, and locations of planted species in the wetland would be part of the as-built report. For each planting area, observations would include species, typical size and approximate ranges in size, the approximate spacing of plants, and their location relative to elevation and ground or surface water levels. In addition, the edge between wetland classes would be staked and mapped.

During future monitoring efforts (Section 3.6.2), transects would be used for sampling plant species composition, cover, and growth, allowing comparative analysis of these parameters over time. These transects, which would be field-staked during the as-built survey, must be randomly selected to eliminate sampling bias.

Photographs taken during the monitoring period can qualitatively document plant community development in both the wetland and adjacent buffer. Photographs would be used, therefore, to show the extent and rate of plant height and cover. Photographs can supplement quantitative vegetation characterization from the permanent transects. Photographic points established along transects and other appropriate viewpoints would then be described and labeled on maps.

An as-built report summarizes the existing wetland condition when construction is completed. The report would include descriptions of the aerial extent of the wetland (and each vegetation zone planted) relative to mitigation goals, the hydrologic condition of each wetland planting area, and the relationship between each planting zone and observed soil moisture. These wetland features would then be compared to those established as design criteria for the wetland (Table 3.3-1). Any deviations from design parameters would be noted and discussed, including the anticipated significance of any deviations on the eventual development of a functioning wetland system.

3.6.2 10-Year Monitoring Plan

Monitoring activities would focus on the collection of baseline hydrology, vegetation, and wildlife data to evaluate wetland function and compliance with the performance standards summarized in Table 3.6-1. Monitoring would also include photographic documentation of site features and the development of habitat on the site.

Table 3.6-1. Off-site wetland monitoring methods and reporting schedule.

Design Objective	Performance Standard	Method	Month	Frequency
Forested Wetland Vegetation	In-kind replacement ratio	Aerial photographic or ground-based mapping	May	As-built, Years 3, 5, 10
	Species Composition	Walk-through surveys and plot or belt transect sampling to document all plant species present	July	Years 1, 2, 3, 5, 7, 10
	Tree and shrub density	Measure by line-intercept method along transects	July	Years 3, 6, 10
	Plant growth	Walk-through surveys to estimate annual shoot growth and survival rates	July	Years 1, 2, 3, 5
	Vegetation structure	Describe from walk-through surveys, incorporating data from above analysis as available	July	Years 1, 2, 3, 5, 7, 10
Shrub wetland vegetation	In-kind replacement ratio	Aerial photographic or ground-based mapping	May	As-built, Years 3, 5, 10
	Species Composition	Walk-through surveys to document all plant species present	July	Years 1, 2, 3, 5, 7, 10
	Shrub density	Measure by line-intercept method along transects	July	Years 3, 6, 10
	Plant growth	Walk-through surveys to estimate annual shoot growth and survival rates	July	Years 1, 2, 3, 5
	Vegetation structure	Describe from walk-through surveys, incorporating data from above analysis as available	July	Years 1, 2, 3, 5, 7, 10
Emergent wetland vegetation	In-kind replacement ratio	Aerial photographic or ground-based mapping	May	As-built, Years 3, 5, 10
	Species Composition	Walk-through surveys to document all plant species present	July	Years 1, 2, 3, 5, 7, 10

Table 3.6-1. Off-site wetland monitoring methods and reporting schedule (continued).

Design Objective	Performance Standard	Method	Month	Frequency
	Herbaceous plant coverage/density	Measure by plot sampling method along transects	July	Years 3, 6, 10
	Plant growth	Walk-through surveys to estimate annual shoot growth and survival rates	July	Years 1, 2, 3, 5
	Vegetation structure	Describe from walk-through surveys, incorporating data from above analysis, as available	July	Years 1, 2, 3, 5, 7, 10
Wetland hydrology	Soil saturation	Depth from the soil surface to groundwater measured at permanent sampling stations in forested, shrub, and emergent wetland zones	Monthly February, June	Years 1, 2, 3; Years 5, 10
	Surface Water Depth	Water depths measured at permanent sampling stations in shrub and emergent wetland zones	Monthly February, June	Years 1, 2, 3; Years 5, 10
Flood storage capacity	Topography	Analysis of as-built topographic survey		Year 0
Wildlife	Habitat Structure	Analysis of hydrologic and vegetation data from forested, shrub, and emergent wetlands	February, June	Years 1, 2, 3, 5, 10
		Description of habitat structure from walk-through surveys	February, June	Years 1, 2, 3, 5, 10
	Wildlife usage	Conduct surveys to record wildlife species and activities on-site.	January, April, June, November	Years 1, 2, 3, 5, 10
Long-term protection	Buffers, adjacent land uses	Description of buffer vegetation and adjacent landuses, including proximity and screening.	June	Years 5, 10
	Public access trails	Description of on-site trails and adjacent vegetation, including proximity and screening from permanently or seasonally flooded wetland habitat	June	Years 5, 10

Data compiled by Parametrix

under various hydrologic regimes, and other general observations relevant to mitigation design and implementation.

Most monitoring activities would be completed along the permanent transects and fixed points established and marked during the as-built survey; however, as determined in the field, additional monitoring may be needed to document unique conditions not present at pre-established sampling locations. All monitoring uses standard ecological techniques to sample, measure, or describe vegetation, hydrologic, and wildlife habitat conditions. These techniques include walk-through surveys, line-intercept sampling along transects (Canfield 1941), plot sampling (Daubenmire 1959), and wetland delineation (FICWD 1989; Environmental Laboratory 1987).

3.7 SITE PROTECTION

The Port of Seattle and the city of Auburn are currently negotiating the terms of site protection. A number of alternatives are being considered. The mitigation project would be protected against further development in perpetuity.

3.8 MAINTENANCE AND CONTINGENCY PLAN

The mitigation wetland is designed to achieve the final performance standards without ongoing maintenance. Wetland hydrology is dependent on rainfall and plant communities are adapted to the designed hydrologic regime. Supplemental irrigation during the first two seasons following planting may be necessary to assure plant establishment. This maintenance activity would depend on rainfall quantities and on ongoing planning for a regional storm water facility which could supplement summer water levels in the wetland. The monitoring activities outlined in Table 3.6-1 would identify conditions requiring contingency actions, which are outlined in Table 3.8-1. Contingency actions would be implemented in coordination with the U.S. Army Corps of Engineers.

Since reed canarygrass is present in adjacent wetland areas, and this aggressive species could invade the wetland through seed dispersal, maintenance actions may be require to control its spread. These actions could include periodic mowing, treatment with herbicide, and reseeding with native wetland grasses, or more extensive restoration of the on-site wetland that would remain.

Table 3.8-1. Contingency measures for the Sea-Tac Airport off-site wetland mitigation project.

Design Feature	Monitoring Year(s)	Condition	Contingency Action
Forested and Shrub Wetland Plantings	1-5	>80% survival of planted stock	None
		60 - 80% survival	Evaluate reason(s) for mortality, and replant to achieve performance standard.
		<60% survival	Evaluate reason(s) for mortality; consider species suitability for site conditions; replant with the same or alternate species.
	3-5	Average stem elongation (by species) of at least 2 inches	None
		Average stem elongation (by species) less than 2 inches	Evaluate potential reasons for lack of plant growth; consider fertilization; consider species suitability for the site; or increase planting quantities to achieve performance standard.
	5-10	Tree density at least 200 stems per acre.	None
		Tree density less than 200 stems per acre.	Evaluate reason(s) for mortality and replant to achieve performance standard.
		Presence of seed and/or fruit production on shrub species	None
		Lack of seed and/or fruit production on shrub species.	Evaluate potential reasons for lack of seed and/or fruit production; evaluate health and vigor; consider fertilization.

Table 3.8-1. Contingency measures for the Sea-Tac Airport off-site wetland mitigation project (continued).

Design Feature	Monitoring Year(s)	Condition	Contingency Action
Emergent Wetland Vegetation	1	Total cover by emergent wetland species at least 20%, and at least 10% cover by the emergent wetland species planted	None
		Total cover by emergent wetland species less than 20%, and less than 10% cover by the emergent wetland species planted	Re-evaluate the suitability of the plant species for site conditions and re-establish if necessary. Consider use of fertilizers or alternate species.
	2	Total cover by emergent wetland species at least 40%, and at least 20% cover by the emergent wetland species planted	None
		Total cover by emergent wetland species less than 40%, and less than 20% cover by the emergent wetland species planted	Re-evaluate the suitability of the plant species for site conditions and re-establish if necessary. Consider use of fertilizers or alternate species.
	3 - 10	Total cover by emergent wetland species at least 70%, and at least 30% cover by the emergent wetland species planted	None
		Total cover by emergent wetland species less than 70%, and less than 30% cover by the emergent wetland species planted	Re-evaluate the suitability of the plant species for site conditions and re-establish if necessary. Consider use of fertilizers or alternate species. When invasive species (reed canarygrass) represent greater than 20% cover, control of this species by herbicide will be evaluated.

Table 3.8-1. Contingency measures for the Sea-Tac Airport off-site wetland mitigation project (continued).

Design Feature	Monitoring Year(s)	Condition	Contingency Action
Hydrologic Regime	1-10	In forested areas, saturation within 12 inches of surface late December to April (normal rainfall years) In shrub areas, saturation within 6 to 12 inches of surface year-round (normal rainfall years); flooded up to 6 inches deep between December and May In emergent areas, saturation within 6 inches of surface year-round (normal rainfall years); flooded permanently below 41-ft elevation; flooded up to 24 inches deep above 41 -ft elevation November to June	Evaluate reasons for non-attainment. Possible solutions include modification of off-site drainage to wetland, revision of planting plan to correlate to the hydrologic regime, or addition of waterlevel control structures to regulate water levels.
Forested Buffer		[same standards as forest/shrub wetland]	[same contingency actions as forested wetlands]
Flood Storage Capacity	1-10	A minimum of 29 acres below 45-ft elevation; direct connection to 100-yr floodplain	Maintain hydraulic connection to 100-yr floodplain.

Data compiled by Parametrix

4. MILLER CREEK

4.1 ECOLOGICAL ASSESSMENT OF IMPACT SITE

The Miller Creek basin, located in southwest King County, is bordered on the east and southeast by Sea-Tac Airport; the city of Normandy Park lies to the south, the plateau above Seahurst to the west, and the hill north of Arbor Lake to the north. The basin encompasses about 8 mi^2 and includes a small portion of Sea-Tac Airport, as well as parts of the cities of SeaTac and Burien. Sea-Tac Airport covers an estimated 5 percent of the entire basin. The Miller Creek watershed consists of tributaries that originate at Arbor, Burien, and Tub lakes; surface water and seep drainages from the north end of Sea-Tac Airport; and overflows from Lake Reba and Lora Lake. The creek generally flows south and southwest toward Puget Sound.

4.1.1 Stream Classification

The lower reaches of Miller Creek are Class II salmon-bearing waters, as defined by the Washington State Department of Fish and Wildlife (WDFW). However, the upper reaches (starting about 0.2 mi upstream of Southwest 160th Street) are believed to be inaccessible to anadromous salmonids (Shapiro 1995e). The other tributary streams that flow through or adjacent to the study area are Class III or unclassified reaches that function primarily as drainage or groundwater conveyances. Class III streams are classified according to their intermittent or ephemeral characteristics during normal rainfall years. The watershed is generally classified by Ecology as having Class AA (extraordinary) water quality. Storm water runoff from residential, commercial and agricultural properties has contributed to water quality degradation. As a result, Miller Creek fails to meet many of the state water quality standards (Landrum & Brown 1995).

Water quality in the basin has degraded as a result of pollutants commonly found in urban storm water runoff. Nutrients, organics, metals, fecal coliform bacteria, and suspended solids have contributed to occasional violations of Class AA water quality standards and federal water quality criteria. In addition, occasional violations of Class AA water standards for pH, dissolved oxygen, and ammonia have also occurred in the basin (Landrum & Brown 1995).

4.1.2 Primary Uses/Function in the Watershed

Most of the 5,000-acre Miller Creek watershed is fully developed with residential and commercial properties. Approximately 60 percent of the land use in the basin is residential, 20 percent is commercial, and the remaining 20 percent is open space or forested. The single largest commercial facility in the watershed (approximately 5 percent of the area) is Sea-Tac Airport. Other commercial facilities are scattered along Des Moines Way, Ambaum Boulevard, and First Avenue South. Some agricultural uses are also found in the upper watershed, including the impact site. Although urbanization has significantly altered the stream and riparian habitat, these areas continue to support some fish and wildlife species.

4.1.3 Existing Fish Habitat

Historically, Miller Creek supported anadromous fish runs of coho and chum salmon and sea-run cutthroat trout, as well as resident populations of pumpkinseed sunfish, sculpin, and cutthroat trout (Landrum & Brown 1995). The creek currently supports a small coho salmon run that is maintained by annual releases of hatchery-reared fingerlings raised by the Des Moines Chapter of Trout Unlimited (Shapiro 1995e). The last WDFW-sponsored spawner survey in 1985 did not observe any spawning coho. However, the Des Moines Salmon Chapter of Trout Unlimited reported about 91 fish in a recent coho spawner survey. No comprehensive population study has been conducted on Miller Creek.

Residential development in the watershed has resulted in a general deterioration of fish habitat owing to the removal of native riparian vegetation, stream channelization and bank armoring, filling of riparian wetlands, reducing the availability of large organic debris, and increasing the non-point source pollution loading. The expansion of impervious surface area in the basin has also led to increased runoff volumes and velocities; the result has been increased bank erosion, downcutting, landslides, and debris jams. These factors have contributed to a general lack of (1) instream and overhead cover, (2) available low- and high-flow habitat or refugia, (3) available spawning habitat in the basin, (4) habitat complexity, and (5) high-quality water (KCSWM 1987; Landrum & Brown 1995; Shapiro 1995a).

In addition to the deteriorated habitat conditions in the basin, several natural and manmade barriers appear to be limiting anadromous fish species access to the upper basin. The most prominent barrier on Miller Creek is an 8-ft waterfall about 0.2 mi upstream of Southwest 160th Street. Other potential barriers in the basin include several corrugated metal and concrete box culverts (Shapiro 1995a). These seasonal or year-round barriers appear to limit upstream habitat use to non-salmonid resident fish species, such as pumpkinseed sunfish and sculpin (Shapiro 1995e).

In addition to these barriers, habitat availability may be contributing to the current fish distribution pattern. Shapiro (1995a) found suitable coho salmon spawning gravel limited to the area downstream of First Avenue South, while suitable cutthroat spawning habitat was scattered in small patches between South 156th Way and First Avenue South. Areas upstream of First Avenue South, however, consisted predominantly of fine silt and sand substrate, which is more suitable habitat for the non-salmonid fish species that occur there.

King County Surface Water Management (KCSWM; 1987) reported that natural, unaltered stream reaches in the basin are essentially nonexistent, and that major portions of the mainstem and all tributary streams are channelized or otherwise modified. The mainstem section that would be relocated as a result of the proposed airport development project is a low-gradient, channelized stream, with low-density riparian vegetation, no large woody debris, and limited habitat complexity. This reach is dominated by slow-flowing water and shows signs of excessive sedimentation. This sedimentation appears to be at least partially caused by agricultural runoff. Shapiro (1995e) estimated that some 10 tons of sediment are transported to the creek annually from the adjacent 11 acres of agricultural land. The factors mentioned contribute to the lack of high-quality fish-rearing pools in the reach. Such pools are important over-wintering habitats that provide refuge for fish during high-flow events (Shapiro 1995a).

Several small tributaries originating from groundwater seeps under the runway flow west to Miller Creek. These reaches are intermittent surface and groundwater conveyance ditches that do not appear to provide fish habitat at any time (Shapiro 1995b). The habitat in these reaches consists of a series of small, shallow, runs and riffles with occasional pocket-water. During winter flow periods, these tributary reaches consist of shallow rivulets that are approximately 1-3 inches deep and typically less that 1 ft wide.

4.1.4 Hydrology

The addition of fill and impervious surface areas as a result of the proposed Master Plan Update improvements would decrease the amount of rainfall infiltration in soils (groundwater recharge) and increase the volume and flow rate of storm water runoff in the basin. Unless mitigated, these changes are expected to cause increased flooding, erosion, and instream habitat degradation in areas downstream of the study area. These problems already occur in the area due to previous basin development.

KCSWM (1987) estimated that 40 percent of the basin's surface area was impervious in 1986; an increase to 50 percent was predicted when the area was fully developed. Increased runoff rates and volumes resulting from urbanization and development in the watershed have contributed to erosion and downcutting in the steep ravine areas, and sedimentation and aggradation in the low-gradient areas (Shapiro 1995e). The impervious surface areas also limit the groundwater recharge in the area, resulting in less groundwater seepage during low-flow periods.

Since 1991 (KCSWM 1994) KCSWM has monitored flow rates at the outlet of Lake Reba. The available flow data provide a good record of base flows, normal wet and dry season flows, and annual peak flows. Stream flow rates are typically highest between October and April and lowest between May and September (Landrum & Brown 1995). Montgomery Water Group (1995) modeled hydrologic characteristics in the basin and found that in some years no flow occurs in the upper watershed areas during portions of the summer. They also reported that summer flows only exceed 0.5 ft^3 per second (cfs) about 10 percent of the time. A range of flow rates for channel design have been determined from these data sets (Tables 4.1-1 and 4.1-2).

Table 4.1-1. Estimated base flow rates (cfs) at the Lake Reba outlet structure.

Base Flow Rates	Flow Rate (cfs)
Dry Season (May – September)	0.5
Wet Season (October – April)	5.0
Approximate Annual Peak	40.0

Source: KCSWM 1994

In addition to monitored flows, a detailed hydrologic study was prepared (Montgomery Water Group 1995) that includes a peak flow rate for flood frequencies up to the 100-year flood (Table 4.1-2). The 2-year-flood peak flow rate is estimated at about 75 cfs (just downstream of the Lake Reba detention facility), and the 100-year flow rate is about 175 cfs.

Table 4.1-2. Flood frequency estimates - Miller Creek at the Lake Reba control structure.

Return Period (years)	Peak Flow Rate (cfs)
1.01	21
1.11	40
2	75
10	125
20	141
50	161
100	175

Source: Montgomery Water Group 1995

4.1.5 Channel Configuration

Miller Creek from the Lake Reba detention facility outlet to South 156th Way is not a natural stream; the creek has been dredged and straightened for farmland reclamation and wetland drainage. Land contours, soil types, and flat profiles indicate that the study segment was historically a poorly drained wetland that overflowed to the south where Miller Creek follows a topographic incision. Ditches were constructed to connect the upper watershed, Lake Reba, and Lora Lake to Miller Creek south of the study area. The channel currently overflows its banks with at least a 2-year frequency with full flow velocity of 1.7 ft per second (Shapiro 1995e). Frequent flooding is primarily the result of inadequate channel capacity, in part because of the flat channel slope. A side channel in the study area runs parallel to the main channel, providing positive drainage for the farm fields. The side channel is not a true tributary, as it does not drain runoff from a subbasin area nor does it provide additional channel capacity to the main channel. Rather, its function is to provide positive drainage for a portion of the relatively flat farm located in the study area.

Miller Creek through the study area is approximately 4 to 10 ft wide at the bottom and two ft deep below the outfall of the Lake Reba detention facility. Large rocks line the edge of the creek in the upper segments near Lora Lake, and the channel has a very silty bottom. Red alder saplings shade the stream, and the banks are vegetated with nightshade and reed canarygrass. Stream floodplains in the lower segments are tilled and farmed.

4.1.6 Floodplain

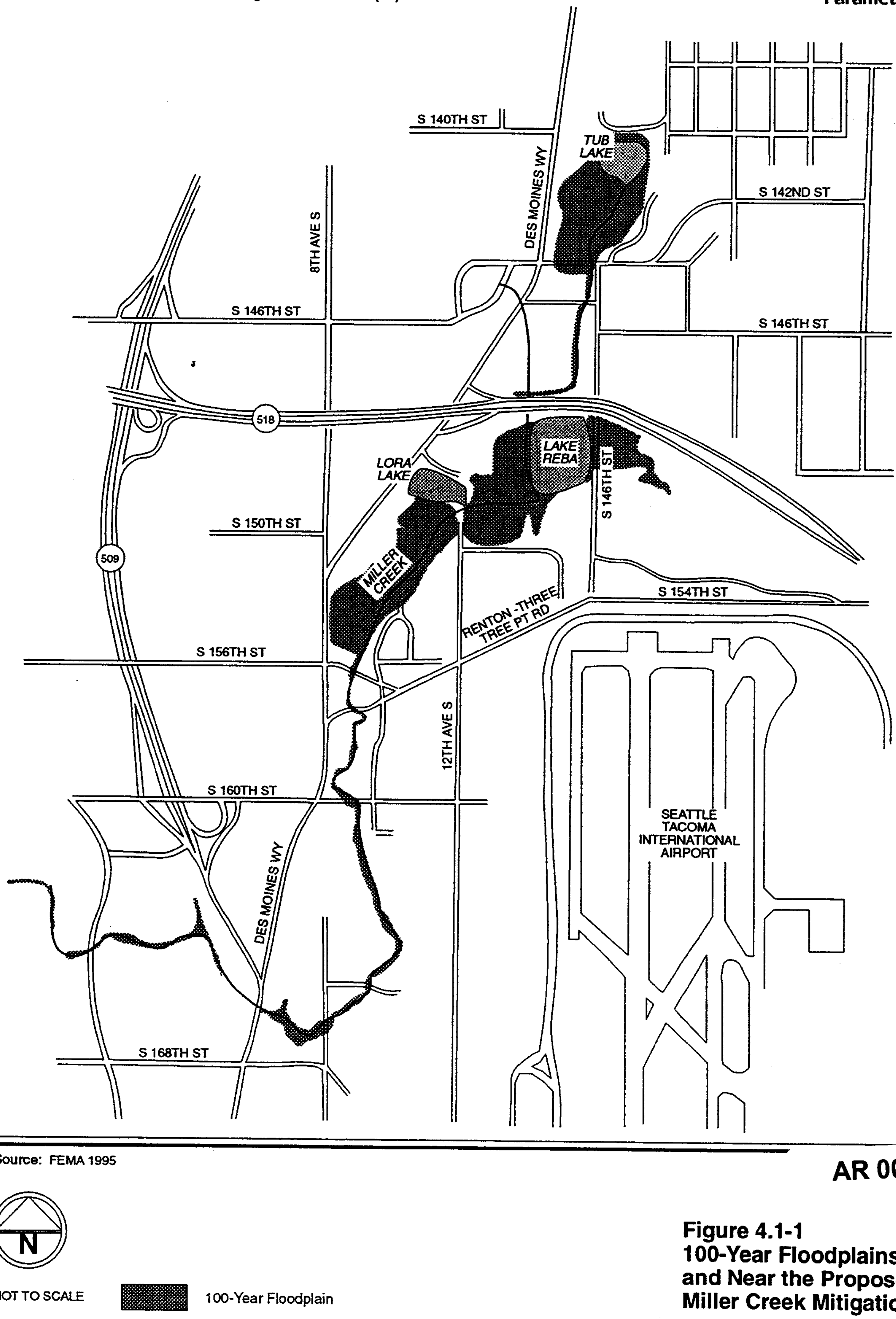
Existing floodplains have been significantly altered by urbanization and agricultural development in the Miller Creek basin. Development activities that have contributed to current floodplain conditions include the filling of wetlands, removal of riparian vegetation, and stream bank armoring. These activities have reduced both stream channel and floodplain capacities. In addition, the construction of roads, residences, and commercial facilities have increased storm water runoff rates and volumes. These factors have contributed to an increased flooding potential in the basin (Landrum & Brown 1995).

The 100-year floodplain in the vicinity of the channel relocation is quite extensive. The wetland ponding and poor drainage that existed prior to the land drainage activities are evident with the 100-year floodplain estimated by the Federal Emergency Management Agency (FEMA) (Figure 4.1-1). The approximate 100-year flood elevations, determined by FEMA as part of their study, vary from 266 ft at the Lake Reba Detention Facility outlet to 265 ft at the downstream end of the proposed stream relocation.

Without mitigation, construction and operation of the proposed Master Plan Update improvements could result in significant adverse floodplain impacts, including reductions in the 100-year floodplain area and storage capacity, increased storm water runoff rates and volumes, and increased flood potential in downstream areas. Ecology floodplain development standards and floodway management requirements prohibit reductions in floodplain area or storage capacity, or significant increases in peak flow rates. Therefore, the implementation of the mitigation plan is expected to result in no significant floodplain or flooding impacts.

4.1.7 Existing Riparian Vegetation

The riparian areas associated with Miller Creek and its tributaries are primarily classified as forested wetlands. Both upland and wetland plant communities are dominated by an overstory of Western red cedar, red alder, black cottonwood, and Pacific willow trees. The understory



Source: FEMA 1995

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Figure 4.1-1
100-Year Floodplains On and Near the Proposed Miller Creek Mitigation Site

vegetation is dominated by Himalayan blackberry, Douglas spirea, salmonberry, lady-fern, field horsetail, cattail, soft rush, slough sedge, burreed, reed canarygrass, mixed grasses, and creeping buttercup (Landrum & Brown 1995; Shapiro 1995e; and Parametrix 1991).

4.2 CREEK MITIGATION GOALS, OBJECTIVES, AND PERFORMANCE STANDARDS

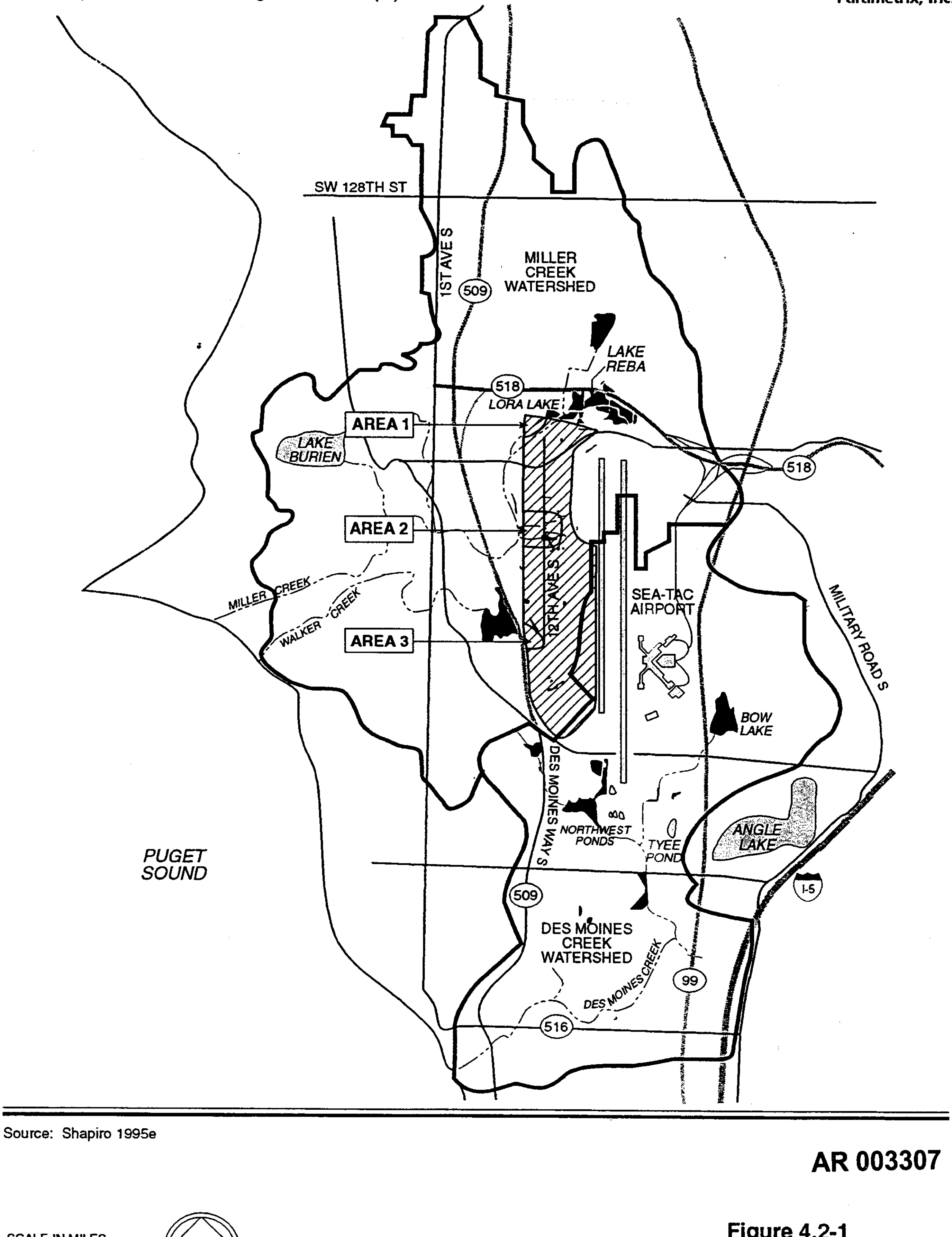
4.2.1 Goals and Objectives

The primary mitigation goal is to replace the basic characteristics and functions of the three portions of Miller Creek and its tributaries (Areas 1, 2, 3) that will be affected by the proposed airport improvements (Figure 4.2-1). Area 1 is located northwest of the current runway at the outlet of Lake Reba. Areas 2 and 3 are drainage tributaries flowing west from the runway embankment to Miller Creek. The impacts to Area 1 require relocating approximately 1,080 ft of Miller Creek. Areas 2 and 3 will be affected by the filling of the drainage channels from the western edge of the existing fill slope to the western edge of the proposed fill slope.

Miller Creek in Area 1 is not a natural stream because the creek has been dredged and straightened for farmland reclamation and wetland drainage. Land development, roadway construction, and past airport development have also altered the segment. Replicating the marginal existing stream habitat with the proposed mitigation channel is insufficient. The goal of the Miller Creek relocation (Area 1) is to provide a new stream channel with enhanced habitat features having at least the same length as the existing ditch.

A farm ditch located in the impact area flows parallel to Miller Creek for approximately 800 ft. The ditch provides positive drainage for the westerly portion of the farm, connecting to the main channel near South 156th Way. A small segment of the side channel (approximately 250 ft) would be impacted by the fill; however, because this segment is at the upper end of the side channel, drainage and conveyance would not be affected. No habitat would be impacted, since the ditch flows intermittently in response to rain, and has little riparian habitat due to farming. For these reasons, no mitigation is proposed.

Area 2 consists of two small intermittent ditches with an indication of minor seepage. Area 3, the headwater of Walker Creek, contains a short segment of drainage channel. All three tributaries have been affected by existing airport drainage, perimeter road crossings, or channelization. The mitigation goal for Areas 2 and 3 is replacing the drainage function of the tributaries.



Source: Shapiro 1995e

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Figure 4.2-1
Areas Affected by the Proposed Earthwork

4.2.1.1 General Mitigation Objectives

The new Miller Creek channel would be constructed near the lowest path through the broad flat trough that defines the creek floodplain in the project area, with the channel edge offset from the proposed fill a minimum of 25 ft to provide a buffer. Channel slope and minimum flow depth would influence final channel alignment. The new creek would connect with the existing Miller Creek channel downstream at the earliest possible point to minimize stream relocation impacts. Channel relocation guidelines presented below may vary due to the limited space available between Lora Lake and the proposed fill area. High flows would be diverted through Lora Lake in the upper segments of the proposed Miller Creek channel.

Careful consideration of the benefits that Miller Creek and the three tributaries now provide must be given when determining the required features for the post-mitigation stream. Streams and waterways can provide many important functions such as conveying surface water and storm water, including flood waters, and providing in-stream and riparian habitat for fish and other water-dependent animals.

The proposed mitigation plan must ensure that present uses are not reduced and that other beneficial uses be included or enhanced. Beneficial use criteria provide design considerations and require consistency with the overall mitigation plan. Goals are prioritized from the most critical function that the existing channel provides to enhancements that would improve channel habitat. A list of impact compensation goals describes the decision-making priorities for the proposed relocated creek. If goals conflict, the higher priority takes precedence.

Miller Creek Goal 1:	The stream and tributaries must continue to provide base flow conveyance functions
Miller Creek Goal 2:	The new Miller Creek channel should provide improved fish habitat
Miller Creek Goal 3:	The channels must accommodate peak flows up to the 100-year flow; no net 100-year floodplain storage lost
Miller Creek Goal 4:	Minimum channel flow velocity should minimize fine sediment deposition
Miller Creek Goal 5:	The channels must replace or enhance riparian habitat
Miller Creek Goal 6:	The channels cannot include expansive, long-standing water pools or wetlands that could potentially attract wildlife
Miller Creek Goal 7:	The proposed Miller Creek corridor should accommodate passive recreational uses, such as walking trails

Three Miller Creek tributaries would be impacted by the fill for the proposed third parallel runway. All three would have fill placed from the outlet to the base of the proposed fill slope. The channels provide flow conveyance during storms, and minor seepage is collected as the channels drop down the bluff. Beneficial uses include flow conveyance, base flow seepage, water quality benefits from natural filtration, and limited habitat. Mitigating fill impacts would include:

Tributary Goal 1:	The tributaries must continue to provide adequate flow conveyance.
Tributary Goal 2:	The tributaries would collect seepage to maintain base flows.
Tributary Goal 3:	The new tributary must provide an open channel of equivalent length as the existing tributary.

Specific Miller Creek and tributary design standards are described Tables 4.2-1 and 4.2-2.

4.2.1.2 Appropriate Habitat

Design and implementation of a mitigation program for the airport is especially challenging because of flight safety issues. Collisions between birds and aircraft are a serious safety issue. Open-water areas, wetlands, and tall trees can attract waterfowl, small flocking birds (such as starlings), and raptors that may feed on small resident mammals. Large fish populations can also attract many birds and small mammals to places where shorelines and open-water fish habitat are accessible. The closer these habitat features are to airport runways, the greater the potential for interference with aircraft.

That portion of Miller Creek lying within the proposed study area is characterized by sections of lower-quality instream and riparian habitat. Stream channelization, streambank armoring, riparian vegetation removal, filling of riparian wetlands, poor culvert design and installation, increased development, and non-point source pollution have degraded stream and riparian habitat in several locations in the watershed (KCSWM 1987; Shapiro 1995e). These conditions presently constrain aquatic production in the Miller Creek basin.

Because the proposed airport improvements would not change anadromous fish passage conditions, and because wildlife attractants are not encouraged (see Section 2.2), the mitigation plan for Miller Creek does not include measures to remove existing anadromous fish barriers. However, the plan does include design features that would enhance habitat for resident fish by using performance standards developed for the more environmentally sensitive salmonid species. Because resident fish typically do not experience the dramatic seasonal population changes that occur with anadromous fish, there is little likelihood that a wildlife attractant would be created by providing higher-quality resident fish habitat.

Table 4.2-1. Mitigation goals, design objectives, design criteria, and final performance standards for Miller Creek.

Goal / Design Objectives	Design Criteria	Final Performance Standard
Miller Creek Goal 1: The stream and tributaries must continue to provide base flow conveyance functions		
Provide minimum flow depth to prevent fish stranding and water quality problems	Design a natural channel assuming a gravelly or stony bottom and a Manning's *n* of 0.035	Minimum average flow depth is 0.25 ft (at 0.5 cfs)
	Construct vertical channel side slopes from the bottom up to 0.5 ft deep; construct side slopes at 1:1 or flatter (typical) from 0.5 to 1.0 ft to provide capacity for wet season base flow	Approximate wet season (October – April) average base flow (5.0 cfs) depth is 1 ft
	Set channel slope to provide minimum and maximum velocity criteria (Goal 4)	
	Adjust channel bottom width for minimum depth	
Maintain existing hydrology from Lora Lake	Construct overflow structure from Lora Lake that replicates the existing lake outflow hydrology	Lora Lake outflow structure replicates the existing discharge hydrology
Miller Creek Goal 2: The new Miller Creek channel should provide enhanced fish habitat		
Provide enhanced fish habitat without fish passage barriers	Provide minimum flow depth (Goal 1)	New channel meets design criteria (Goal 1)
	Provide a natural channel configuration, 0.5 ft vertical slopes, 1:1 slopes from 0.5 ft to 1 ft depth (Goal 1)	
	Provide habitat features, including in-stream features such as deflectors and overhanging logs as needed to maximize available habitat	Stream habitat features are stable
	Provide channel substrate that enhances habitat; design channel to manage flow velocity that is consistent with substrate types (Goal 4)	
	Reduce silting, sedimentation, and scouring by meeting minimum and maximum average flow velocity standards	

Table 4.2-1. **Mitigation goals, design objectives, design criteria, and final performance standards for Miller Creek (continued).**

Goal / Design Objectives	Design Criteria	Final Performance Standard
Miller Creek Goal 3: The channels must accommodate peak flows up to the 100-year flow; no net 100-year floodplain storage lost		
Accommodate the 100-year peak flow	Do not confine or constrict 100-year flood flows in the new channel; flows in excess of the channel design will freely overflow the channel into the flood plain	The 100-year flood stage outside the study area is not changed by more than 0.1 ft
Allow no net 100-year floodplain storage loss in the project area	Mitigate 100-year floodplain storage by providing lost storage compensation.	The 100-year flood stage outside the study area is not changed by more than 0.1 ft
Limit channel scouring for the 100-year flow	Channel velocity cannot exceed the gravel movement velocity for the 100-year flow (Goal 4)	Channel substrate present; no bare scoured channel sections in excess of 25 ft
Miller Creek Goal 4: Minimum channel flow velocity should minimize fine sediment deposition		
Minimize sedimentation with minimum flow velocity	Adjust channel slope, by channel segment, to provide minimum dry season base flow velocity. Channel flow velocity for seasonal base flows cannot be less than the silt transport velocity (0.7 ft/sec) Adjust channel bottom width to achieve minimum velocity	Minimal summer season sedimentation in riffles or runs; minimal winter season sedimentation in riffles, runs or gravel substrate
Minimize channel scouring with a maximum design flow velocity	Channel flow velocity cannot exceed the gravel movement velocity (4 ft/sec) for the 100-year flow Increase channel capacity above 0.5 ft depth (up to 2 ft depth) to reduce peak flow channel velocity	Channel substrate present; no bare scoured channel bottom sections in excess of 25 ft
Divert high flows around channel Segment 1	Construct a stream diversion structure to reduce flows in channel Segment 1 to peak annual flow rate (40 cfs) for the 100-year peak flow	Peak flow less than 50 cfs in Segment 1 during the 100-year peak flow
Miller Creek Goal 5: The channels must replace or enhance riparian habitat		
Provide riparian habitat	Provide a minimum 25-ft buffer on the airport side (east) of the channel from the edge of the proposed channel Provide a minimum 50-ft buffer on the west side of the channel that accommodates public access (Goal 7)	Buffers contains minimum densities of 200 trees per acre and 300 shrubs per acre. Eighty percent of trees and shrubs are native species

Table 4.2-1. **Mitigation goals, design objectives, design criteria, and final performance standards for Miller Creek (continued).**

Goal Design Objectives	Design Criteria	Final Performance Standard
Miller Creek Goal 6: The channels cannot include expansive, long-standing water pools or wetlands that could potentially attract wildlife		
Provide surface drainage for depressions and pools in the replacement channel floodplain	Provide positive floodplain drainage to reduce persistent standing water	No permanent or persistent floodplain or riparian pools develop that support waterfowl habitat
Prevent long-term standing water in the Miller Creek floodplain	Provide positive floodplain drainage to reduce persistent standing water	
Miller Creek Goal 7: The proposed Miller Creek corridor should accommodate passive recreational uses, such as walking trails		
Provide for passive recreation and public access to the new channel	Provide a channel buffer that allows for pedestrian trail construction	A minimum buffer width is provided to allow for trail construction

Data compiled by Parametrix

Table 4.2-2. Mitigation goals, design objectives, design criteria, and final performance standards for Miller Creek tributaries.

Design Objectives	Design Criteria	Final Performance Standard
Miller Creek Tributary Goal 1:	The tributaries must continue to provide adequate flow conveyance functions	
Provide drainage flow capacity	Provide channel capacity for the 100-year, 24-hour design storm	Maximum flow depth 2 ft in proposed channel
	Provide adequate capacity and channel slope to minimize erosion during the design storm	Maximum channel velocity 6 ft per second
Collect runway surface drainage and convey to Miller Creek	Collect runway surface drainage at the existing and proposed discharge points	Flow patterns and drainage from the proposed airport improvements are not significantly different from existing drainage discharge points
Miller Creek Tributary Goal 2:	The tributaries would collect seepage to maintain base flows	
Collect existing seeps from slope for maintaining base flow in Miller Creek	Collect existing slope seepage near the source in subsurface drainage systems	More than 50% of the existing observed seepage is discharged into or collected by the proposed channel
Collect drainage and seepage from the base of the proposed fill slope	Collect seepage at the base of the fill on the uphill (east) side of the proposed perimeter road	More than 50% of the existing observed seepage is discharged into or collected by the proposed channel
Miller Creek Tributary Goal 3:	The new tributary must provide an open channel of equivalent length as the existing channel	
Construct new channels with equivalent length, substrate, and streamside vegetation	Construct new channels with equivalent channel lengths: 1,200 ft for Segment B, 200 ft for Segment C	New channels that provide conveyance with well-established vegetation and limited scouring, erosion, or bank failures.
	Minimum channel slope 1%; channel side slopes 4:1 or flatter	
	Stream banks and side slopes replanted with a native mix of plants for riparian habitat	
	Channel substrate a mix of sands and gravels; channel velocity below substrate erosion velocity	
	If steep channel slope is required, protect from downcutting with log weirs	

Data compiled by Parametrix

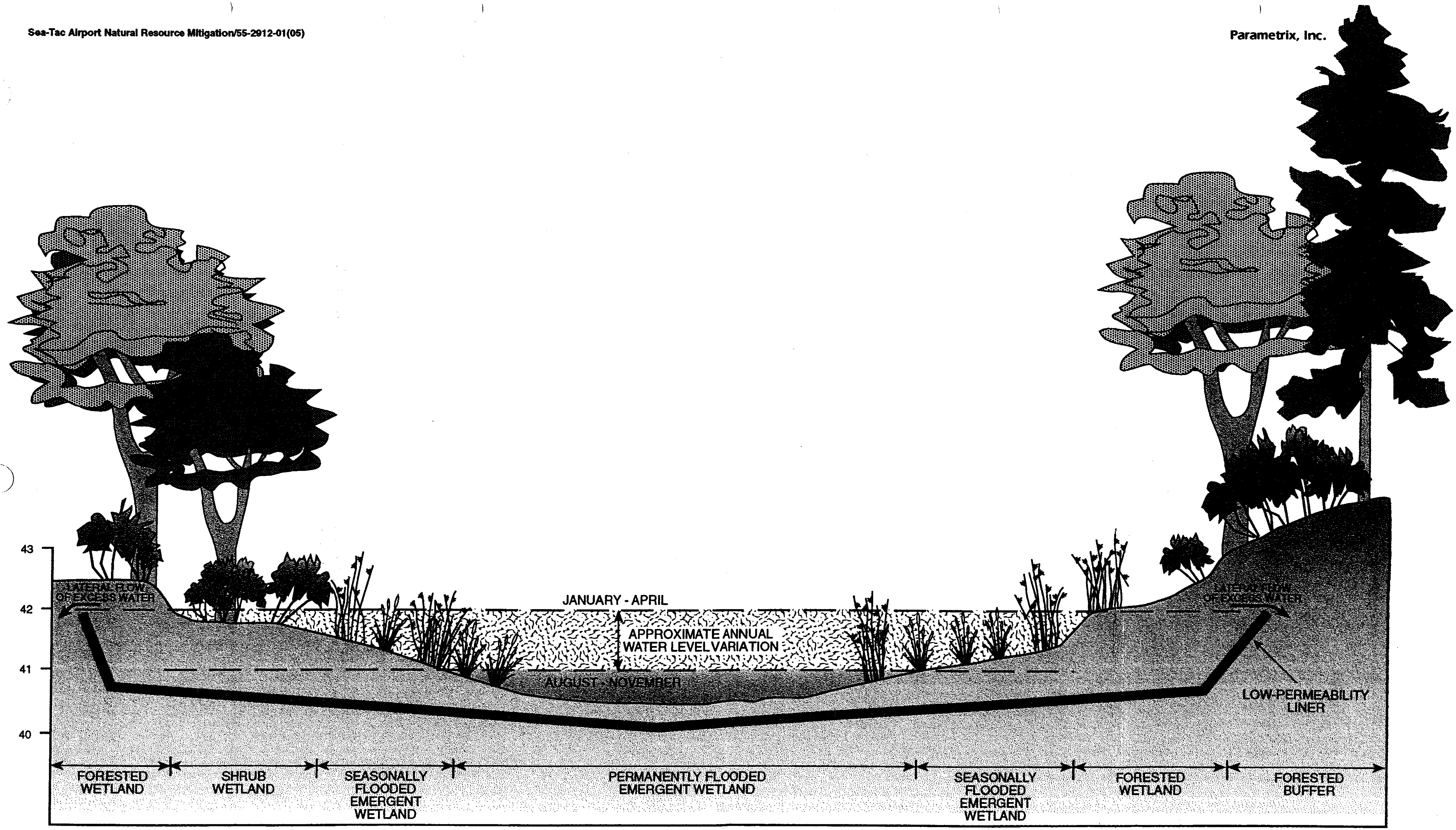

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Figure 3.5-8
Conceptual Wetland Design
Typical Restoration/Enhancement
Cross Section

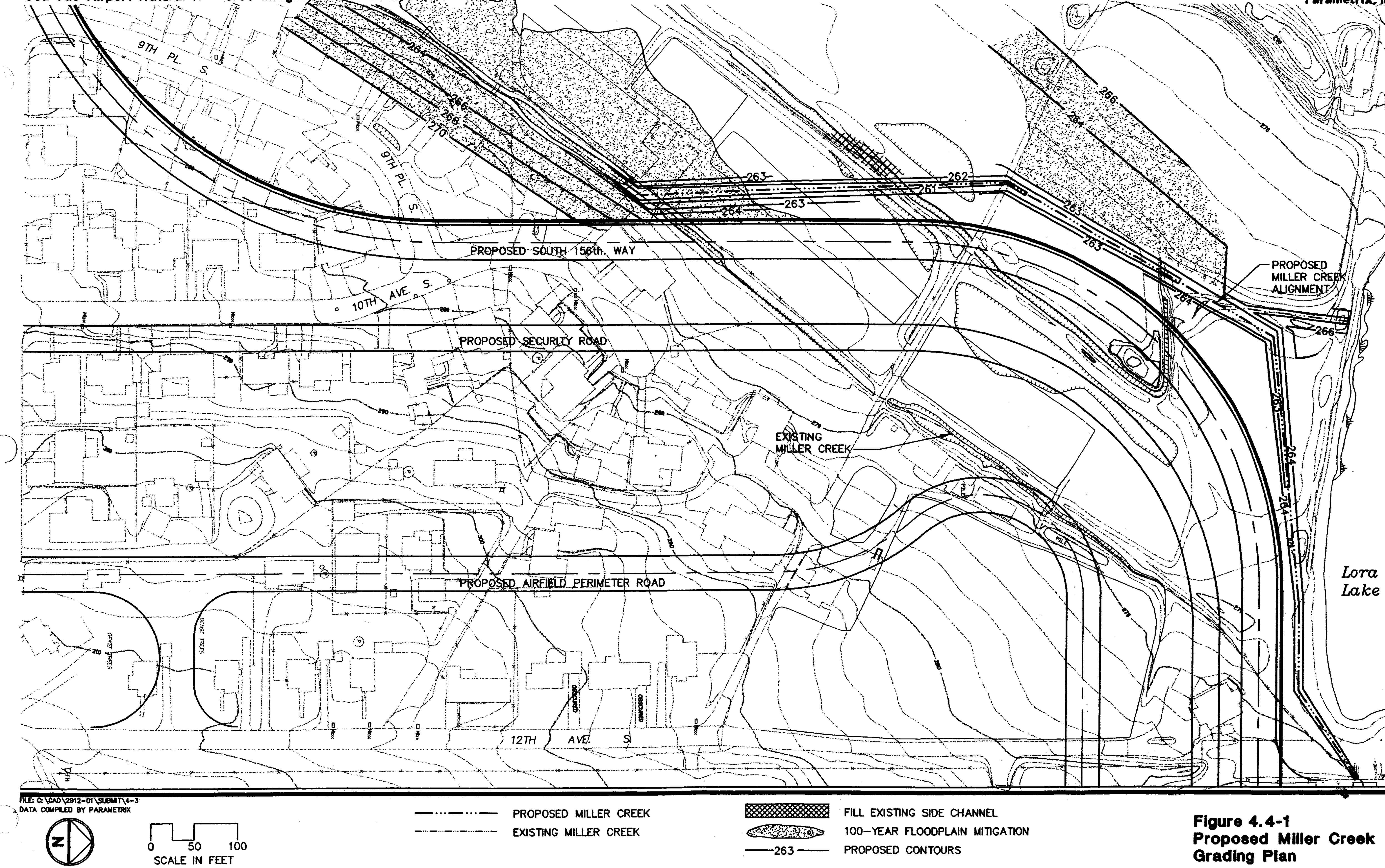


Figure 4.4-1
Proposed Miller Creek Grading Plan

4.3 PROPOSED MITIGATION SITE

4.3.1 Site Description

The proposed Miller Creek channel would be constructed near the bottom of a broad, flat valley located south of Lora Lake. The existing 1,080-ft-long main channel of Miller Creek would be displaced approximately 200 ft to the west (Figure 4.4-1).

The Miller Creek tributaries would be mitigated in the proposed new parallel runway embankment construction areas. Both mitigation channels would be constructed adjacent to the proposed airport perimeter access road. The road is in a restricted access area, and a vegetated filterstrip buffer must protect the proposed channel from road runoff.

4.3.2 Ownership

The land for the stream relocation would be purchased by the Port of Seattle as part of the larger property acquisition program for the proposed Master Plan Update improvements. It would be designated in airport planning documents as a sensitive area to be protected in perpetuity, with the exception of possible future bridge crossings.

4.3.3 Rationale for Choice

The mitigation site was chosen because it is relatively close to the edge of the parallel runway embankment, therefore, require the shortest stream relocation length. Also, extremely flat site conditions dictate that the proposed channel be as short as possible to provide the maximum possible slope. The proposed realigned creek would be located as close to the base of the proposed fill slope of the new parallel runway as possible. The channel would connect with the existing Miller Creek channel at the earliest possible point to minimize stream relocation impacts. The channel edge would be a minimum of 25 ft from the base of the slope, to accommodate a riparian buffer. However, because of the limited space between Lora Lake and the proposed embankment, narrower buffers might be required in this area. To compensate for the restrictive high flow area, flows in excess of channel capacity are planned to be diverted from the main channel of Miller Creek into Lora Lake and then reintroduced at the lake outlet channel.

The tributary mitigation site was selected as the only appropriate option for recreating the equivalent drainage length for the filled drainage channels. The existing channels could not be left undisturbed or reconstructed on the fill slope because of airport operation and fill stability requirements.

4.3.4 Ecological Assessment of the Mitigation Site

See Section 4.1, Ecological Assessment of the Impact Site.

4.3.5 Constraints

A few constraints outside of the Port's control could affect the success of the stream relocation. As discussed in Section 4.1.4, the water level of the Lake Reba complex is regulated by a control structure and gate downstream of the lake outlet. The gate is not moved in the present operation procedure. There are no existing plans to change the operation procedure, however, if a different control structure procedure were implemented, it would not affect the mitigation design because stream hydrology would not be significantly modified.

The proposed channels would be constructed on Port property and collect Port drainage. Although collecting all ground water in the vicinity of the existing seeps may prove difficult, base flows can be maintained by collecting seepage at both the source and at the toe of the proposed slope, the point where uncollected seepage water is expected to surface.

4.4 MITIGATION SITE PLAN

The description of the mitigation site plan is divided into two main sections: Miller Creek (Section 4.4.1) and the Miller Creek tributaries (Section 4.4.2).

4.4.1 Miller Creek

4.4.1.1 Site Grading

The proposed channels would be excavated and constructed as shown in Figure 4.4-1. Regrading is also necessary to provide floodplain mitigation. Approximately 5,030 yd^3 of floodplain storage would be lost in the proposed fill area. As shown in Figure 4.4-1 approximately 5,070 yd^3 of floodplain storage would be created, not including storage for the proposed stream channel. Although no additional site grading is proposed, some additional grading may be required to ensure a positive drainage flow to the new channel and prevent long periods of standing water in the floodplain.

4.4.1.2 Project Hydrology

The hydrologic design criteria for the Miller Creek mitigation plan are listed in Table 4.4-1. Because expected storm water runoff increases from the proposed airport improvements would be mitigated in separate storm water management facilities, this mitigation plan does not provide for increased flows.

Table 4.4-1. Estimated flow rates for channel design.

Flow Regime	Flow Rate (cfs)
Dry season base flow	0.5
Wet season base flow	5
"Normal" storm flow	10
Annual peak flow	40
2-year peak flow	75
10-year peak flow	125
100-year peak flow	175

Source: Montgomery Water Group 1995;
additional data compiled by Parametrix

KCSWM has monitored flow rates at the outlet of Lake Reba since 1988 (1994). Although the period of record is short, the flow data provide a good record of "normal" base flows, seasonal peak flows, and average flows by season. Design criteria for base flow and annual peak flow conditions were established from these data (Table 4.4-1). No statistical analysis of the flow monitoring data was conducted; the design flow rates were selected by examining the data and using best professional judgment to identify data trends.

In addition to monitored flow rate data, a detailed hydrologic modeling study was prepared (Montgomery Water Group 1995) that calculated peak flow rates for flood frequencies up to the 100-year flood (Table 4.1-2). The flood return frequencies were calculated assuming that the Lake Reba detention system and control structure are in place. The calculated flow rates appear to be consistent with the flow monitoring data. The peak monitored flow rate (225 cfs) on November 24, 1990, was in excess of the predicted 100-year flood flow (approximately the 500-year flood flow). The control structure was constructed after the 1990 storm; it is likely that the peak flow rate of November 1990 would have been reduced by the detention system.

4.4.1.3 Creek Hydraulics

Creek hydraulics refer to existing or proposed physical conditions that influence the direction, depth, and flow velocity in the proposed relocated creek. Several factors influence flow hydraulics including: flow rates, channel slope, channel cross section, channel roughness, and flow depth. While several of these features would be designed, factors such as flow rate or average channel slope cannot generally be modified. The following sections describe the design parameters that apply to all channel segments, the design process used, and the proposed channel configuration for each segment. The proposed creek location is shown on Figure 4.4-2. Channel substrate design is included in Section 4.4.3, Habitat.

Channel Alignment

The proposed channel cross section is shown centered on the proposed alignment. However, the channel would be constructed to meander within the limits of the stream corridor (Figure 4.4-2). Meandering would be limited, however; minimum channel slope must be maintained to meet flow velocity goals.

Channel Roughness and Side Slopes

Channel roughness, a factor in determining channel capacity is described by using Manning's roughness factor, *n*. The assumed Manning's channel roughness for the relocated stream is 0.035; this corresponds to a natural channel with a gravelly or stony bottom and little instream vegetation.

The bottom 6 inches of the channel side slopes would be vertical (Figure 4.4-3). From 6 inches to 1 ft, channel side slopes would continue at 1:1 slopes, primarily to enhance stability, provide additional capacity, and simplify construction. From 1 to 2 ft, the side slope would be 6:1 or flatter, depending on channel capacity requirements and channel planting and buffer requirements. Above 2 ft of depth, natural grades would be used; however, if natural slopes are too flat, slope or drainage alterations would be considered to prevent ponding.

Channel Slope

Average channel slope is determined by the physical constraints of the site. The bottom elevation at the upstream end of the proposed channel (at the control structure outlet of Lake Reba) is approximately elevation 264.0 ft. The approximate elevation at the point where the relocated creek rejoins the existing channel is 260.0 ft. With a proposed channel length of approximately 1,080 ft, the average channel slope is 0.37 percent. However, natural land slope along the proposed stream channel does not drop continuously. The proposed alignment's existing grade is approximately level at the start, then gradually slopes as the alignment turns south. The alignment moves through a shallow depression, then begins to rise slightly before rejoining the existing stream. To work with existing topography, the channel was divided into three segments (A, B, and C) to determine how the slopes must vary through the proposed alignment.

Flow velocity that meets the proposed design goals is primarily a function of channel slope. Because the site offers little slope to increase flow velocity, compromises must be made for providing flows that minimize sedimentation. Slopes in segments A and B (0.3 percent and 0.4 percent, respectively) were designed to limit sand deposition at base flow, while Segment C (0.2

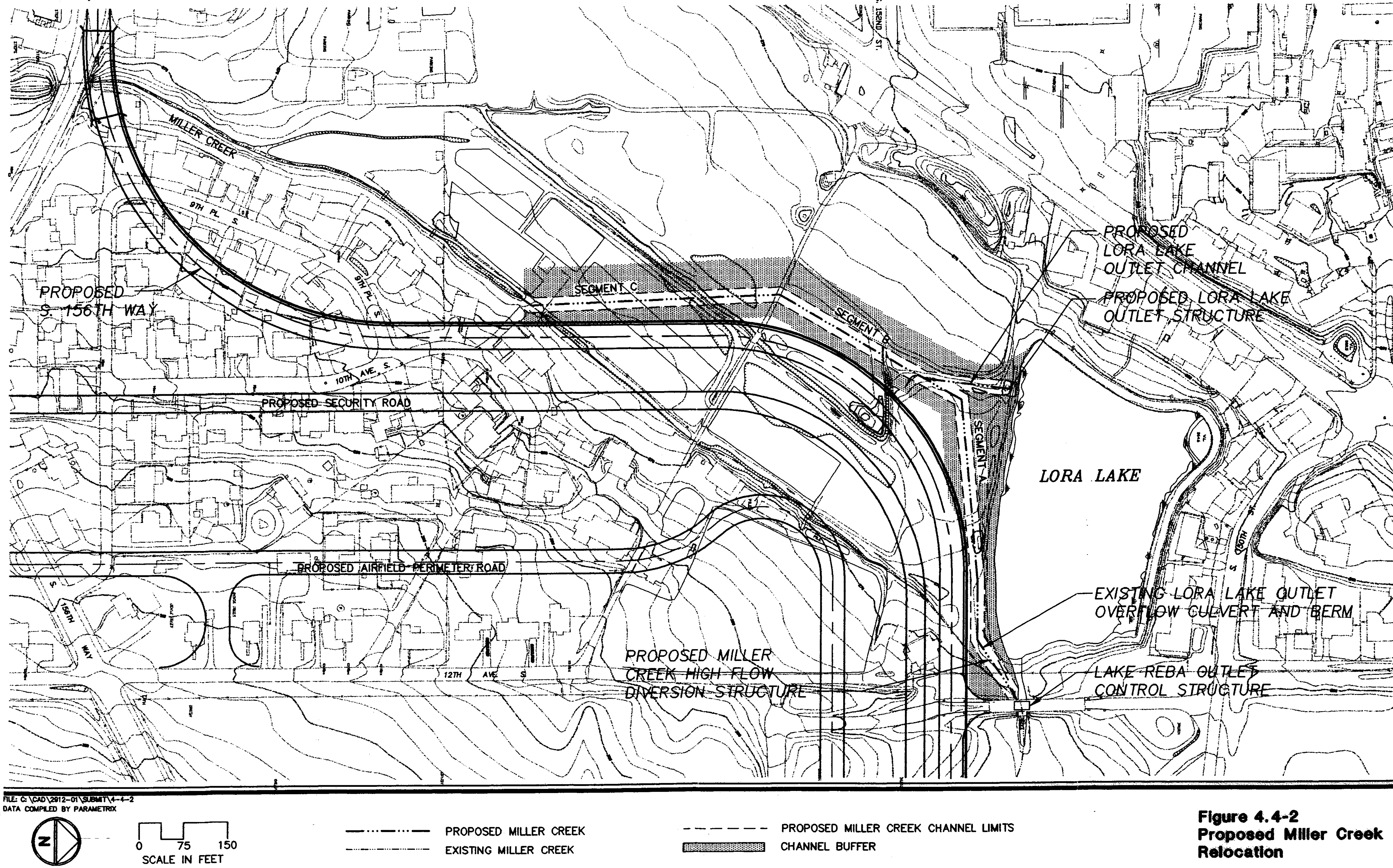
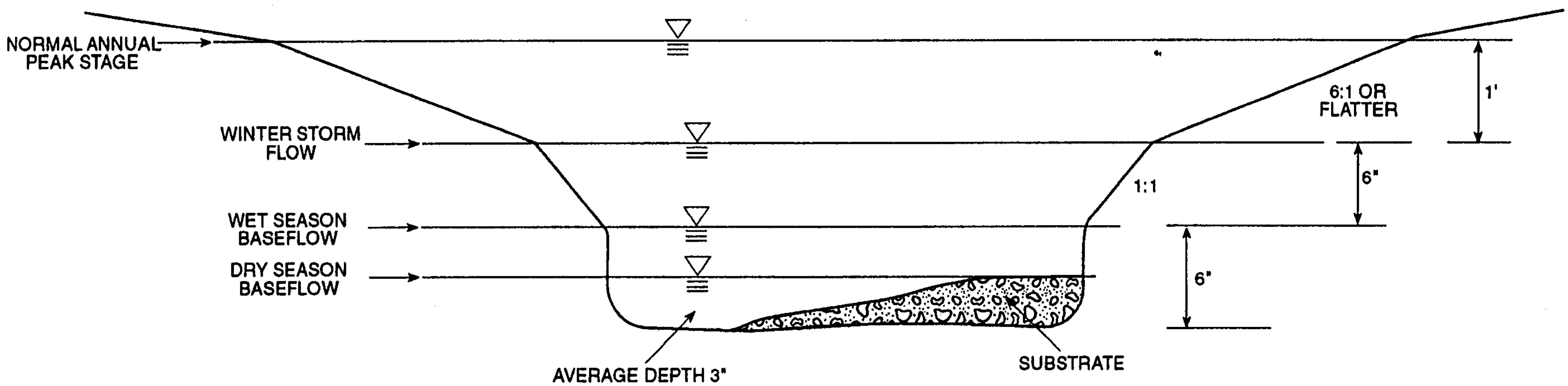
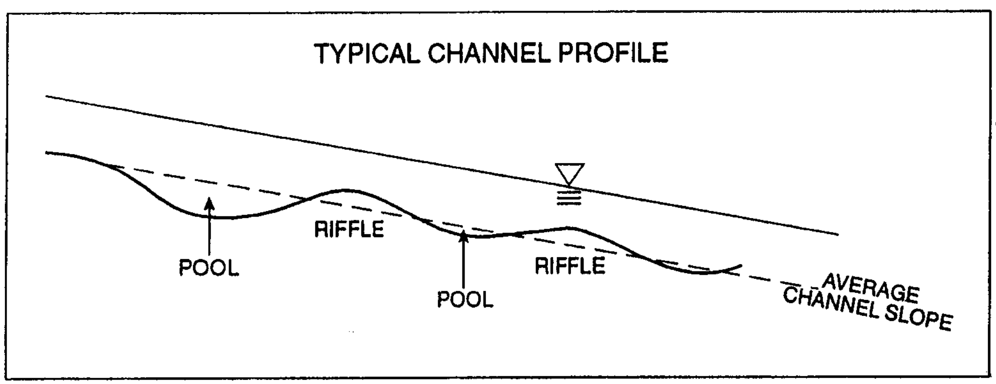
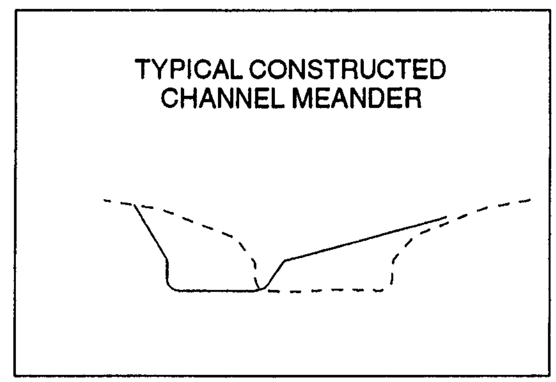


Figure 4.4-2
Proposed Miller Creek Relocation







Data Compiled by Parametrix

Figure 4.4-3
Proposed Miller Creek Stream Channel

percent slope) was designed to prevent silt deposition at base flows. A more complete discussion of flow velocity is included in the following section.

Flow Velocity

Channel flow velocity is the primary variable influencing channel design. The goal is to minimize fine-grained (silt and finer) material sedimentation in all proposed channel segments during normal dry season base flows. If possible, sand deposition should also be limited. Conversely, the flow velocity at peak design flows must not exceed rates that would erode the channel and scour loose sediment and substrate larger than small gravel. With a minimum flow depth of 0.25 ft at the base flow rate, and with channel roughness and side slopes fixed, channel velocity is a funçtion of channel bottom width and slope. Figure 4.4-4 shows the relationship between flow velocity and sediment transport velocity. If the flow velocity equals or exceeds that shown for each grain size, the sediment can be expected to move until the velocity decreases.

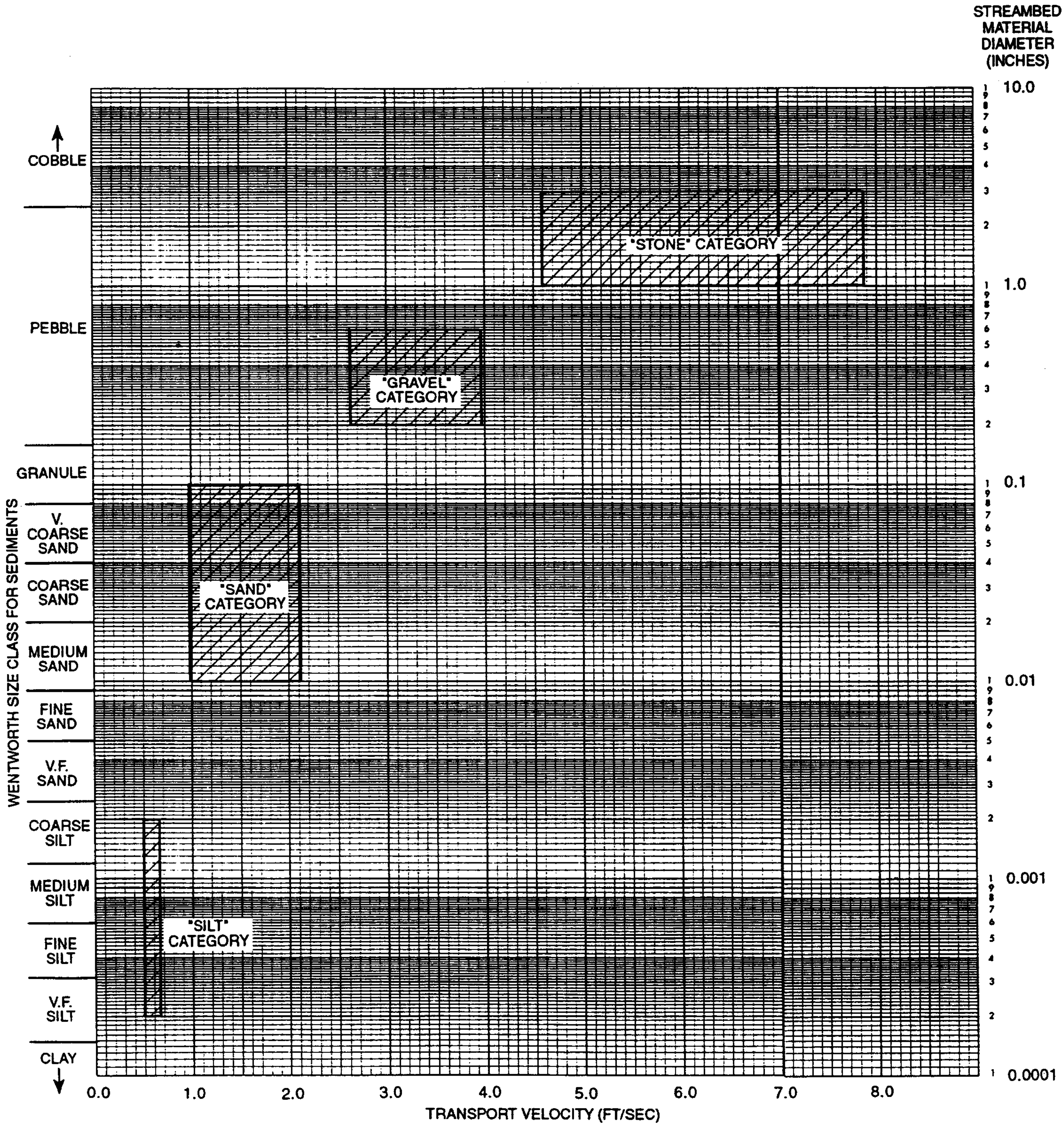
Channel design is a process whereby variables are adjusted until all of the design parameters are met. Initial channel slope was estimated using the available drop for Segment A. The corresponding channel bottom width was determined and adjusted until the minimum flow depth (0.25 ft) was achieved. The slope was then adjusted until the base flow velocity was strong enough to move sediments smaller than sand. Using the adjusted slope, the channel was then checked for peak flow rate velocity (in connection with maximum depths and channel configurations described in the following sections). Channel widths and flow depth were adjusted until flow velocity was less than the transport velocity for gravel. These steps were used in each alignment section.

Channel Bottom Width

The channel bottom width, within the narrow range of possible channel slopes and using the fixed side slope and roughness values, is controlled primarily by the minimum flow depth. Dry season base flow depth must average at least 0.25 ft to provide minimum depth for fish movement. To determine the channel bottom width, the base flow rate, slope, roughness, and side slopes were fixed, and the bottom width was adjusted until the flow depth was at least 0.25 ft. The results were checked to ensure that no other design criteria were changed to exceed design parameters.

Channel Flow Depth

Several design channel flow depths are available, depending on the flow rate and the design event. Three flow depth standards have been determined: (1) dry season base flow depth of 0.25 ft; (2) wet season base flow depth of 1 ft; (3) annual maximum flow rate depth of 2 ft. Flows greater than the annual maximum flow rate (40 cfs) will overflow into the floodplain.



Source: B.C. Fisheries, 1980

Figure 4.4-4
Sediment Transport Velocity vs. Sediment Diameter

Figure 4.4-5 shows the approximate extent of the design storm floodplains.

Maximum Design Channel Flow

Segment A, located between Lora Lake and the proposed fill, is somewhat narrower than Segments B and C. As a result, limited area is available for constructing a large channel that can convey the 100-year storm, while maintaining a minimum flow depth for dry season base flows. This mitigation plan proposes a high-flow diversion structure near the beginning of the proposed channel relocation, to divert flows in excess of the channel capacity (the 2-year storm) through Lora Lake. The lake acts as a bypass channel that buffers peak flows and releases water at a reduced rate to other segments that have adequate capacity. The proposed control structure design is shown on Figure 4.4-6.

Lora Lake Outlet Channel and Structure

Runoff flowing into Lora Lake overflows into Miller Creek through a 12-inch concrete culvert located in a berm that forms the south shore of the lake (Figure 4.4-2). When inflow exceeds the lake storage and outlet pipe capacity, water flows over a low spot in the berm. In extreme conditions, it is likely that the lake becomes part of the Miller Creek floodplain and completely overwhelms the south shore berm.

The proposed Lora Lake outlet channel and structure is designed to release base flows in a manner approximating the existing outlet structure. The proposed structure has a controlled overflow feature that maximizes lake storage without adversely affecting lake stages or inflows. A 12-inch low-flow orifice and 10-ft overflow weir would be constructed. The elevations of the existing pipe and overflow basin would be used for the proposed outlet. The overflow weir would have a broad-crested overflow, approximately 5 ft wide, with erosion control such as rock and wire mesh. The existing Lora Lake outlet channel has similar design slopes to Segment A, and potentially provides stream habitat.

4.4.2 Miller Creek Tributaries

Three tributaries of Miller Creek in Areas 2 and 3 would be affected by the proposed airport improvement. All three are intermittent streams, primarily fed by rainfall, but supplemented by groundwater seepage. The tributaries flow intermittently from culverts at the airport and from seeps; however, no flow monitoring is available. The proposed channel design will be based on hydrologic model calculations. Portions of all three channels have been partially modified at road crossings, and Tributary B has been channelized for approximately 300 ft in a roadside ditch. The primary goals of tributary mitigation are to provide equivalent open channel lengths, peak storm conveyance, and groundwater seepage (base flows).

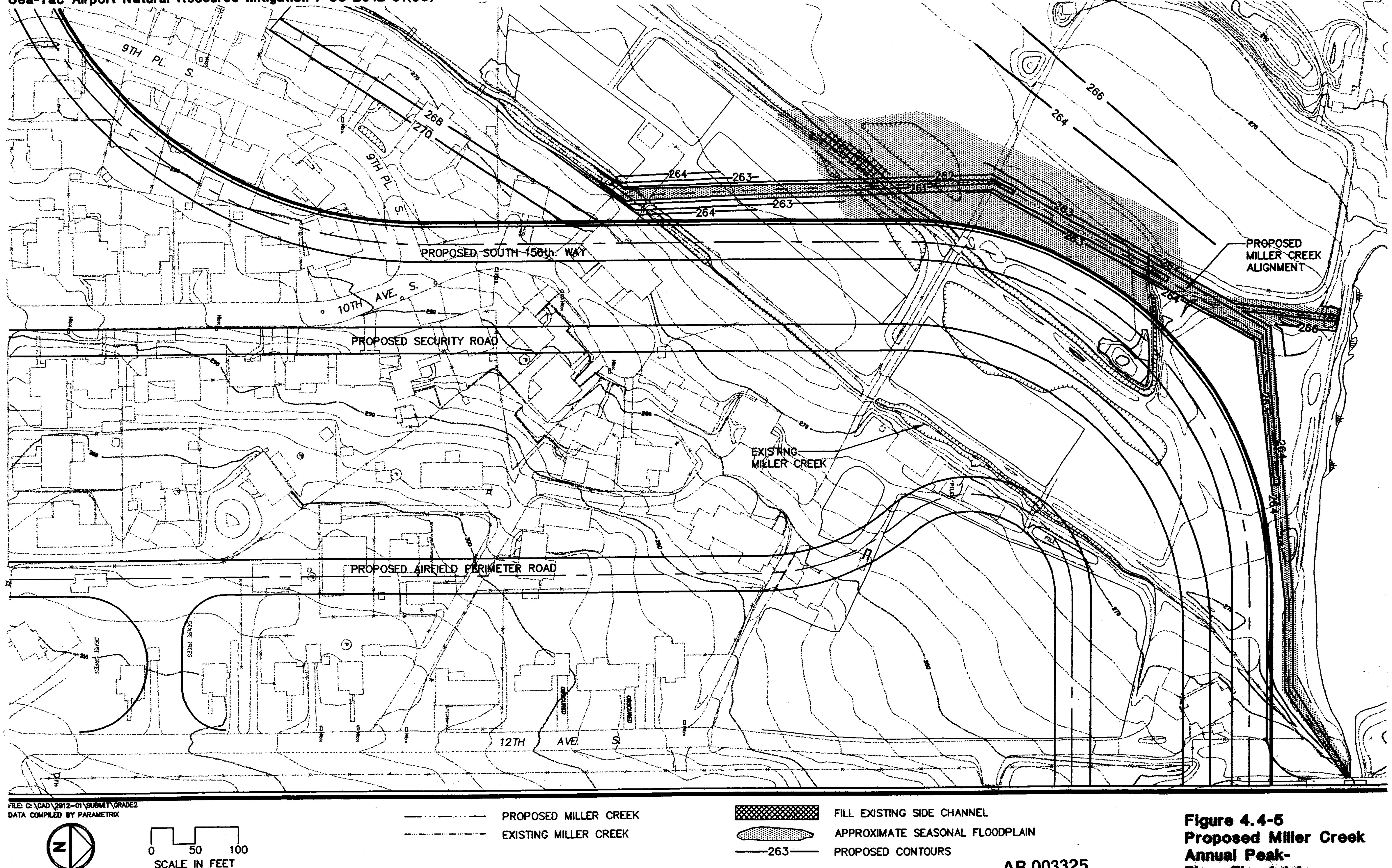


Figure 4.4-5 Proposed Miller Creek Annual Peak-Flow Floodplain

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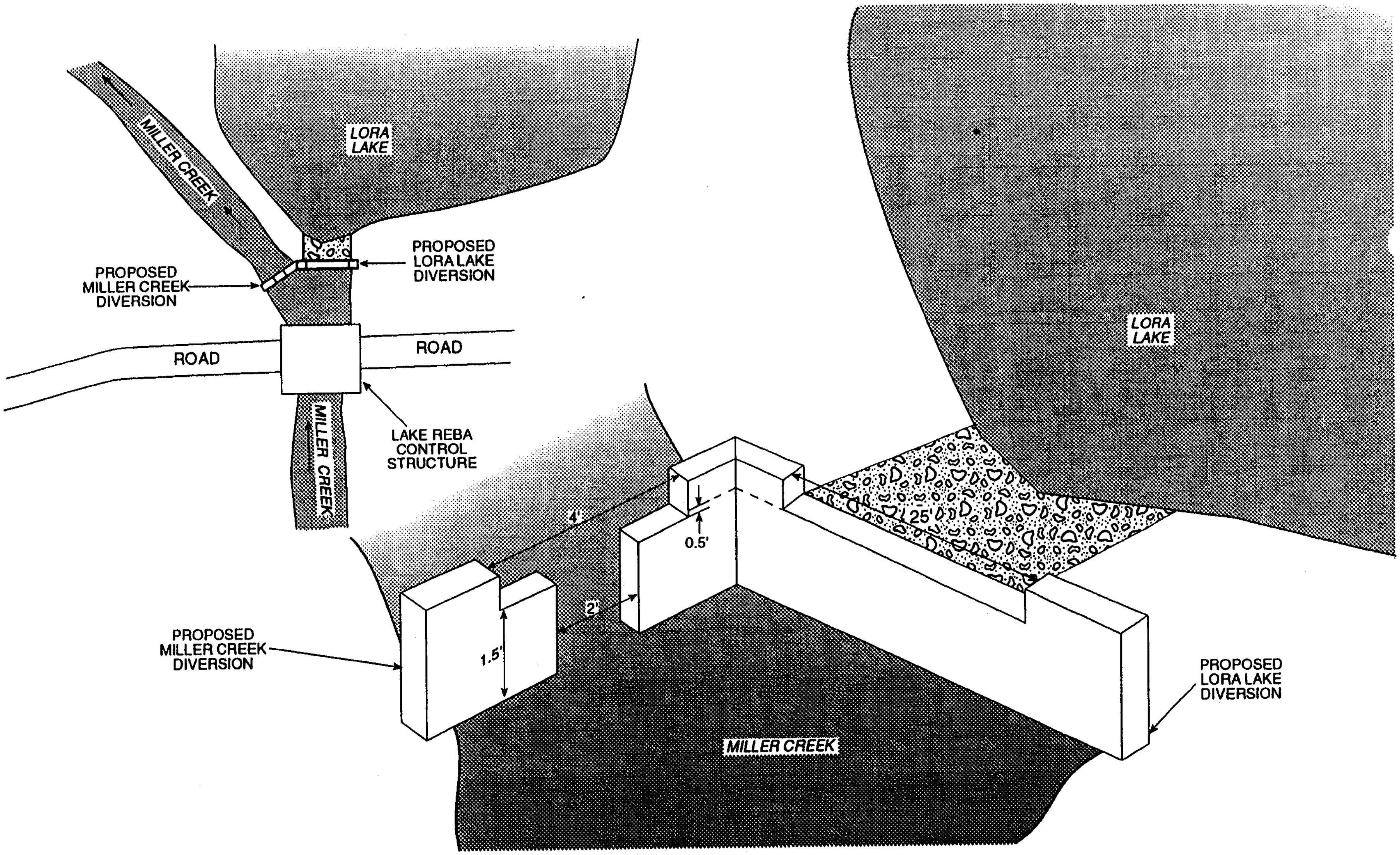

Data Compiled by Parametrix

Figure 4.4-6
Proposed Miller Creek High-Flow Bypass Structure

The Miller Creek tributaries mitigation plan has three requirements: to provide adequate capacity to handle the design flow (100-year storm), provide an equivalent open flow channel, and maintain base flows by capturing seepage from the proposed fill slope. Mitigation for all three tributaries would share the same design goals. The proposed mitigation channel for Tributary B is shown in Figures 4.4-7, and 4.4-8.

4.4.2.1 Channel Configuration

The tributary mitigation channels would be constructed on the east side of the proposed perimeter access road. The bottom channel width may vary, but a minimum 2-ft bottom would convey the peak design flow, assuming a one percent slope. The proposed channel would be incorporated into the fill slope; therefore, final design parameters, such as peak flow rates and channel slope, would be used to adjust the channel configuration. Minor modifications to the preliminary design would not significantly alter the proposed mitigation channels. A vegetated filter strip would separate the perimeter access road from the mitigation channel.

4.4.2.2 Channel Length

Equivalent channel length would be provided for each of the disturbed tributaries. Tributary B would be accommodated in a single mitigated channel constructed adjacent to the proposed perimeter road. Tributary A, the drainage ditch that collects runoff from the fill slope and perimeter road, provides little or no habitat functions. The proposed mitigation would replace the tributary's primary function, which is to provide drainage. Approximately 1,200 ft of Tributary A channel would be lost; the proposed channel would be the same length. Seepage and drainage from the Tributary A basin would be collected in the Tributary B mitigation channel. The Tributary C mitigation channel would be approximately 300 ft long.

4.4.2.3 Channel Size and Slope

The proposed channel would be designed to convey the 100-year peak flow rate. While maximum flow depth would be determined by road design considerations, it would be less than 2 ft deep. Minimum slope would be 1 percent.

4.4.2.4 Discharge Point

Both mitigation channels would discharge into the existing channel at the edge of the proposed fill slope.

4.4.2.5 Channel Cross Section

The proposed channel cross section would have side slopes at a maximum slope of 4:1. The bottom width, to be determined in final design, would be controlled by the design depth and slope. Flow control would use check dams, log weirs, or channel widening to prevent erosion,

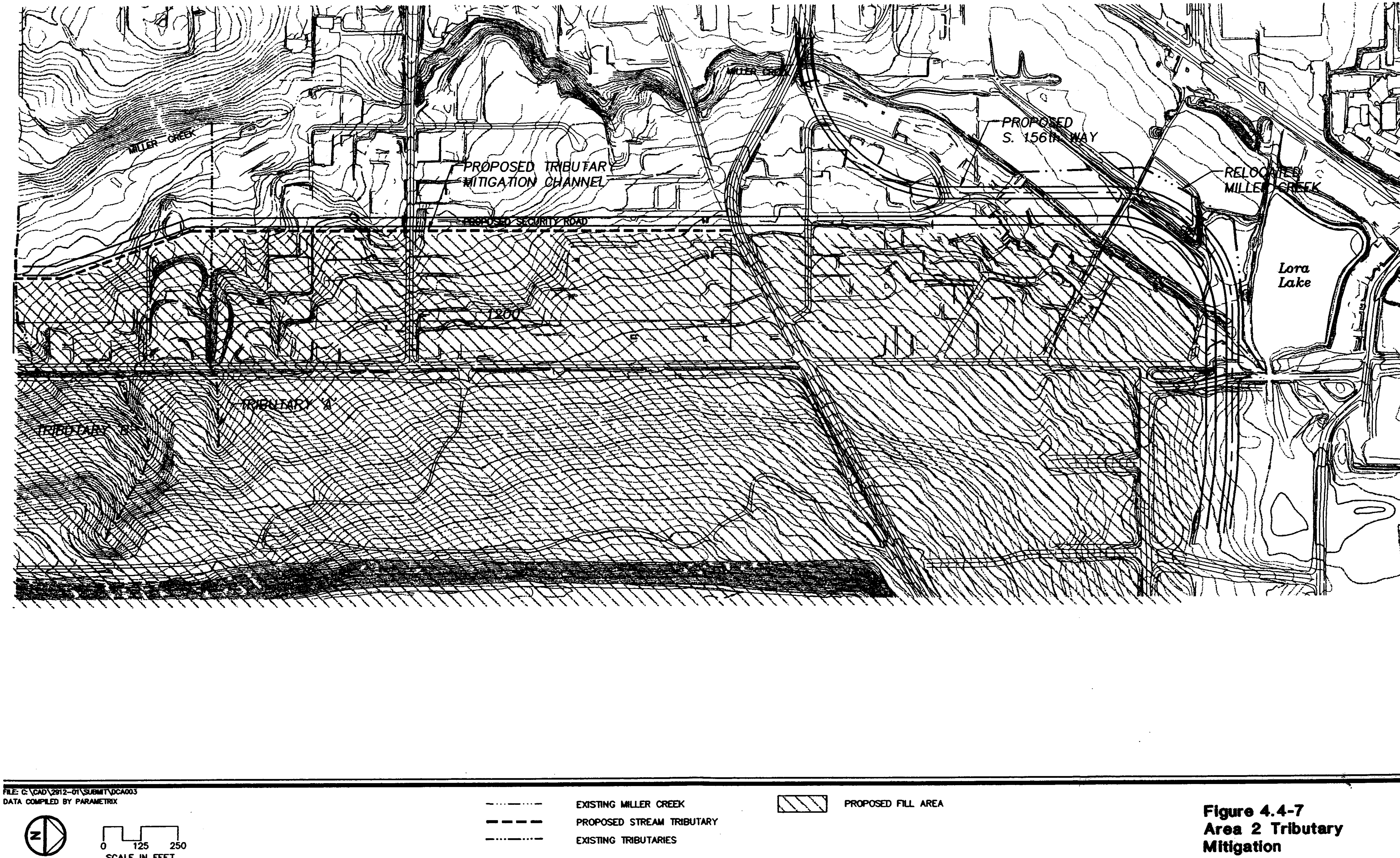

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Figure 4.4-7
Area 2 Tributary Mitigation

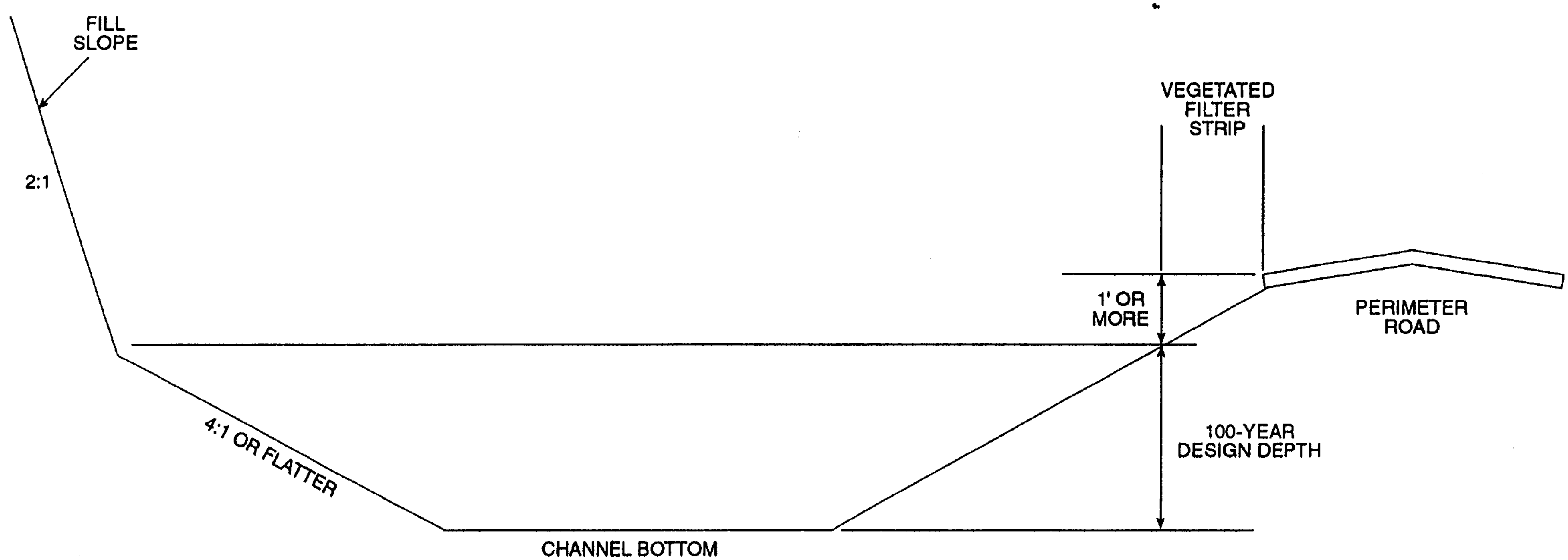

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Figure 4.4-8
Typical Tributary
Channel Section

sedimentation, scouring and downstream deposition impacts. The structures would be built to control flow and not to provide habitat.

4.4.2.6 Channel Vegetation

The side slopes and buffers would be planted with native vegetation to provide shade and nutrient loading to the channel. Section 4.4.3.1 includes a discussion of appropriate plant species.

4.4.2.7 Groundwater Seepage and Hydrology

Hydrology would be maintained by constructing a subsurface drainage system to collect the seepage from the hillside that is currently surfacing to form the existing channels. The system would consist of a field of perforated pipes packed with highly porous sand or gravel. Seepage would be collected, conveyed, and discharged to the edge of the new fill slope at the head of the proposed channels.

4.4.3 <u>Habitat</u>

4.4.3.1 Instream Habitat

The instream habitat criteria used in the relocated channel design are based on general habitat requirements of salmonids. The purpose of using these criteria is to provide the highest quality habitat and environmental conditions for fish. A stream that provides quality salmon habitat is a community goal. Compared to most resident fish species, salmonids are typically very sensitive to environmental conditions such as habitat and water quality. Salmonid prey items, such as aquatic insects, also tend to have similar environmental requirements. Therefore, designing the relocated stream to meet the needs of these sensitive species would help ensure that the best possible fish habitat is created. Although anadromous salmonids are currently restricted from the proposed impact areas, resident cutthroat trout might be present.

In general, salmonids require cool, well-oxygenated water, spawning gravel that is free of accumulated silt, and abundant instream cover habitat. In addition, because habitat requirements vary as life stages change, habitat complexity within the stream is also necessary. General habitat requirements include:

- Access to habitat
- Stable flows
- Stream substrate
- Riparian and instream cover.

Habitat Access

The various habitat areas should be accessible to resident fish populations under all flow conditions. They should provide protected areas during high flows and avoid stranding problems during low-flow conditions. Fish access throughout the entire relocated stream section would be provided by the design minimum depth requirements. The channel is designed to provide an average minimum depth of 0.25 ft during dry season base flows. This minimum depth requirement allows fish access throughout the length of the channel to avoid stranding problems during low-flow periods.

Stable Flows

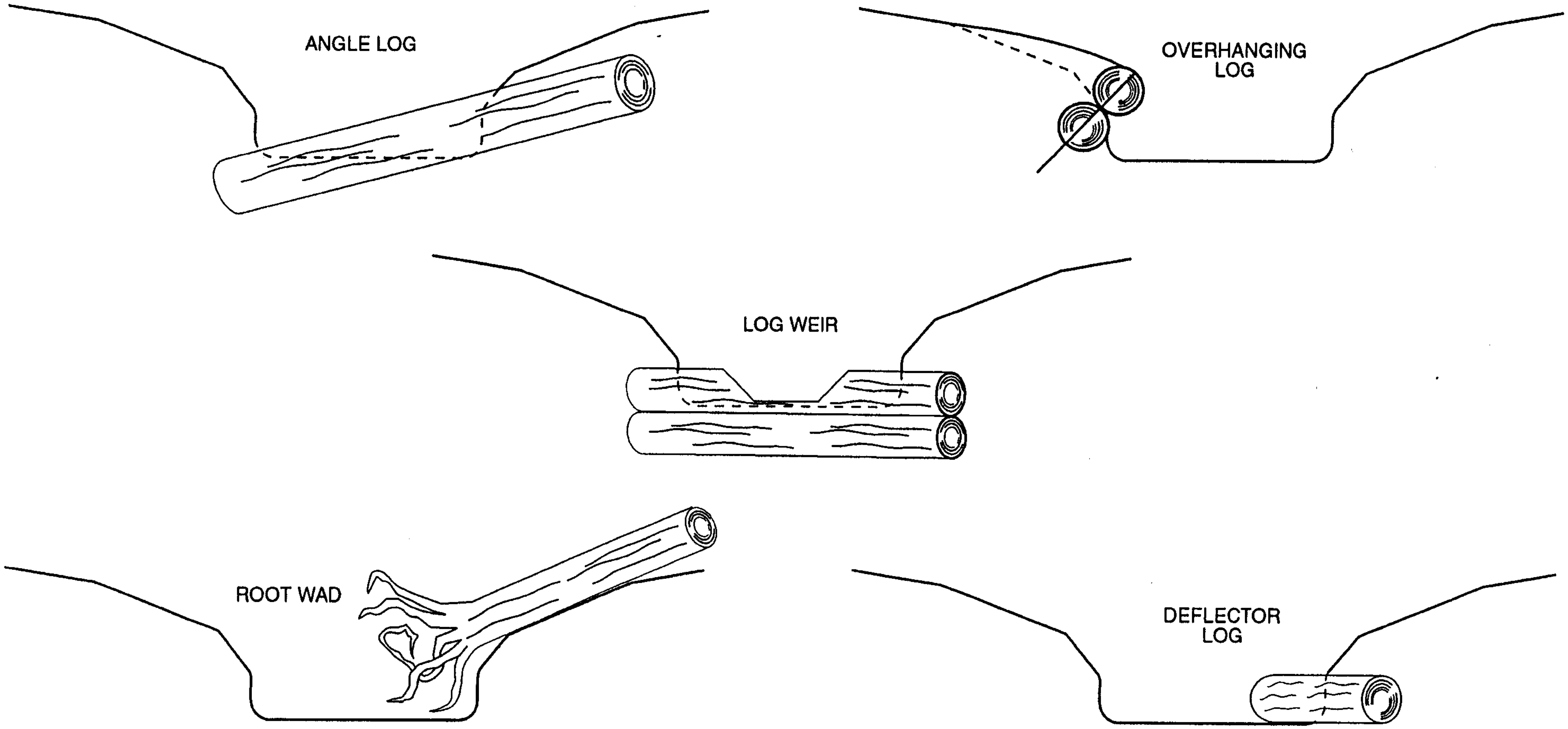
Stable flows ensure habitat access and protect the habitat against erosion or scouring; they also minimize the displacement of fish to less preferred habitats. The flow velocity criteria for the channel are set to minimize both the accumulation of fine-grained material in the channel during low-flow periods, and excessive scouring of larger-grained materials during high flows. However, since these flow velocities are also an average over the entire channel (similar to the depth criteria), sedimentation is expected to occur on the inside of bends and in deeper pools during low-flow periods. These sediments do, however, flush out again with higher flows. The channel width and bank slopes criteria have also been incorporated in the design to maintain relatively stable flow velocities throughout typical flow variations. In addition, a high-flow diversion structure has been included for Segment A to minimize erosion and fish displacement processes during unusually high-flow periods.

Stream Substrate

Salmonids typically require stable gravel that is essentially free of accumulated silt for spawning and early rearing life stages, as well as for optimum production of desired prey. Substrate in the relocated channel would consist of gravel and cobble material to provide good stable spawning and rearing habitat. However, portions of the channel would naturally be seeded with sandy material over time.

Riparian and Instream Cover

Salmonids require cover habitat provided by such features as undercut banks, logs and boulders, deep pools, and overhanging riparian vegetation for feeding, hiding, and resting. In addition, these features help to stabilize stream banks and substrate during high-flow periods. The relocated channel, which is designed with vertical banks in the low-flow depth range, would encourage minor undercutting to provide cover habitat during low-flow periods. Large woody debris (deflector logs, angle logs, and root wads), as well as boulders would be used to stabilize the substrate, protect the upper banks from excessive erosion, and provide hiding and holding habitat for fish during higher flow periods (Figure 4.4-9).



Data Compiled by Parametrix

Figure 4.4-9
Representative Fish Habitat Enhancement Features

Overhanging riparian vegetation would be used to maximize stream shade and provide overhanging cover habitat. This vegetation would deposit organic debris (leaves, branches, etc.) into the stream to provide a food source for aquatic insects; it also provides a mechanism for terrestrial insects to enter the stream, thereby providing valuable food sources for fish. Suitable plants include red-osier dogwood, Pacific willow, and salmonberry shrubs (Table 4.4-2).

Table 4.4-2. Suggested plants for riparian fringe relocation.

Scientific Name	Common Name	Streamside Zone	Upland Buffer Zone
Trees			
Acer circinatum	Vine maple	X	X
Alnus rubra	Red alder	X	X
Corylus cornuta	Western Hazel		X
Fraxinus latifolia	Oregon ash	X	
Rhamnus purshiana	Cascora		X
Salix scoulerana	Scouler willow	X	
Shrubs			
Cornus stolonifera	Red-osier dogwood	X	
Gaultheria shallon	Salal		X
Physocarpus capitatus	Pacific ninebark	X	X
Rosa woodsii	Wood's rose		X
Salix sitchensis	Sitka willow	X	
Salix lasiandra	Pacific willow	X	
Salix hookerana	Hooker willow	X	
Spiraea douglasii	Hardhack spirea	X	

Data compiled by Parametrix

Riparian and buffer areas would be planted with species that provide rapid development of woody plant cover to shade the stream and function as a riparian buffer, while minimizing the potential for attracting wildlife. Plants suitable for stream riparian areas are listed in Table 4.4-2. Riparian buffers plantings would have a tree density of about 250 stems per acre, and a shrub density of about 400 stems per acre. Buffers would extend 25 ft from the edge of the floodplain on the east side of the creek and 50 ft from the floodplain edge on the west side of the creek.

Several strategies have been used to ensure rapid development of shade along the relocated stream. The landscape design concentrates plantings on the stream bank to ensure partial shading of the stream immediately following planting. Streamside plantings including fast-growing willow and red-osier dogwood should provide substantial shade within 3 years.

Upland buffers would include a variety of plant species including red alder, cascara, western hazel, rose, and salal. Plant selection favored species that would be unlikely to attract large populations of birds (due to aircraft flight safety concerns). The planting design discourages human intrusion into the buffer by using thorn-bearing plants and/or split-post fencing. Exposed areas between plantings in the upland buffer would be hydroseeded with upland grass mixtures.

4.4.3.2 Channel Substrate

Erosion and movement of streambed sediments need to be considered when designing stream habitat features. As discharge increases in a stream channel, not only does the water level rise, but the streambed may be scoured. In general, smaller diameter particles tend to be transported at lower stream velocities relative to larger particles. The substrate criteria used in the relocated channel design are based on the general characteristics that encourage salmonid production. These criteria provide suitable spawning gravel, while minimizing the risks of scouring and transporting this material downstream during high flows.

The minimum transport velocities for various sizes of streambed particles are summarized in Figure 4.4-4. This figure was developed from data contained in the British Columbia Department of Fisheries and Oceans' *Stream Enhancement Guide* (British Columbia Fisheries 1980). If the maximum velocity of a specific section of a stream channel is known, an estimate of the size of the bed material that would be relatively stable can be determined. This is particularly important where gravel is being added to modify stream characteristics, such as to improve spawning conditions.

Miller Creek relocation requires a balance between a minimum base flow velocity, to prevent sedimentation, and a maximum peak flow velocity that could scour sediment. Therefore, it is desirable to have base flow velocities sufficient to transport finer-grained particles (such as silt), and peak flow velocities that do not remove coarser-grained particles such as gravel. High flows are required to initiate particle movement, and slightly lower flows have carrying power to keep the particle moving. Using Figure 4.4-4, the channel parameters were adjusted to maintain base flow velocity greater than the silt movement velocity, but less than the gravel movement velocity for peak flow. Gravel recruitment from upstream of the mitigation channel would be limited by the Lake Reba detention facility.

4.4.4 Floodplains

4.4.4.1 Floodplain Storage Mitigation

The proposed channel was designed not to impede the 100-year flood; however, flood flows are not expected to be completely contained within the stream banks. One hundred-year flood storage lost by the proposed fill would be approximately 5,030 yd^3. Equivalent effective floodplain storage, as shown on Figure 4.4-1, would provide approximately 5,070 yd^3 of floodplain storage mitigation.

4.4.4.2 Floodplain Conveyance

The 100-year floodplain elevation in the proposed study area was determined by FEMA when the Flood Insurance Rate Maps (FIRM) were prepared. The proposed channel capacity was checked for the 100-year flow rate peak capacity. No encroachment or fill is proposed in the 100-year conveyance area or floodway. No backwater calculations were made to estimate 100-year flood elevation impacts. However, no impacts are expected since the floodplain storage has not been altered and the 100-year conveyance channel has adequate capacity.

4.4.5 Implementation Schedule

Construction of the proposed parallel runway, which would affect Miller Creek, is currently scheduled as part of Phase I (1996 - 2000) of the proposed Master Plan Update implementation. The new stream channel must be constructed and fully stabilized before stream flow is diverted from the old channel. Therefore, the stream channel would need to be constructed during the early years of runway construction.

4.5 MONITORING PLAN

4.5.1 Hydrology and Hydraulics

The effectiveness of the relocated stream can be measured in several ways, but fish habitat stability is an important gauge. Because erosion and sedimentation are the primary indicators of stream hydraulic conditions, they are the critical criteria to be included in the proposed monitoring plan. The following activities would be included in the stream monitoring plan:

- Inspect the constructed habitat features (log weirs, root wads, etc.) to ensure that they have not been damaged or displaced (to the extent that they are not providing habitat).

- Inspect the substrate to ensure that sedimentation and erosion prevention goals are met.

- Inspect for erosion or scouring.

- Inspect stream structures and channel after major storms, as monitored by the KCSWM gage.

- Inspect for adverse flooding impacts and ponding water.

- Inspect diversion and outlet structures for debris accumulation, scouring, and damage.

4.5.1.1 Inspection Schedule

Table 4.5-1 includes the inspection schedule for monitoring the Miller Creek stream relocation and tributary enhancement. The schedule includes routine inspections and emergency inspections, in case of a major flood.

4.6 SITE PROTECTION

The site would be owned by the Port of Seattle and be designated in airport planning documents as a sensitive area to be protected in perpetuity. However, because of potential access needs, one or more road crossings may be developed sometime in the future.

4.7 MAINTENANCE AND CONTINGENCY PLAN

4.7.1 Maintenance

A design goal for the stream channel is that it functions as a natural channel, requiring little or no maintenance. To ensure that this goal is achieved, a monitoring program (described in Section 4.5) is required. Monitoring activities and frequencies are listed in Table 4-5.1. As indicated in this table, periodic maintenance may be required to correct a variety of detrimental conditions.

4.7.2 Contingency

The proposed channel configuration has two basic conveyance criteria: (1) maintain minimum flow depths and velocity for fish passage, water quality, and sedimentation; and (2) provide flow capacity for peak flows. The channel was configured to provide the required design criteria by developing a narrow channel cross section to accommodate base flow conditions and a wider channel cross section (at slightly higher elevations) to accommodate flood flow conditions. If flow rates and stream hydrology are substantially different from the design flows used to develop this plan, the channel may not function as designed. If channel hydrology is substantially different from data used to create this design, the channel section can be modified by:

Table 4.5-1. Miller Creek mitigation monitoring schedule.

Inspect	Frequency	Action Threshold	Action
Habitat structures	Annually (May), or after flows in excess of the 2-year peak flow	structure displaced[1], causing erosion or collecting debris	repair or replace[2]
Buffer Vegetation	Annually	Mortality results in less than 200 trees per acre or less than 300 shrub stems per acre	Evaluate reasons for mortality, replace plantings, and substitute with other species if appropriate.
Substrate	Semi-annually (February/August)	Winter - sediments (sand or smaller) in shallow, flowing segments or riffles Summer - Fine sediment (silt or smaller) in flowing segments or riffles	Prepare options for reducing stream bottom width (i.e., lateral logs, boulders) if sedimentation persists for a second year.
Erosion or Scouring	Annually (May), or after 2-year storm	bottom sediment gone; excessive streambank erosion causing sloughing; excessive habitat damage	Repair damage (bioengineering, etc.) and enlarge channel if damage increases in the 2nd year.
Control Structures	Annually (May), or after 2-year storm	structural damage or failure; obvious scouring or cavitation	determine cause and repair
Adverse Flooding	Twice yearly (November/February)	trapped standing or ponding water; persistent slow drainage	improve surface drainage paths

Data compiled by Parametrix

[1] A structure can be damaged or displaced and still provide habitat consistent with mitigation goals.

[2] The benefits of repair or replacement would be balanced against the potential impacts.

- widening the base flow channel width to reduce velocities, and improve capacity
- narrowing the base flow channel with logs or boulders to increase base flow depth and velocity.
- widening the flood flow of the channel (above 0.5 ft) to improve capacity and reduce velocity

- adding log weir "steps" to flatten stream slope, reducing velocity and increasing base flow depth.

- adding a bypass flow channel to convey peak flows past the main channel in Segments 2 and 3.

If standing water persists in the floodplain, side channels could be graded to provide positive drainage to the main channel.

KCSWM has a control structure at the outlet of Lake Reba with an adjustable gate. Under current operating conditions this gate is not adjusted and it is unlikely that operations would be modified to allow more water to discharge from Lake Reba. However, future needs could allow higher flows under certain conditions. Since the Miller Creek diversion structure would divert most floodwater into Lora Lake, increased flow from Lake Reba would have only a modest effect on the new stream channel. If the Lake Reba outlet were modified, contingency actions could include simple modifications to the diversion structure at the head of the channel to direct more flow into Lora Lake (for detention purposes) and away from the new Miller Creek channel.

Contingency measures for buffer vegetation include replanting areas if high mortality is observed. If significant plant loss occurs, site conditions would be evaluated to determine whether the conditions can support the species planted.

Major factors likely to contribute to large-scale plant loss include improper hydrologic condition, improper soil conditions, and pest infestation. Depending on the cause of the plant loss, a variety of remedial actions may be necessary to allow successful plant survival and restoration. If necessary, site conditions can be altered to enhance planting success. Poor soil conditions could be improved through amendments as necessary to optimize plant growth. Protection of plantings from herbivores and control of insect populations may be necessary to allow initial survival of young plant material.

5. DES MOINES CREEK

5.1 ECOLOGICAL ASSESSMENT OF IMPACT SITE

As mentioned in Chapter 1, the mitigation plan for the impacts to Des Moines Creek assumes that the layout for the SASA development described in the proposed Master Plan Update EIS would be built.

The impacts to wetlands in the SASA area have been considered in the development of the off-site wetland mitigation that is outlined in Chapter 3. Building the compensatory wetlands during the first phases of Master Plan Update would help ensure that they are functional by the time the actual impacts occur to the existing wetlands at the SASA site.

5.1.1 Stream Classification

As a tributary to Puget Sound, Des Moines Creek is designated as an extraordinary (Class AA) quality water body by the Water Quality Standards for Surface Waters of the State of Washington (WAC 173-201).

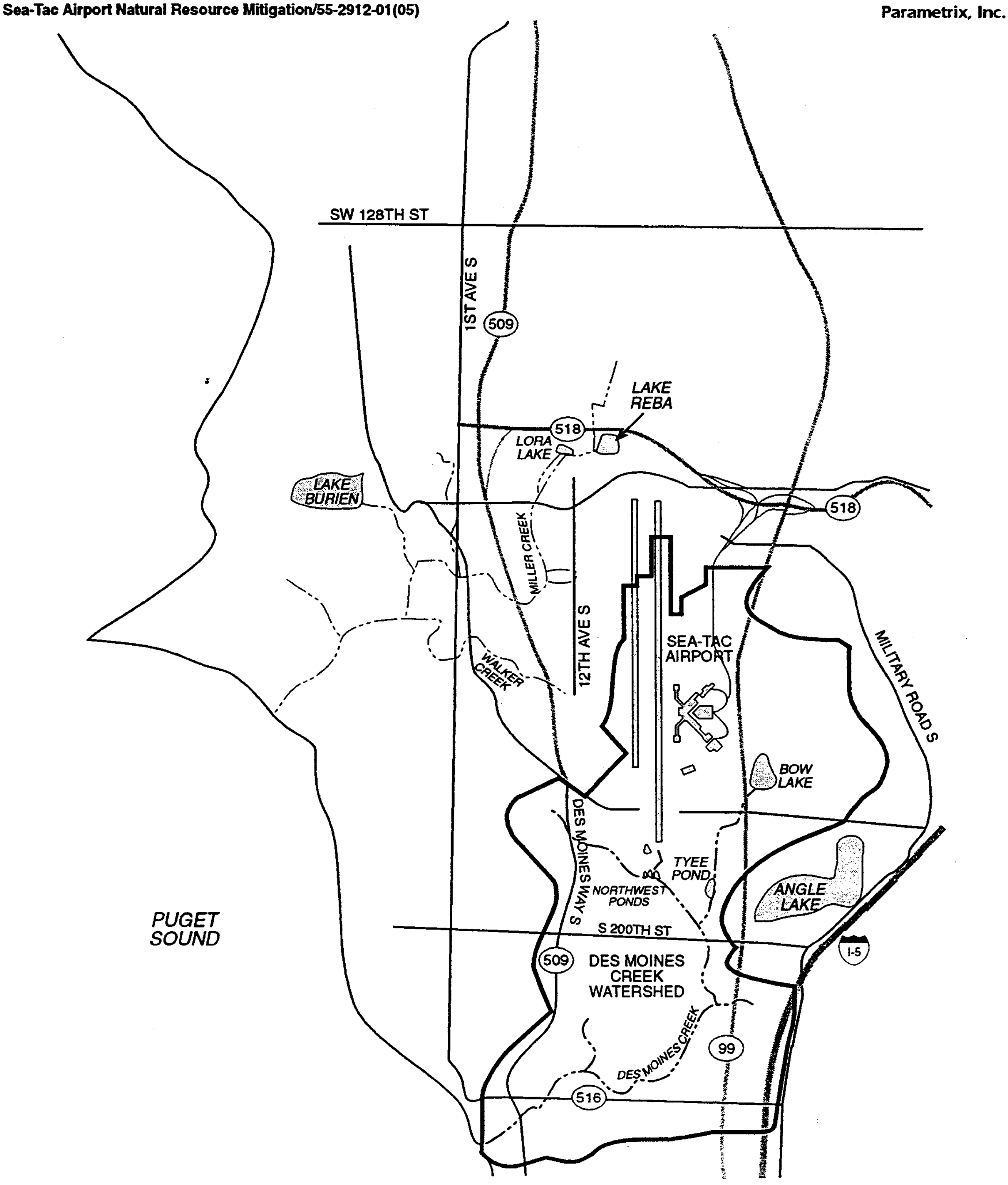
5.1.2 Primary Uses/Function in the Watershed

The Des Moines Creek watershed includes several large developed areas including parts of Sea-Tac International Airport, the city of SeaTac, and the city of Des Moines (Figure 5.1-1). Developed areas range from (1) highly impervious areas (mostly pavement and rooftops) found around the airport and commercial development along SR 99, to (2) areas with a moderate amount of impervious surfaces such as the residential areas in the city of Des Moines. Tyee Valley Golf Course and the airport clear zones are relatively undeveloped. The Des Moines Creek watershed extends from the northern parts of the airport at the headwaters to the mouth at Des Moines Beach Park on Puget Sound.

Des Moines Creek provides surface and storm water runoff conveyance and some limited fish habitat.

5.1.3 Existing Fish Habitat

The Des Moines Creek drainage basin consists of about 3,700 acres situated primarily south and southeast of the airport. The primary surface water conveyance in the basin is Des Moines Creek which originates from Bow Lake and extends about 3.5 miles southeast to Puget Sound, while dropping about 300 ft in elevation. Three major unnamed tributaries enter the creek at about river miles (RM) 0.7, 1.9, and 2.4 (Williams et al. 1975).



Source: Shapiro 1995e

Figure 5.1-1
Des Moines Creek
Basin

Des Moines Creek flows through a natural channel, except for the section at about South 200th Street, most of which is the SASA site. This reach includes a 3,600-ft-long culverted and channelized reach immediately downstream of Bow Lake. This culverted reach contains little or no salmon spawning or rearing habitat, although cutthroat trout and some warm-water fishes from Bow Lake may use it for some portion of their life stages. The streambed consists of silt and sand intermixed with small gravel. Bank vegetation in the open channel areas consists of very dense brush and small trees providing a good shade canopy.

The reach between RM 2.8 and South 200th Street (RM 2.1) flows through the Tyee Valley Golf Course. This reach is characterized by an open, grassy bank channel. The stream is culverted for 270 ft at the north end of the golf course and again at the outlet from the Tyee Pond at RM 2.4. The outlet structure appears to be a barrier to fish, and the stream channel provides little rearing habitat for fish. The detention facility is actually a large grass-lined bowl that the creek runs through during low flows; the facility only impounds water during storm events. This creek section consists primarily of a straight, narrow run reach (relatively shallow, fast-moving water) with virtually no pools, instream cover, or under-cut banks. As a result, there is very little fish-rearing habitat in this area.

Due to the presence of the golf course, and because of FAA and Port concerns about attracting birds to areas under the flight path, the canopy in this reach is largely absent. This lack of shade probably causes water temperatures to rise during summer months; this might be a problem for juvenile salmonids. The lack of trees also reduces the stability of the banks and results in excessive erosion and bank sloughing which increases the silt loading in the creek.

The golf course reach has limited salmonid spawning habitat and marginal rearing habitat. The wetlands on the west side of the golf course are probably not used extensively by juvenile salmon due to stagnant water and warm summer temperatures. Bass have been reported to inhabit these wetlands although the size of these populations or usage of the wetlands by other fish species is not known. These wetlands are connected to Des Moines Creek by an unnamed tributary at RM 2.4. This tributary is characterized by slow-moving water; soft, marshy banks; and heavy accumulation of fine sediment. The streambed in the rest of the golf course reach is predominantly sand and silt with some small patches of gravel and small cobble. Three drop weirs (dams) are located in the golf course just north of South 200th Street. The culvert at South 200th Street is flat-bottomed and at low flows its downstream end is higher than the plunge pool. This requires fish to leap into the culvert which at low flows does not have sufficient water depth for them to swim. The weirs, along with the box culvert under South 200th Street, might create problems for some fish. Although these barriers are probably not significant blockages for coho salmon, they might be for trout and other smaller fish species. In addition, the outlet control structure appears to be a barrier to most fish.

5.1.4 Hydrology

The Des Moines Creek watershed is highly urbanized and includes the cities of Des Moines, Normandy Park, SeaTac, and Burien. Sea-Tac Airport occupies approximately 20 percent of the watershed and is the watershed's dominant hydrological influence. The area directly southeast of the airport, once residential, has largely been purchased by the Port as part of the Noise Remedy Program. The Tyee Valley Golf Course occupies the area immediately south of the airport. The remainder of the watershed is mixed residential, commercial, and industrial uses.

The two branches of Des Moines Creek (formerly known as Bow Lake Creek) are shown on Figure 5.1-1. The west branch headwaters originate upstream of three wetlands areas, collectively identified as the Northwest Ponds on Figure 5.1-1. The west branch merges with the east branch approximately 1,200 ft north of South 200th Street.

The east branch headwaters originate from Bow Lake. Bow Lake provides significant flow attenuation before discharge to the east branch. The control structure that limits discharge from Bow Lake was evaluated to determine whether modifications could provide additional storage volume. The modeling conducted for the SASA EIS (Parametrix 1994) showed that Bow Lake is currently operating at its full capacity during the 100-year design storm.

After discharge from Bow Lake, Des Moines Creek flows through 2,000 ft of 36- to 54-inch storm sewers under South 188th Street and SR99/International Boulevard to the northwest corner of the SASA site, where it combines with pipes carrying runoff from SR99/International Boulevard and areas north and east of the airport. The creek comes into an open channel flowing west from the storm sewer in a narrow ravine that crosses the Alaska Airlines Training Facility parking lot. The creek corridor widens as it turns to the south. The creek then flows through several 84-inch-diameter, and smaller, culverts before discharging into the Tyee Detention Pond shown in Figure 5.1-1.

KCSWM constructed the Tyee Pond in 1989; it was a priority recommendation identified in the 1988 SeaTac Area Update for providing surface water flow controls in the Des Moines Creek basin. In addition to flow control, the pond was designed with an automatic shutoff gate and alarm that is activated by a hydrocarbon sensor. The shutoff gate was designed as a spill control device in response to two large jet fuel spills from the tank farm. The pond is "in-stream" which means that Des Moines Creek flows into the pond and out of the control structure at the south end of the pond. The pond has a peak capacity of 24 acre-ft. The outlet structure was designed to limit flows to non-erosive velocities during the 2-year frequency storm, and optimized to limit flooding for 25-year and 100-year storms. No stream flow data are available to compare flow rates before and after construction of the pond to verify performance.

The creek discharges from the detention pond control structure into an approximately 535-ft-long, 36-inch-diameter culvert. The 36-inch culvert discharges at the confluence of the creek with the west branch. The creek continues south under South 200th Street through wooded ravines approximately 2.25 miles to Puget Sound.

5.1.5 Floodplain

Because of the extensively developed area around the two main stream channels originating at Bow Lake and the Northwest Ponds and extending down to South 200th Street, no 100-year floodplain has been identified (Landrum & Brown 1995).

5.1.6 Existing Riparian Vegetation

With the exception of a short stretch of wooded area at the northeast corner of the SASA site, riparian vegetation along the section of Des Moines Creek flowing through the SASA site is primarily grass, which is regularly mowed.

5.2 CREEK MITIGATION GOALS, OBJECTIVES, AND PERFORMANCE STANDARDS

5.2.1 Goals and Objectives

Because of its large size, the proposed SASA development would require relocation of a portion of Des Moines Creek. Currently, the portion of the stream requiring relocation flows through a manmade channel. The primary mitigation goal is to retain the general drainage pattern within the Des Moines Creek valley. Improving fisheries habitat and access is a potential long-range goal.

To create a more natural stream course with increased fisheries habitat, the stream relocation plan could include habitat features such as meanders, weirs, spawning gravel, and streamside plantings. Meanders would replace the current straight alignment of most of the stream; shallow weirs would create small pools and riffles along the stream course. Plantings on the banks would shade the stream. Due to flight safety concerns, plantings would be selected to discourage use of the mitigation site by birds, particularly waterfowl, raptors, flocking birds, and blue heron.

5.2.2 Performance Standards

Specific performance standards would be similar to those proposed for Miller Creek (see Section 4.2).

5.3 PROPOSED MITIGATION SITE

5.3.1 Site Description

The available area for the relocation of Des Moines Creek is constrained by the development of the SASA site, the extension of Runway 34R and its RSA, and one possible alignment of the region's proposed South Access freeway. During the site planning for these projects, an appropriate corridor would be identified for Des Moines Creek. This corridor would include a 25-ft buffer on either side of the creek.

5.3.2 Ownership

The proposed mitigation site is owned by the Port of Seattle. The land the Tyee Valley Golf Course occupies is leased by the Port to the private golf course operator. The lease contains a special termination provision that gives the Port the option to reclaim all, or a portion of, the land to expand airport operations or facilities in the leased area. The lease expired in April 1992 and has been renewed on a month-to-month basis, subject to closure after 30-days notice.

5.3.3 Rationale for Choice

The new stream channel location would limit the amount of stream relocation required. The section to be relocated is limited primarily to reaches that are currently culverted, or that have been channelized or modified during previous development projects.

5.3.4 Ecological Assessment of the Mitigation Site

See Section 5.1 for an ecological assessment of the area.

5.4 MITIGATION SITE PLAN

5.4.1 Grading Plan

Construction of the streambed would require excavation, grading, installation of habitat features, stabilization, and planting of wetland and riparian areas prior to the diversion of water from the existing stream channel. The grading plan must identify channel elevations, riparian wetland elevations, and buffer contours. The plan would design the stream channel to contain normal flows. The floodplain grade would help detain peak flood flows. Creation of upland buffers would provide a variety of micro-environments to sustain different plant species. This enhanced habitat would be partially accessible to fish currently inhabiting habitat south of South 200th Street. The barrier (a culvert at South 200th Street) could be removed as part of future project mitigation, if stream cover on the SASA site has developed enough to discourage other wildlife use of the stream.

5.4.2 Hydrology/Hydraulics

An assessment of the soils underlying the location of the proposed streambed must be made to determine whether significant amounts of surface water would be lost through seepage. If potential losses would be significant, the streambed design may need a clay layer to ensure that base flows are adequately maintained.

Any road crossings would be designed as box culverts or tunnels with natural streambeds. Channel widths would be designed to allow low-flow levels suitable for fish passage through these structures.

The use of large boulders, root wads, or logs could stabilize areas likely to experience erosion. These erosion-prone areas include the outside bank at the beginning of a curve in the stream, immediately upstream and downstream of weirs, and opposite banks of habitat features that alter flows, such as root wads and digger logs.

5.4.3 Erosion Control

An erosion control plan should include proper placement of sediment control fences and hay bales during site excavation to reduce erosion during construction. Before diverting stream flow from the old channel to the new, streambeds and stream banks should be allowed to stabilize to limit erosion and prevent sedimentation of downstream areas during early flows. Hydroseeding and mulching areas outside of the stream channel would also reduce erosion.

Diversion of Des Moines Creek flows into the new channel should be delayed as long as possible to allow plants to establish. Irrigation will be necessary during this period. To prevent washout of the stream channel, riparian fringe, or upland buffer caused by flood flows, a temporary bypass could also be created. A culvert or ditch bypass would shunt portions of storm flows past the newly relocated stream, allowing establishment of riparian vegetation. After plants are well established (2 to 3 years), storm flows are less likely to erode stream banks and wash away planted vegetation, and the by-pass channel could be removed.

5.4.4 Habitat Structures

Several fish habitat structures would be included in the stream relocation effort. These are described below.

Shallow weirs constructed across the stream channel would oxygenate the water and create pools and riffles. The notched weirs concentrate flows within the notch during dry periods and dissipate flows over a large area during flooding. Pools that develop behind the weirs provide rearing habitat for fish. Dense overhanging vegetation would be planned to minimize use by waterfowl; birds pose a flight safety threat for airport operations. High flow downstream of the weir would maintain spawning gravel beds created during stream construction. Weirs should be placed at approximately 200-ft intervals. In some areas greater numbers of weirs may be required to increase low oxygen levels which might occur during storms.

Other habitat features that could be included in the stream design include boulders, root wads, and digger logs to provide cover for fish, reduce water velocities, and encourage pool formation. Placed midstream, large boulders create areas of low flow and shelter for fish. Root wads and digger logs placed near the edge of the stream bank and anchored by buried cement blocks and/or riprap would provide cover for fish and shade the stream, while log-covered and rock-covered overhangs would create additional fish habitat. Fish habitat structures would be placed at approximately every 50 to 100 ft of open channel.

5.4.5 Riparian and Buffer Plantings

The landscape design of the riparian and buffer areas should emphasize rapid development of plant cover to shade the stream and creation of floodplain benches for wetland plants, while minimizing the potential for attracting wildlife. Plants suitable for the stream riparian areas are listed in Table 5.4-1, while plants suitable for the upland buffer area are listed in Table 5.4-2.

Several strategies would ensure rapid development of shade along the relocated stream. The landscape design should concentrate plantings on the stream bank to ensure partial shading of the stream immediately following planting. Larger nursery-grown red alders could be planted along the stream channel to provide an immediate source of summer shade. Streamside plantings including fast-growing willow and red-osier dogwood should provide substantial shade within 3 years.

Stream banks and riparian areas could be planted with emergent wetland species. Wetter streamside areas could be planted with bulrush, and arrowhead. Riparian floodplains could be planted with burreed, small-fruited bulrush, and slough sedge. Willow and red-osier dogwood may be included in both streamside and floodplain plantings. Exposed areas between plantings would be hydroseeded with a mixture of grasses including fescue, water foxtail, colonial bentgrass, and clover.

Upland buffers could include a variety of plant species including red alder, cascara, western hazel, rose, and salal. Plant selection should favor species that would not attract birds, due to aircraft flight safety concerns. Small trees should be planted within the clear zone because of flight safety. The planting design should discourage human intrusion into the buffer by including thorn-bearing plants and/or split-post fencing. Exposed areas between plantings in the upland buffer should be hydroseeded with upland grass mixtures.

Table 5.4-1. Suggested plants for Des Moines Creek riparian fringe.

Scientific Name	Common Name
Trees	
Acer circinatum	Vine maple
Alnus rubra	Red alder
Fraxinus latifolia	Oregon ash
Salix scoulerana	Scouler willow
Shrubs	
Cornus stolonifera	Red-osier dogwood
Physocarpus capitatus	Pacific ninebark
Salix hookerana	Hooker willow
Salix lasiandra	Pacific willow
Salix sitchensis	Sitka willow
Spiraea douglasii	Hardhack spirea
Herbs	
Alopecurus geniculatus	Water foxtail
Athyrium filix-femina	Lady-fern
Carex obnupta	Slough sedge
Juncus ensifolius	Daggerleaf rush
Oenanthe sarmentosa	Water-parsley
Sagittaria spp.	Arrowhead
Scirpus microparpa	Small-fruited bulrush
Sparganium emersum	Simplestem bur-reed
Tolmiea menziesii	Pig-a-back-plant
Veronica spp.	Speedwell

Data compiled by Parametrix

Table 5.4-2. Suggested plantings for upland buffer communities.

Scientific	Common Name
Trees	
Alnus rubra	Red alder
Acer circinatum	Vine maple
Corylus cornuta	Western hazel
Rhamnus purshiana	Cascara
Shrubs	
Gaultheria shallon	Salal
Physocarpus capitatus	Pacific nine-bark
Rosa woodsii	Wood's Rose
Herbs	
Agrostis tenius	Colonial Bentgrass
Alopecuris geniculatus	Water foxtail
Festuc arunchinacea	Tall Fescue
Trifolium sp	Clover

Data compiled by Parametrix

5.4.6 Implementation Schedule

Site excavation and planting needs to occur prior to diversion of flows. To reduce the possibilities of erosion and potentially damaging storm flows, construction of the relocated stream channel is recommended during the dry season between mid-May and mid-October. [illegible] irrigation of plants would be required to ensure their survival. Plantings should occur [illegible]n mid-October and mid-May when water stress is low and transplant survival is highest. [illegible]ew stream channel would be constructed and fully stabilized before stream flow diversion [illegible] the old channel.

5.5 MONITORING PLAN

As stated in Section 5.2.2, in order to monitor and determine the success of the stream relocation project, a set of performance goals must be established. These performance standards allow determination of planting success, fisheries use, and retention of habitat features. Water quality parameters and flow rates would be monitored as identified in the storm water and water quality improvement facilities sections of this report.

Planting standards should include 100 percent shrub survivorship during the first year as guaranteed by the landscape contractor, and average survivorship rates of 85 percent over the first three years. Survivorship is determined by quantitative sampling of living tree and shrub species planted during the growing season. A series of 15 permanent transects should be established to measure percent cover; percent cover would be evaluated for the tree, shrub, and herb layer and identified for each species.

The success of fisheries habitat features, such as root wads, digger logs, weirs, and overhangs must be assessed. If greater than 10 percent of the designed habitat features are destroyed or significantly damaged, replacement should occur when the ecological benefits of the features outweigh the potential disturbance created by reconstruction.

While spawning habitat is included in this plan and would be constructed as part of the SASA-related stream relocation, the ultimate success of this element depends on future actions. Access to this habitat by downstream fisheries would be limited until modification or replacement of the South 200th Street culvert occurs. The monitoring program should assess the stability of this habitat; however, any failure may appropriately be addressed as part of future mitigation activities. Performance standards for the gravel spawning beds could assume retention of 50 percent of the spawning area placed in-stream. Extra bed area could be necessary to ensure success because some gravel beds are buried by silt or washed downstream. Qualified fisheries biologists should measure the area of gravel beds during site monitoring.

For the first five years following construction, annual reports would summarize the mitigation performance. The reports would evaluate performance standards, methods, and discussion of the reasons for success or failure of the restoration. The monitoring report would evaluate the success of the restoration, based on performance standards, and recommend appropriate action if standards are not met.

5.6 SITE PROTECTION

The site is owned by the Port of Seattle and would be designated in airport planning documents as a sensitive area to be protected in perpetuity.

5.7 MAINTENANCE AND CONTINGENCY PLAN

5.7.1 Hydrology

Maintenance of summer base flow and stormflows within design limits is essential. If flow rates exceed maximum rates, possible solutions could include the alteration of storm water control facilities to detain more water or construction of a permanent high-flow bypass channel.

A decline in low summer flows could result from the loss of water sources to the stream or infiltration into soils through the streambed. To determine the cause of water loss and identify appropriate action, a hydrologic analysis could be completed. Potential solutions to identified water losses could include increasing flow from other sources, and preventing loss of water from the streambed by using a synthetic liner. The current water sources and their flow rates suggest that significant loss of water flow appears unlikely.

5.7.2 Stream Channel

Due to the hydrologic forces associated with streams, some natural changes in the stream bank and channel are expected to occur. The plan would allow the stream channel to evolve without interference as long as it remains within the performance standards. Excessive erosion would require stabilization with plantings, logs, or other natural materials.

5.7.3 Habitat Features

Habitat features, such as root wads, digger logs, weirs, and overhangs, can be damaged by storm events, vandalism, or structural failure. If performance standards are not met, the damaged feature may be replaced. The monitoring ecologist may decide that replacement is necessary if: (1) replacing the structure coincides with the natural evolution of the stream; and (2) the ecological benefits of the habitat feature outweigh the disturbance created by its reconstruction. Any replacement will be done in conjunction with the U.S. Army Corps of Engineers.

If significant amounts of gravel beds are lost or silted, several corrective actions could be taken. New gravel beds could be created in high-flow areas as part of future project actions. Silted gravel beds could be cleaned, as long as the source of silt has been removed. However, extreme flow events in the SASA reach of Des Moines Creek are largely caused by conditions off-site in the upper watershed. Remedial measures due to high flows may need to be assessed jointly with upstream property owners, or through a city-wide rehabilitation program.

5.7.4 Plantings

Success of the restoration plan depends in a large part on the survival of the plantings. The landscape contractor, based on a guarantee required during the bidding process, would replace any plantings that die during the first year. If significant plant loss occurs, site conditions would be re-evaluated to determine whether the conditions can support the species planted.

Major factors likely to contribute to large-scale plant loss include improper hydrologic conditions, improper soil conditions, and pest infestation. Depending on the cause of the plant loss, a variety of remedial actions may be necessary to allow successful plant survival and restoration.

If necessary, site conditions can be altered to enhance planting success. Hydrologic conditions could be altered by adjustments to weirs or the outflow from the storm water detention facility. Poor soil conditions could be improved through amendments, as necessary, to optimize plant growth. Protection of plantings from herbivores and control of insect populations could be necessary to allow initial survival of young transplants.

6. REFERENCES

Adamus, P.R., E.J. Clairain, R.D. Smith, and R.E. Young. 1987. Wetland evaluation technique (WET), Volume II Methodology. Department of the Army, U.S. Army Corps of Engineers. Washington D.C. 206 pp.

Auburn, City of. 1987. Comprehensive zoning map. Auburn, Washington.

Brinson, M.M. 1993. A hydrogeomorphic classification for wetlands. Technical report WRP-DE-4. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

CH2M Hill and Associated Firms. 1995. Draft environmental impact statement for the Des Moines Creek Technology Campus. Lead Agencies, City of Des Moines and Port of Seattle, Seattle, Washington.

Canfield, R. 1941. Application of the line intercept method in sampling range vegetation. J. Forestry 39:388-394.

Cooperative Extension Service. 1968. Washington climate (King, Kitsap, Mason, Pierce counties). Washington State University, Pullman, Washington. 67p.

Cowardin, L.M., V. Carter, F.C. Golet, and E.T. LaRoe. 1979. Classification of wetlands and deep water habitats of the United States. Office of Biological Services, U.S. Fish and Wildlife Service, U.S. Department of Interior, Washington, D.C.

Daubenmire, R.F. 1959. Canopy coverage method of vegetation analysis. Northwest Sci. 33:43-64.

David Evans and Associates, Inc. 1989. Survey of soils with hydric characteristics, the Riverbend Estates–a master planned community. Unpublished report by David Evans and Associates to P. Hendley and Associates, Kent, Washington. 21p. + Appendices.

David Evans and Associates, Inc. 1995. Wetland determination on the Riverbend Master Plan Community. Unpublished report by David Evans and Associates to Gentra Capital Corporation, Seattle, Washington. 26p. + Appendices.

Department of Commerce. 1982. Climatography of the United States no. 18. Monthly normals of temperature, precipitation, and heating and cooling degree days (1951-80) Washington. National Climatic Data Center, Asheville, North Carolina.

Ecology. 1994. Guidelines for developing freshwater wetlands mitigation plans and proposals. Washington State Department of Ecology, Olympia, Washington. 37 p.

Ehrlich, P.R., D.S. Dobkin, and D. Wheye. 1988. The birder's handbook: a field guide to the natural history of North American birds. Simon & Shuster, New York, New York.

Environmental Laboratory. 1987. Corps of Engineers wetland delineation manual. Technical Report Y-87-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

FEMA (Federal Emergency Management Agency). 1989. Flood Insurance Rate Map, King County, Washington and Incorporated Areas. Panel 310 of 650, Map Number 53033c0310D, Effective Date September 29, 1989.

FICWD (Federal Interagency Committee for Wetland Delineation). 1989. Federal manual for identifying and delineating jurisdictional wetlands. USACOE, USEPA, USFWS, USDA Soil Conservation Service, Washington D.C. Cooperative technical publication. 76p.

Herrera Environmental Consultants. 1989. Des Moines Creek restoration project. Prepared for Municipality of Metropolitan Seattle by Herrera Environmental Consultants, Inc. and Hall and Associates. Seattle, Washington.

Hruby, T., and W.E. Cesanek, and K.E. Miller. 1995. Estimating relative wetland values for regional planning. Wetlands 15, No. 2: 93-106.

Johnson, A. 1989. Flow analysis and water quality related to salmonid in Des Moines Creek. Draft technical report to R.W. Beck and Associates for the Sea-Tac Master Drainage Plan. Watershed Dynamics, Inc., Auburn, Washington.

King County Surface Water Management Division. 1987. Reconnaissance report no. 12, Miller Creek basin. King County Surface Water Management Division, Seattle, Washington.

King County Surface Water Management Division. 1994. Mean daily discharge from Lake Reba outflow at Lake Reba detention pond. Water years 1990-1994.

Landrum & Brown. 1995. Draft environmental impact statement for proposed master plan update development actions at Seattle-Tacoma International Airport. Lead Agencies, Federal Aviation Administration and Port of Seattle, Seattle, Washington.

Montgomery Water Group, Inc. 1995. Hydrologic modeling study for SeaTac Airport master plan EIS. Appendix G of draft environmental impact statement for proposed master plan update development actions at Seattle-Tacoma International Airport. Lead Agencies, Federal Aviation Administration and Port of Seattle, Seattle, Washington.

Nussbaum, R.A., E.D. Brodie, Jr, and R.M. Storm. 1983. Amphibians and reptiles of the Pacific Northwest. University of Idaho Press, Moscow, Idaho.

P&D Aviation. 1991. Phase II: development of alternatives, Puget Sound Air Transportation Committee Flight Plan Project, final report. Prepared for the Puget Sound Air Transportation Committee by P&D Aviation. June 1991.

Parametrix. 1994. Final environmental impact statement for the South Aviation Support Area. Lead Agencies, Federal Aviation Administration and Port of Seattle, Seattle, Washington.

Parametrix. 1995. Proposed wetland mitigation site Phase I site assessment report. Prepared for Landrum & Brown by Parametrix, Inc., Seattle, Washington. 14p. + Appendices.

Parametrix. 1996. Environmental report: Port of Seattle master plan improvments wetland mitigation site, Auburn, Washington. Prepared for Landrum & Brown by Parametrix, Inc., Seattle, Washington. 28p.

Payne, N.F. 1992. Techniques for wildlife habitat management of wetlands. Biological Resource Management Series, McGraw Hill, New York, New York.

Pennak, R.W. 1989. Fresh-water invertebrates of the United States, 3rd ed. John Wiley & Sons, Inc., New York, New York.

Powers, P.D. and J.F. Orsborn. 1985. Analysis of barriers to upstream fish migration. Report prepared for Bonneville Power Administration by Albrook Hydraulics Lab. Washington State University, Pullman, Washington. DOE/BP-297.

Reimold, R.J. 1994. Wetlands functions and values. Chapter 4 in Applied wetlands science and technology. Donald Kent, editor. Lewis Publishers, Boca Raton, Florida.

Reppert, R.T., W. Sigleo, E. Stakhiv, L. Messman, and C. Beyers. 1979. Wetland values: concepts and methods for wetland evaluation. U.S. Army Corps of Engineers, Institute for Water Resources, Fort Belvoir, Virginia. 109p.

Richter, K.O., and A.L. Azous. 1995. Amphibian occurrence and wetland characteristics in the Puget Sound basin. Wetlands 15, No. 3: 305-312.

SeaTac, City of. 1993. Wetlands and streams classifications in the City of SeaTac–map folio.

Shapiro. 1995a. Stream survey report for Miller Creek. Appendix F of draft environmental impact statement for proposed master plan update development actions at Seattle-Tacoma

International Airport. Lead Agencies, Federal Aviation Administration and Port of Seattle, Seattle, Washington.

Shapiro. 1995b. Jurisdictional wetland delineation. Appendix H-A of draft environmental impact statement for proposed master plan update development actions at Seattle-Tacoma International Airport. Lead Agencies, Federal Aviation Administration and Port of Seattle, Seattle, Washington.

Shapiro. 1995c. Wetland function and values assessment. Appendix H-B of draft environmental impact statement for proposed master plan update development actions at Seattle-Tacoma International Airport. Lead Agencies, Federal Aviation Administration and Port of Seattle, Seattle, Washington.

Shapiro. 1995d. Elements of a conceptual wetland mitigation plan. Appendix P-A of draft environmental impact statement for proposed master plan update development actions at Seattle-Tacoma International Airport. Lead Agencies, Federal Aviation Administration and Port of Seattle, Seattle, Washington.

Shapiro. 1995e. Opportunities and constraint report for Miller Creek. Appendix P-B of draft environmental impact statement for proposed master plan update development actions at Seattle-Tacoma International Airport. Lead Agencies, Federal Aviation Administration and Port of Seattle, Seattle, Washington.

Sheldon & Associates and Butler & Associates. 1992. Sea-Tac Airport wetland mitigation plan. Unpublished report by Sheldon & Associates and Butler & Associates to the Port of Seattle, Seattle, Washington.

U.S. Department of Agriculture, Soil Conservation Service. 1952. Soil survey of King County, Washington.

U.S. Department of Agriculture, Soil Conservation Service. 1973. Soil survey of King County Area, Washington.

U.S. Department of Agriculture, Soil Conservation Service. 1991. Hydric soils of the United States.

WDFW. 1995. Unpublished letter of October 30, 1995 regarding threatened and endangered species at the wetland mitigation site. Washington Department of Fish and Wildlife, Olympia, Washington.

Williams, R.W., R.M. Laramie, and J.J. Ames. 1975. A catalog of Washington streams and salmon utilization. Volume 1, Puget Sound. Washington Department of Fisheries, Olympia, Washington.

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APPENDIX Q

WATER STUDIES

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AGI
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APPENDIX Q-A
BASELINE GROUNDWATER STUDY
FINAL ENVIRONMENTAL IMPACT STATEMENT
PROPOSED MASTER PLAN UPDATE
SEA-TAC INTERNATIONAL AIRPORT
SEATAC, WASHINGTON

January 3, 1996

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AGI TECHNOLOGIES

TABLE OF CONTENTS

TABLE OF CONTENTS

AGI TECHNOLOGIES

LIST OF TABLES

LIST OF ILLUSTRATIONS

EXECUTIVE SUMMARY

This report characterizes the baseline hydrogeology of the Sea-Tac International Airport and vicinity, and evaluates potential groundwater quality and quantity impacts from proposed improvements associated with the updated Sea-Tac Airport Master Plan. The proposed improvements are discussed in the draft Environmental Impact Statement (EIS), which was issued in April 1995. Most of the improvements involve the development of a third runway and additional terminal facilities. These improvements will require extensive importation and placement of fill that will be excavated from a number of borrow areas within the study area. The purpose of this report is to respond to comments on the draft EIS. As such, this report addresses impacts to the aquifers below the EIS study area resulting from the development of impervious areas associated with airport facility development and utilization of Port-owned borrow source areas.

HYDROGEOLOGY

The sediments in the study area have been divided into 10 stratigraphically distinct nonglacial and glacial deposits including (from youngest to oldest): Fill (Qaf), Alluvium (Qal), Vashon Recessional Outwash (Qvr), Vashon Till (Qvt), Vashon Drift, Vashon Advance Outwash (Qva), Lawton Clay (Qvl), Third Coarse Grained Deposit (Qc[3]), Third Fine Grained Deposit (Qf[3]), Fourth Coarse Grained Deposit (Qc[4]), Fourth Fine Grained Deposit (Qf[4]), and Tertiary Bedrock (Tbr).

The uppermost groundwater is perched within Alluvium, Recessional Outwash, and discontinuous porous zones of till. The primary aquifers in the study area occur as the Shallow (Qva), Intermediate (Qc[3]), and Deep (Qc[4]) Aquifers. Groundwater in the study area generally flows downward through each of the aquifers, and outward towards Puget Sound and the Green River Valley.

POTENTIAL IMPACTS

In areas where fill will be placed and compacted, recharge to the Shallow (Qva) Aquifer will be reduced by an estimated 0.18 million gallons per day (mgd). In borrow areas where the till will be removed to expose the Esperance Sand, Shallow (Qva) Aquifer recharge will be increased an estimated 0.32 mgd. The proposed improvements may therefore result in a net increase in recharge to the Shallow (Qva) Aquifer of an estimated 0.14 mgd.

Regional groundwater flow directions are not likely to change as a result of the improvements. Small changes in local groundwater flow, however, could occur in the borrow areas as a result of the possible elevation of the water table in these areas. These changes are likely to occur primarily in the Shallow (Qva) Aquifer.

Elevation changes of the Shallow (Qva) Aquifer water table in the borrow areas associated with increased recharge may result in temporarily increased discharge to nearby streams, and to upstream expansion of zones of perennial flow in Des Moines or Miller Creeks, where they intersect the Shallow (Qva) Aquifer.

Groundwater quality in the Shallow (Qva) Aquifer could potentially be impacted by the proposed improvements through either infiltration of contaminated surface water associated with construction activities or with future airport operations or borrow area development.

Potential construction-related impacts to water quality include a range of pollutants used during construction. The potential for construction impacts is considered low due to the relatively short period of construction and the likely requirement for implementation of best management practices.

Potential operational impacts to groundwater quality in the proposed runway and ancillary improvement areas are related to new impervious surface area and associated stormwater runoff. This potential is also considered low because of plans to convey new surface water runoff to Des Moines Creek and Miller Creek, thereby eliminating infiltration. Potential groundwater quality impacts due to future airport operations are primarily those resulting from the use or leakage of hazardous materials. The potential for these contaminants to infiltrate is considered low if best management practices are implemented.

Because of the potential for direct recharge to the Shallow (Qva) Aquifer within borrow areas, future development in the areas could potentially present significant water quality impacts to the groundwater system. Application of proper management techniques can reduce or eliminate the potential for groundwater contamination.

MITIGATION MEASURES - AQUIFER RECHARGE AND DISCHARGE

The results of our study indicate a net increase in recharge to the study area groundwater system may result from the proposed improvements. Little or no mitigation will likely be needed under these circumstances. However, Shallow (Qva) Aquifer discharge from borrow areas may result if seasonal water table elevations rise above the base of borrow area excavations. Containment of this potential discharge could be constructed such that this water is detained within the borrow area, or the base of the borrow pit could be kept above the seasonally highest water table.

MITIGATION MEASURES - GROUNDWATER QUALITY

Most potential impacts to groundwater quality associated with the airport improvements will likely be prevented by continued implementation of existing management plans and techniques, and those that will be adopted for the improvements.

For construction of airport improvements and the borrow areas, potential contamination spills can be mitigated by implementation of best management practices, phasing of construction activities, and conducting activities during the dry season.

As indicated in the draft EIS, various mitigation requirements stipulated by applicable laws, policies, and design standards will be implemented during construction and operation of the proposed airport developments. It is assumed that construction and operational impacts on water quality will be mitigated through implementation of National Pollutant Discharge Elimination System (NPDES) permit requirements, and other guidelines.

In the event of future development of the borrow areas, mitigation against potential groundwater quality impacts to the Shallow (Qva) and Intermediate (Qc[3]) Aquifers will be necessary. This mitigation could include preventing surface water run-on into the borrow areas from outside areas, reserving the borrow areas for activities with little or no potential for groundwater contamination, or developing the borrow areas with appropriate engineering controls.

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1.0 INTRODUCTION

1.1 GENERAL

This report characterizes the baseline hydrogeology of the Sea-Tac International Airport and vicinity and evaluates potential groundwater quality and quantity impacts from proposed improvements associated with the Sea-Tac Airport Master Plan Update. AGI was retained by the Port of Seattle (Port) under a subconsultant agreement with Shapiro Associates to perform this study in response to comments on the Draft Environmental Impact Statement (EIS) for the Master Plan update. The Draft EIS was prepared by the Port and the Federal Aviation Administration (FAA) and was issued in April 1995. Information in this report will be incorporated into a Final EIS.

The airport is located in SeaTac, Washington, approximately 12 miles south of downtown Seattle. The area considered by this hydrogeologic characterization is a subarea of the Draft EIS study area and is shown on **Figure 1**. The study area encompasses the Master Plan improvements.

1.2 BACKGROUND

Sea-Tac Airport was first developed in 1943 and began operating commercially by 1948. When opened, the airport consisted of four runways, with the main runway approximately 6,100 feet in length. By 1956, the main runway was lengthened to 11,900 feet, and during the 1960s and 1970s, extensive additions and improvements were made to the passenger terminal. From 1967 to 1973, a second parallel runway, the north and south satellite terminals, and the passenger terminal were constructed. Airport physical features have not significantly changed since that time.

Most of the development alternatives proposed by the Master Plan Update are associated with a proposed third runway and additional terminal facilities. These improvements will require extensive importation and placement of fill that will be excavated from a number of sites within the study area. Details of the improvements are described in the Draft EIS.

This report is intended to be a companion report of the EIS. The Draft EIS is therefore referenced extensively in discussions of the proposed improvements. Information in this study was also derived from a number of investigations focused on the airport vicinity. The reference section at the end of this report lists selected documents available from these investigations.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of the baseline hydrogeologic characterization is to identify the general hydrogeologic conditions of the EIS study area, based on existing hydrogeologic data, as a basis for evaluating effects of the proposed construction activities on groundwater recharge, quality, and flow. In particular, this study addresses impacts to the aquifers below the study area from increased impervious areas associated with airport facility development and from utilization of Port-owned borrow source areas. The specific objectives of the baseline hydrogeologic characterization are to:

- Characterize three-dimensional subsurface geology.
- Identify aquifers and aquitards.
- Characterize existing groundwater occurrence and movement, including recharge and discharge relationships.
- Qualitatively evaluate the impact of the proposed airport development on groundwater conditions.
- Identify mitigation measures, as appropriate.

To accomplish these objectives, we reviewed information obtained through meetings with the Port, Seattle Water Department, and Highline Water District (HWD). We also compiled and reviewed data from the following sources:

- Regional geologic literature
- Hydrogeologic studies of the Des Moines Upland
- Hydrogeologic studies of the Sea-Tac Airport vicinity
- Department of Ecology records
- Seattle Water Department records
- HWD records

The specific references we reviewed in preparing this report are listed in Section 5.0.

2.0 EXISTING CONDITIONS

2.1 REGIONAL PHYSIOGRAPHY

The study area is located on the Des Moines Upland within the Puget Lowland, a north-south trending structural and topographic depression bordered on the west by the Olympic Mountains and on the east by the Cascade Mountains. Sea-Tac Airport occupies approximately 2,500 acres of gently south- and west-sloping land near the crest of the Upland. Physiographic details of the study area are described in the Draft EIS (Port of Seattle, 1995). Topography of the study area is shown on **Figure 1**. Elevations at the airport range from approximately 350 to 420 feet above Mean Sea Level (elevations in this report refer to Mean Sea Level datum). Outside the study area, land surface elevations drop off steeply east and west to the Green River Valley and Puget Sound, respectively.

The study area includes watersheds of two streams: 1) Miller Creek, north and west of the airport, and 2) Des Moines Creek, south and southwest of the airport (**Figure 1**). The Des Moines and Miller Creek watersheds are discussed in the Draft EIS (Port of Seattle, 1995). The study area is primarily lightly to moderately forested land of mixed commercial, light industrial, and residential use. An undeveloped noise buffer area exists on the north, south, and west sides of the airport.

2.2 HYDROGEOLOGY

2.2.1 *Regional Geologic History*

The Des Moines Upland occurs as an elevated drift plain underlain by Quaternary glacial and non-glacial sediments and by Tertiary volcanic and sedimentary bedrock. Deposits of at least six glaciations have been identified in the Puget Lowland (Crandell, 1958; Easterbrook, 1967). The last of these major glaciations was named the Vashon. Armstrong, et al. (1965) renamed the youngest glaciation the Fraser, and modified it to include two glacial advances or stades, separated by one interstade. The youngest stade of the Fraser Glaciation is the Sumas and the oldest is the Vashon. Only deposits of the Vashon Stade are present in the study area.

The majority of surficial deposits and landforms in the study area can be attributed to fluvial, lacustrine, and direct ice contact processes associated with the advance and recession of the Vashon Glacier (Waldron, 1961, 1962). Glacial drifts from two older glaciations--Salmon Springs Glaciation and the older Stuck Glaciation--have also been mapped near the study area (Waldron, 1961, 1962), although more recent work by Easterbrook (1994) suggests that the widespread correlation of pre-Vashon deposits with Salmon Springs Drift may be invalid. Each of these glaciations had erosional and depositional processes similar to the Fraser Glaciation; consequently, deposits of the older glaciations often appear physically and hydraulically similar to those of Vashon age. Interglacial deposits commonly occur between glacial drift sequences and are often represented by volcanic ash, mudflow, and stream delta deposits.

2.2.2 *Study Area Geology*

Waldron (1962) completed the first surficial geologic map of the Des Moines 7.5 minute quadrangle, which includes the study area. His map shows deposits of the Vashon and Salmon Springs Glaciation and the Puyallup nonglacial sequence overlying Tertiary bedrock; no other pre-Vashon glacial or nonglacial deposits are recognized. However, a considerable number of geologic studies completed since 1962 in the Puget Lowland have suggested that additional glacial and nonglacial deposits occur between those of Vashon and Salmon Spring age (Easterbrook, et al., 1967; Luzier, 1969; Noble, 1990). In particular, geologic studies in the study area conducted for the South King County Groundwater Management Plan (SKCGMP) (South King County Advisory Committee, 1989) identified a number of previously unrecognized glacial and nonglacial sequences beneath the Des Moines drift plain. These include a nonglacial deposit between the Vashon and Salmon Springs drift, and a possible older glacial and nonglacial sequence beneath the Salmon Springs drift. Because the SKCGMP recognizes these additional deposits and presents the most comprehensive stratigraphic framework developed to date for the study area, this report generally follows the stratigraphic nomenclature used in the SKCGMP.

Sediments in the study area have been divided into 10 stratigraphically distinct deposits based on the SKCGMP nomenclature. Correlation of these deposits is based on common nomenclature in which upper Vashon and post-Vashon deposits are named based on their genesis, and deeper deposits are identified by their stratigraphic location and general particle size distribution. Study area deposits and their corresponding geologic map symbols are, from youngest to oldest:

- Fill (Qaf)
- Alluvium (Qal)
- Vashon Recessional Outwash (Qvr) ¦
- Vashon Till (Qvt) ¦ Vashon Drift
- Vashon Advance Outwash (Qva) ¦
- Lawton Clay (Qvl) ¦
- Third Coarse Grained Deposit (Qc[3])
- Third Fine Grained Deposit (Qf[3])
- Fourth Coarse Grained Deposit (Qc[4])
- Fourth Fine Grained Deposit (Qf[4])
- Tertiary Bedrock (Tbr)

These deposits are presented in order of increasing depth and age on the Generalized Stratigraphic Column shown on **Figure 2**. Surficial geology of the study area is shown on **Figure 3**. Generalized geology beneath the study area is depicted by Cross Sections A-A', B-B', and C-C', which intersect the study area as shown on **Figure 4**. Cross Sections A-A', B-B', and C-C' are shown on **Figures 5, 6, and 7**. Geology shown on these figures is based on well log information compiled from the references listed at the end of this report and is simplified to show general, large-scale subsurface relationships. Specific boring logs used to construct the cross sections are included in **Appendix A**. Actual geologic conditions are much more complex than depicted on the cross sections.

The following paragraphs describe the deposits in the study area in order of increasing depth and age.

Fill : Fill placed during construction of airport facilities is present over an extensive area as shown on **Figure 3**. Although fill is only shown on **Figure 3** as underlying the airport, fills also occur scattered throughout the study area supporting roads, buildings, and other structures. Fill deposits consist of a variety of earth materials, but typically comprise silty sand and gravel. Fill density ranges from loose in landscaped areas to dense where compacted below runways, roadways, and buildings. Fills in the study area may range up to approximately 30 feet in thickness.

Quaternary Alluvium (Qal) : Alluvium in the study area typically consists of loose fine-grained sand, silt, clay, and peat deposits, located in low-lying areas. These deposits are primarily associated with post-glacial fluvial and low energy depositional processes.

Vashon Recessional Outwash (Qvr) : Thin scattered deposits of Recessional Outwash occur below fill or at land surface across the study area. This deposit occurs in a variety of grain sizes, but is typically loose, coarse-grained sand and gravel. Recessional Outwash was primarily deposited by glacial meltwater streams near the front of the receding Vashon glacier.

Vashon Till (Qvt) : Vashon till underlies Recessional Outwash or fill where present, or is exposed at land surface in the study area. The till is typically very dense, and consists of a non-stratified, poorly sorted mixture of gray clay, silt, sand, and gravel, with occasional cobbles and boulders. The Vashon till is interpreted to have been deposited at the base of overriding Vashon glacial ice (lodgement till), causing its highly dense and compact character. The till typically averages approximately 10 to 50 feet in total thickness across most areas of the study area.

Vashon Advance Outwash (Qva) : Advance Outwash, also commonly named the Esperance Sand in the northern Puget Lowland, generally underlies Vashon Till, but also crops out at land surface in some parts of the study area. This deposit comprises beds of fluvial fine to medium-grained sand with minor gravel likely deposited in streams and lakes in front of the advancing Vashon ice. In comparison with the Recessional Outwash, the Advance Outwash is typically denser due to compaction beneath the overriding Vashon glacier. This deposit ranges from 50 to 150 feet thick in the study area.

Lawton Clay (Qvl) : This deposit is composed of beds of finely laminated to massive gray, brown, and blue-gray silt and clay, occurring beneath the Esperance Sand. This clay is absent in several locations beneath and north of Sea-Tac Airport, as shown on Cross Sections A-A' and B-B' (**Figures 5 and 6**). Regionally, the clay appears to pinch out southward. Lawton Clay was likely deposited in lacustrine environments. This deposit typically ranges from 50 to 100 feet thick where present in the study area.

Third Coarse-Grained Deposit (Qc[3]) : This deposit is ubiquitous throughout the study area, occurring below the Lawton Clay in most areas, and beneath the Esperance Sand where the Lawton Clay is absent. This deposit typically consists of a complex mixture of gravel, sandy gravel, and gravelly sand with varying proportions of silt and cobbles. Some drilled borings in the airport area have encountered wood debris and volcanic ash within this deposit. Qc(3) is interpreted by the SKCGMP and the Seattle Water Department (1990) to be outwash associated with the Salmon Springs Glaciation, and typically ranges from 50 to 250 feet thick in the study area.

Third Fine-Grained Deposit (Qf[3]) : This fine-grained deposit occurs immediately beneath the Salmon Springs. Qf(3) sediments are more heterogeneous than overlying deposits, but are characterized by fine to medium sand, silty sand, and silt fluvial deposits ranging in thickness up to several hundred feet. These sediments are thought to have been deposited during an interglacial period and to be correlative with the Puyallup Formation of Crandell, et al. (1958).

Fourth Coarse-Grained Deposit (Qc[4]) : This deposit typically consists of gravel and sandy gravel, and is likely associated with an older, pre-Salmon Springs Glaciation; however, its origin is uncertain.

Fourth Fine-Grained Deposit (Qf[4]) : This unit comprises predominantly silty clay which appears to occur uniformly below Qc(4) in the study area. The age and origin of the Fourth Fine-Grained Deposit is uncertain.

Tertiary Bedrock (Tbr) : The bedrock below the Des Moines Drift Plain is primarily arkosic, micaceous sandstone with interbedded shale and coal. The sandstone is reported to occasionally contain volcanic conglomerate, tuffaceous siltstone, tuff-breccia, and lava flows (South King County Groundwater Advisory Committee, 1989).

2.2.3 Aquifers and Aquitards

Groundwater in the study area occurs at least occasionally in each geologic deposit below ground surface. The uppermost groundwater occurs perched within Alluvium, Recessional Outwash, and discontinuous porous zones of the till. The primary aquifers in the study area, however, occur within the deeper glacial deposits, and are hydraulically delineated by the interposing deposits of glacial till or low permeability fine-grained sediments. Hydrostratigraphy of the study area is shown on the stratigraphic column (**Figure 2**).

Three deposits, Qva, Qc(3), and Qc(4), are considered the principal aquifers of the study area based on permeability and development as groundwater sources for water supply. These aquifers are identified as Shallow (Qva), Intermediate (Qc[3]), and Deep (Qc[4]). Cross sections A-A', B-B', and C-C' (**Figures 5, 6, and 7**) show these aquifers.

For this report we have generally adopted the aquifer names defined in Final Report; Highline Well Field Aquifer Storage and Recovery Project (Seattle Water Department, 1990). Study area stratigraphic deposits are defined hydrostratigraphically as follows:

• Fill (Qaf)	¦	
• Alluvium (Qal)	¦	Perched Zone
• Vashon Recessional Outwash (Qvr)	¦	
• Vashon Till (Qvt)	-	Aquitard
• Vashon Advance Outwash (Qva)	-	*Shallow (Qva) Aquifer*
• Lawton Clay (Qvl)	-	Aquitard
• Third Coarse Grained Deposit (Qc[3])	-	*Intermediate (Qc[3]) Aquifer*
• Puyallup Formation (Qf[3])	-	Aquitard
• Fourth Coarse Grained Deposit (Qc[4])	-	*Deep (Q[4]) Aquifer*
• Fourth Fine Grained Deposit (Qf[4])	-	Aquitard
• Tertiary Bedrock (Tbr)		

Hydrostratigraphy is shown on the Generalized Stratigraphic Column on **Figure 2.** Hydrostratigraphic units are described in the following paragraphs.

Perched Zone : Most of the perched groundwater in the study area occurs in Quaternary Alluvium and Recessional Outwash where they overlie the till. Groundwater is also occasionally perched within fill on top of till, or may be perched in discontinuous permeable zones within the till. These zones are generally seasonally present within a few tens of feet of land surface and have limited thickness and lateral extent.

First Aquitard : Where present in the study area, compact fill (Qaf) forms the uppermost aquitard restricting downward movement of water to underlying deposits. Over most of the study area, however, the Vashon Till (Qvt) forms the first significant aquitard. The fine-grained, compact nature of these deposits retards surface water infiltration and promotes runoff. Previous AGI studies indicate the vertical hydraulic conductivity of till in the study area is typically in the range of 10^{-5} to 10^{-7} cm/sec, which is several orders of magnitude less than that of the underlying Shallow (Qva) Aquifer (AGI, 1988).

Shallow (Qva) Aquifer : Groundwater in the Vashon Advance Outwash (Esperance Sand) comprises this uppermost aquifer. Groundwater in the Shallow (Qva) Aquifer generally occurs under unconfined (water table) conditions, and is typically protected from direct surface water infiltration by overlying fill or till. However, in some areas those upper deposits are absent, as shown on Cross Sections A-A', B-B', and C-C' (**Figures 5, 6, and 7**). The base of the Shallow Aquifer is between approximate Elevation 200 and 250, and its saturated thickness varies seasonally, typically ranging from approximately 50 to 75 feet. Water table elevations in the study area typically range from approximately 250 to 310 feet, or approximately 10 to 50 feet below ground surface.

The Shallow (Qva) Aquifer is considered to be of moderate permeability. Pumping test information reported by the South King County Groundwater Advisory Committee (1989) indicates a transmissivity of approximately 48,000 gallons per day per ft (gpd/ft). Water supply wells completed in the Shallow Aquifer may yield up to 500 gallons per minute (gpm) (South King County Groundwater Advisory Committee, 1989).

Qvl Aquitard : In most of the study area, the Shallow (Qva) Aquifer is separated from underlying aquifers by the Lawton Clay, which forms the Qvl Aquitard. Hydraulic conductivity of clays representative of the Lawton Clay are typically 10^{-7} to 10^{-10} cm/sec (Freeze and Cherry, 1979). The low permeability of the clay significantly retards flow between the overlying Qva and underlying Qc(3) Aquifer.

A window or gap in the Lawton Clay exists in the north portion of the study area as shown on Cross Section A-A' (**Figures 5**), and also in the middle and south portions of the study area where the Lawton Clay appears to pinch out to the south, as shown on Cross Section B-B' (**Figure 6**). In these areas, the Esperance Sand appears to directly overlie the Salmon Springs Drift, resulting in direct hydraulic connection between the Shallow and Intermediate Aquifers. These conditions may exist beneath portions of Sea-Tac Airport (see **Figure 6**), but existing data are inadequate to define this relationship.

Intermediate (Qc[3]) Aquifer : The Salmon Springs Drift has been studied as an important aquifer in the Des Moines Upland, and is extensively used for water supply. The City of Seattle Highline well field is completed in this aquifer. The aquifer exists under confined conditions where overlain by Lawton Clay. Unconfined conditions may occur south of the study area near Midway Landfill, where the Salmon Springs Drift is reported to occur at land surface (AGI, 1988).

The Intermediate Aquifer typically occurs between sea level and Elevation 200, with a saturated thickness ranging from approximately 50 to 250 feet. Water levels in wells screened in the Intermediate Aquifer are typically above the top of the deposit, but below water levels in the Shallow Aquifer.

Permeability of the Intermediate Aquifer is generally high. Aquifer test results for City of Seattle Highline wells indicate transmissivity of the Intermediate Aquifer in the study area ranges from 20,000 to 460,000 gpd/ft (Seattle Water Department, 1990; Hart Crowser, 1985b), and well yields of 1,500 to 3,000 gpm have been reported for Intermediate Aquifer production wells (South King County Groundwater Advisory Committee, 1989).

Qf(3) Aquitard : Fine-grained sand and silty sand below the Intermediate Aquifer form this aquitard. Significantly lower in permeability than the overlying Intermediate Aquifer, these fine-grained sediments retard downward movement from the Intermediate Aquifer; however, permeable zones within the aquitard may transmit appreciable volumes of water. The Qf(3) Aquitard typically occurs above approximately Elevation -100 and appears to range from approximately 50 to 100 feet thick beneath most of the study area.

Deep (Qc[4]) Aquifer : The Fourth Coarse-Grained Deposit forms the Deep Aquifer. The areal extent of this aquifer in the study area is not known; however, its depth is generally below Elevation -100. The Deep Aquifer is likely highly confined. Where the Deep Aquifer has been encountered, saturated thickness ranges up to 150 feet. Water levels in Deep Aquifer wells are typically above the top of the aquifer, but below water levels in Intermediate Aquifer wells.

Permeability of this aquifer is considered low to moderate, with reported transmissivities of approximately 2,000 to 30,000 gpd/ft (South King County Groundwater Advisory Committee, 1989; Hart Crowser, 1985b). Well yields for the more permeable portions of the Deep (Qc[4]) Aquifer range between 200 and 1,500 gpm (South King County Groundwater Advisory Committee, 1989).

Qf(4) Aquitard : The areal extent of the Qc(4) Aquitard in the study area is also not known. Silty clay comprises this deposit, and typically occurs below Elevation -150. The fine-grained nature of this deposit indicates it likely retards downward flow of groundwater from the Deep (Qc[4]) Aquifer.

2.2.4 *Groundwater Flow*

Upon entering the study area aquifers, groundwater generally flows outward toward the edges of the upland and downward from the Shallow (Qva) Aquifer to the Intermediate (Qc[3]) and Deep (Qc[4]) Aquifers (Luzier, 1969; South King County Groundwater Advisory Committee, 1989). It appears most groundwater eventually reaches Puget Sound to the west, or the Green River Valley to the east.

Local groundwater flow in the study area is complex, reflecting small-scale interlayering of glacial and nonglacial deposits within the subsurface deposits identified in **Section 2.2.** Local flow is also influenced by the distribution and magnitude of recharge and discharge, topography, water levels, and aquifer hydraulic properties. **Figures 8 and 9** show flow directions for the Shallow (Qva) and Intermediate (Qc[3]) Aquifers based on generalized aquifer potentiometric surface contours. Points shown on **Figures 8 and 9** are primarily compiled from the SKCGMP and Hart Crowser Technical Memorandum No. 1 - Summary of Data Review for Highline Well Field Study (1984a), respectively. Water level dates and well designations are not certain.

Groundwater flow in the Shallow (Qva) Aquifer generally appears to radiate outward from the highest portion of the upland toward the edges. A groundwater divide appears to be located east of the airport (**Figure 8**). The primary directions of flow are to the east toward the Green River Valley and to the west toward Puget Sound. In some areas, groundwater in the Shallow (Qva) Aquifer intersects ground surface and discharges to streams. Groundwater discharge is discussed in **Section 2.3**. Downward vertical flow also occurs from the Shallow (Qva) Aquifer, through the underlying Lawton Clay to the Intermediate (Qc[3]) Aquifer. Flow through the Lawton Clay is very slow due to its low permeability. However, in areas where the Lawton Clay Aquitard is absent, downward vertical flow from the Shallow (Qva) Aquifer to the underlying Intermediate Aquifer can occur more quickly.

Groundwater in the Intermediate (Qc[3]) Aquifer also generally flows outward from the crest of the Drift Plain (**Figure 9**). Like the Shallow (Qva) Aquifer, primary directions of flow within the Intermediate Aquifer appear to be east and west, and where the aquifer intersects ground surface, groundwater discharges to streams. Downward vertical flow also occurs in this aquifer, following the regional flow pattern described above. Some water in the Intermediate Aquifer likely eventually reaches the Deep (Qc[4]) Aquifer.

Groundwater flow in the Deep (Qc[4]) Aquifer is not known due to lack of wells completed in this aquifer.

2.3 GROUNDWATER RECHARGE AND DISCHARGE

2.3.1 *Groundwater Recharge and Discharge Areas*

Groundwater in the study area aquifers is recharged by infiltrating precipitation. Recharge occurs everywhere across the study area where impervious surfaces such as roadways, buildings, and airport runways do not exist and where groundwater does not discharge at ground surface. Recharge magnitude is largely governed by the permeability of the surface sediments and topography.

In relatively flat areas underlain by fine-grained, low permeability materials, such as till, peat, and compact fill, precipitation does infiltrate, but at very slow rates. These areas often contain bodies of water, including Angle Lake and Bow Lake (**Figure 1**). Sloped areas underlain by these same fine-grained deposits typically shed water at a much faster rate and allow less infiltration. In areas where till is overlain by alluvium or Recessional Outwash, infiltrating water may be temporarily detained in the Perched Zone. In contrast, areas underlain by coarse-grained sands or gravels allow considerable direct infiltration regardless of slope. These areas are typically considered recharge areas, and are represented in the study areas by exposures of Vashon Advance Outwash.

Figure 10 depicts our interpretation of existing recharge, discharge, and nonrecharge areas. Areas underlain by fill, till, or peat, and existing developed areas of the airport are considered nonrecharge zones despite the fact that some recharge does occur in these areas. Similarly, small-scale recharge/discharge features associated with local sloped and low-lying areas are not mapped. Areas with alluvium, Recessional Outwash, or Advance Outwash at the surface are considered to be recharge areas, except where the alluvium is predominantly peat or where discharge likely occurs. Most of the recharge areas shown on **Figure 10** are based on assumed direct surface exposure of Advance Outwash or absence of the till below the Recessional Outwash. Because boring log data indicate Advance Outwash likely reaches land surface in several locations across the study area (see **Figures 5, 6, and 7**), we assume areas mapped as Recessional Outwash on **Figure 2** (Luzier, 1969) are either areas where till is absent below the Recessional Outwash or where the outwash is actually Advance. In both cases we assume these areas represent direct recharge areas.

Infiltrating water passing through one of the identified recharge zones reaches the Shallow (Qva) Aquifer and provides direct recharge.

The Intermediate (Qc[3]) and Deep (Qc[4]) Aquifers are recharged by groundwater percolating downward from the Shallow (Qva) Aquifer. Most of the recharge from the Shallow to the Intermediate Aquifer probably occurs in areas where the Lawton Clay is absent, as shown on Cross Sections A-A' and B-B' (**Figures 5 and 6**).

Discharge from the study area aquifers primarily occurs as:

- Flow into perennial streams or springs discharging to Puget Sound or the Green River Valley, including Des Moines, Miller, and Walker Creeks, and other smaller, unnamed drainages.
- Underflow to the Green River Valley and Puget Sound.
- Pumping from municipal water supply wells in the Des Moines and Highline areas.

Figure 10 shows discharge areas within the study area. Des Moines and Miller Creek are the primary stream discharge and both generally sustain flow at their mouths throughout the year. While some of this water may come from seasonal water in the Perched Zone, the sustainable flow in these streams is largely attributable to baseflow discharging from aquifers identified in the study area. Below approximately Elevation 300, Des Moines Creek flows through exposures of the Shallow (Qva) and Intermediate (Qc[3]) Aquifers. Baseflow in this stream is therefore attributable to discharge from these aquifers. Miller Creek flows toward Puget Sound through exposures of the Shallow Aquifer west of the airport at elevations close to the water table in that area; some of Miller Creek's baseflow is therefore also likely due to discharge from the Shallow Aquifer.

Puget Sound and the Green River Valley are the other discharge areas for groundwater flowing downward and outward from the study area flow system. Discharge along the sea cliffs or walls of the Green River Valley forms springs. This discharge also likely occurs at depth as groundwater underflow to the Green River Valley and Puget Sound.

Groundwater possibly enters other aquifers not shown on Cross Sections A-A', B-B', and C-C'. However, previous studies indicate the Qf(4) Aquitard overlies, or is near Tertiary Bedrock (Tbr), which is thought to contain little groundwater.

Water supply accounts for a relatively small percentage of discharge from the study area groundwater system. Groundwater use from the Intermediate (Qc[3]) and Deep (Qc[4]) Aquifers is discussed in **Section 2.4.**

2.3.2 *Existing Water Balance*

Recharge and discharge relationships in the study area groundwater system are represented by the water balance schematic for the study area shown on **Figure 11.** The water balance indicates relative volumetric rates for recharge to and discharge from study area aquifers based on a simplified mass balance of the study area groundwater flow system. Generally, inflow enters the groundwater system as precipitation minus direct runoff, evaporation, and plant transpiration; water discharges from the groundwater system as baseflow to streams, as springs or underflow to the Green River Valley or Puget Sound, or as withdrawal from wells.

Inflow and outflow parameters used to develop the water balance are based on those used in previous investigations for the Des Moines Upland (Hart Crowser, 1985; South King County Groundwater Advisory Committee, 1989; Seattle Water Department, 1990). Averages of these parameters for the Des Moines Upland are listed below with volumetric rates based on the approximately 38,800-acre study area.

- Precipitation of approximately 39 inches per year (112.5 million gallons per day [mgd]).
- Evapotranspiration of approximately 17 inches per year, or 44 percent of precipitation (49 mgd).
- Runoff of approximately 8 inches per year, or 20 percent of precipitation (28 mgd).

Infiltration to the Shallow (Qva) Aquifer is the balance of water not lost to evapotranspiration or direct surface runoff as shown on **Figure 11.** The water balance assumes water entering the Shallow Aquifer either flows downward to the Intermediate (Qc[3]) Aquifer, or discharges to streams. Groundwater entering the Intermediate Aquifer either moves downward to the Deep (Qc[4]) Aquifer, out to streams, to Puget Sound or the Green River Valley, or to water supply wells. Similarly, Deep Aquifer groundwater flows to Puget Sound, the Green River Valley, or to water supply wells. Relative volumes of these flows are estimated as shown on **Figure 11.**

Total existing inflow to the Shallow (Qva) Aquifer in the study area is estimated to be approximately 35.5 mgd. Discharge from the study area aquifers that occurs as baseflow to streams is assumed to total approximately 5 mgd, based on data reported for Des Moines and Miller Creeks in SKCGMP and Seattle Water Department, 1990. Groundwater volumes discharged by wells are based on supply well production information discussed in the following section; these total approximately 4.5 mgd for the Shallow (Qva), Intermediate (Qc[3]), and Deep (Qc[4]) Aquifers. The balance of water in the groundwater system, approximately 26 mgd, is assumed to enter the Green or Duwamish River Valley or Puget Sound.

2.4 CURRENT GROUNDWATER USE

2.4.1 *Water Supply*

Each of the study area aquifers has been utilized historically as a source of groundwater for water supply. The Draft EIS states there is currently no known use of the Shallow (Qva) Aquifer water for drinking water supply in the study area; however, water rights information (discussed in **Section 2.4.2**) suggests there may be wells completed in this aquifer which may still be used for domestic, irrigation, commercial, or other uses. The Intermediate (Qc[3]) and Deep (Qc[4]) Aquifers are used by two major water purveyors for municipal water supply. The City of Seattle currently pumps from the Intermediate Aquifer via their Riverton Heights and Boulevard Park production wells located in the city's Highline Well Field located northeast of the airport. The HWD draws water from the Deep Aquifer via the Angle Lake and Des Moines production wells located south of the airport. Well locations are shown on **Figure 4**.

According to their respective records, the city's supply from the Intermediate (Qc[3]) Aquifer averages a total of approximately 1.5 mgd, and HWD's yield from the Deep (Qc[4]) Aquifer currently averages approximately 2.5 mgd. Total groundwater withdrawal by unknown or incidental wells throughout the area is not certain, but for purposes of the water balance we assume these do not exceed 0.5 mgd.

2.4.2 *Water Rights*

Current water rights issued by the Washington Department of Ecology for the study area are included in **Appendix B**. Rights to water supply in the study area provide for the following uses:

- Domestic
- Irrigation
- Commercial/Industrial
- Stock Watering
- Recreation and Beautification
- Fish Propagation
- Fire Protection

Approximately 40 percent of the listed water rights are for municipal and non-municipal wells. The remainder are designated for streams, springs, rivers, and lakes. The water rights information does not indicate which aquifers are screened by these wells; however, based on age and yield, it appears most non-municipal wells are likely completed in the Shallow (Qva) or Intermediate (Qc[3]) Aquifers. This study did not determine which water rights are being exercised; however, total yield from non-municipal wells is expected to be small compared with municipal withdrawals.

2.5 GROUNDWATER QUALITY

2.5.1 *General Groundwater Quality of Study Area Aquifers*

Representative general water quality data for the three study area aquifers are included in **Table 1**. Man's impact on Shallow (Qva) Aquifer groundwater quality is documented near the airport due to the many investigations of airport facility impacts in that area; these studies, however, do not typically identify general water quality parameters representative of background (non-impacted) conditions. Elsewhere, background water quality in the Shallow Aquifer is uncertain. **Table 1** shows data for several Shallow Aquifer wells as reported by Economic and Engineering Services, Inc. (1985). Shallow Aquifer groundwater is generally assumed to be of good quality (Port of Seattle, 1995).

Intermediate (Qc[3]) Aquifer water quality shown on **Table 1** is based largely on City of Seattle Highline Wellfield Studies (Seattle Water Department, 1990). Intermediate (Qc[3]) Aquifer water quality is generally considered to be excellent throughout most of the study area.

Deep (Qc[4]) Aquifer water quality is based on HWD records of recent testing. Based on these data and information in the Draft EIS, general water quality in the Deep (Qc[4]) Aquifer is excellent. The HWD data indicate manganese is occasionally elevated. However, naturally occurring manganese in the Deep (Qc[4]) Aquifer sediments are likely the source of these concentrations.

2.5.2 *Existing Contamination Sources*

Existing sources of contamination in the airport area are presented in the Draft EIS (Port of Seattle, 1995) and are documented in various airport area investigations (see Section 5.0). Several areas of known jet fuel hydrocarbon contamination exist in the Shallow (Qva) Aquifer near the airport. The Draft EIS reports this contamination has not migrated nor has it been identified at significant distances from its sources. Characterization and cleanup of these sources are reportedly underway (Port of Seattle, 1995).

There are also numerous sources of known and potential contamination throughout the study area outside of the airport. Commercial development along major transportation corridors and the overall increasing level of development in the area all pose potential long-term risk to groundwater quality in the Shallow (Qva) Aquifer and underlying aquifers. This risk cannot be quantified with the data available for the study.

Puget Sound is a potential source of high salinity to the Deep (Qc[4]) Aquifer, whereby high pumping rates in Deep Aquifer wells could reduce the hydrostatic pressure in this aquifer sufficiently to cause intrusion of Puget Sound water. Under these conditions, Deep Aquifer groundwater quality could deteriorate significantly.

2.5.3 *Contamination Receptors*

The contamination receptors of interest in the study area are currently operating water supply wells in the Intermediate (Qc[3]) and Deep (Qc[4]) Aquifers and Des Moines and Miller Creeks. Specific wells are the City of Seattle's Boulevard Park and Riverton Heights wells, which are completed in the Intermediate Aquifer, and HWD's Angle Lake and Des Moines wells, which are completed in the Deep Aquifer. Based on the groundwater system described in **Section 2.2.4**, contamination introduced at the ground surface may enter the Shallow (Qva) Aquifer, particularly in identified recharge areas. **Figure 10** shows areas where recharge conditions exist. Upon entry of contaminants to the Shallow Aquifer, direct or indirect downward flow routes could result in impacts to the underlying Intermediate, and possibly the Deep Aquifer. Although the Qvl and Qf(3) Aquitards significantly inhibit downward flow, areas where the Lawton Clay is absent provide a direct flow pathway from the Shallow to the Intermediate Aquifer.

3.0 POTENTIAL IMPACTS

3.1 PROPOSED IMPROVEMENTS

Improvements associated with the Master Plan Update are detailed in Section II of the Draft EIS. The EIS considers four alternatives; Alternative 1 is "Do Nothing" and is not considered further in this report. Alternatives 2, 3, and 4 consist of a new runway and associated taxiways or roads, and terminal facility improvements. The following basic elements are common to Alternatives 2, 3, and 4.

- A 7,000- or 8,500-foot-long by 150-foot-wide runway. The proposed runway will parallel the existing primary runway on the west. Runway grades will likely range between about Elevation 400 at the north end and about Elevation 350 at the south end.

- Other ancillary improvements, including: a safety area extending 250 feet west from the new runway centerline; a 75-foot-wide parallel taxiway situated 600 feet east of the proposed runway; and a 40-foot-wide perimeter access road with its centerline 285 feet west of the proposed runway centerline.

The three alternatives also include the following terminal improvements:

- Alternative 2: Centralized Terminal
- Alternative 3: North Unit Terminal
- Alternative 4: South Unit Terminal

Figure 12 shows existing airport facilities together with proposed improvements and borrow areas associated with Alternatives 2, 3, and 4.

Construction of the new runway and ancillary improvements associated with Alternatives 2, 3, and 4 will require importation and placement of substantial quantities of fill. Anticipated fill volumes and design thickness are referenced in Chapter 4, Section 24 of the EIS. Potential borrow areas for the new fill are located within Port-owned properties north and south of the airport. The borrow areas are shown as Areas 1 through 5 on **Figure 12**. The runway and ancillary facilities will be permanent. Long-term plans for the borrow areas are not currently defined.

Alternatives 2, 3, and 4 would each disturb surficial geology of the study area to some degree. Construction of the runway and other airport facilities will largely be completed by placing fill over native soil or other fill to reach design grades and foundations. Specifically, the 8,500-foot runway and other proposed improvements would result in approximately 193 acres of new impervious surfaced fill and 544 acres of unsurfaced fill area. The impervious area would be approximately 18 percent and less for the 7,000-foot runway than for the 8,500-foot runway (Port of Seattle, 1995). In the borrow areas, native soils will be removed for construction of the proposed airport facilities. **Table 2** summarizes the area and maximum volume of soil available from each borrow area.

3.2 POTENTIAL IMPACTS ON GROUNDWATER RECHARGE AND DISCHARGE

Construction and excavation associated with Alternatives 2, 3, and 4 will alter existing areas of recharge areas shown on **Figure 10**. In areas where fill will be placed and compacted, including the runway and airport facility improvements, direct surface water runoff will be increased and recharge reduced. According to the EIS, this water will be directed to Des Moines and Miller Creeks via stormwater management facilities. In borrow areas, recharge should increase since excavation will remove till and expose permeable Advance Outwash.

Alteration of recharge or discharge in the study area will change existing inflow to the groundwater balance depicted on **Figure 11**, and therefore will affect flow and volume in the Shallow (Qva), Intermediate (Qc[3]), and Deep (Qc[4]) Aquifers. Effects on groundwater recharge and discharge are discussed in more detail in the following sections.

3.2.1 *Aquifer Recharge Volume*

The new runway and airport facilities associated with Alternatives 2, 3, and 4 will generally be surfaced with impervious material, or be filled and compacted, significantly reducing surface permeability. With the 8,500-foot runway, approximately 97 acres of new impervious surface area and 262 acres of unsurfaced fill area would drain to Miller Creek, and approximately 95 acres of new impervious surface area and 283 acres of unsurfaced fill would drain to Des Moines Creek (Port of Seattle, 1995). For purposes of this study, we have assumed that all new fill areas will be nonrecharge areas (recognizing that some recharge does occur in these areas). **Figure 13** shows existing recharge areas defined by this study that would be filled by the proposed improvements, and thus be converted to non-recharge areas. The total reduction in recharge area based on **Figure 13** is approximately 77.5 acres (3,376,000 square feet).

Evapotranspiration and runoff in areas of direct recharge are less than the regionwide values used in **Section 2.3.2** due to more direct percolation of precipitation. For such areas, evapotranspiration and direct surface water runoff may each be estimated as approximately 10 percent of precipitation (Viessman, et al., 1989). Assuming these values, up to 31 inches of annual precipitation may infiltrate the recharge areas in **Figure 13** (39 inches minus 3.9 inches minus 3.9 inches). The reduction of 77.5 acres in recharge area would thereby reduce recharge to the Shallow (Qva) Aquifer approximately 0.18 mgd.

The Shallow (Qva) Aquifer is overlain by low-permeability till in portions of Borrow Areas 1, 2, 4, and 5. In these areas the till inhibits surface water infiltration to the Shallow Aquifer. In areas where the till will be removed sufficiently to expose the advance outwash, Shallow Aquifer recharge will be increased. Borrow Areas 1 and 5 appear to overlie zones in which the Lawton Clay is absent (see Cross Sections A-A', B-B', and C-C'); recharge from these borrow areas may also directly recharge the Intermediate (Qc[3]) Aquifer.

Current excavation plans suggest existing till will be completely removed from the borrow areas. **Table 2** provides estimates of the area of till that will likely be removed from each borrow area; **Figure 13** depicts these as recharge areas created by till removal. (Note that recent borrow studies indicate the till is not present in Area 3 despite its being mapped there on the surficial geology map (**Figure 3**). The total recharge area created by borrow area till excavation is approximately 158.3 acres (6,896,400 square feet). Assuming evapotranspiration and direct runoff total approximately 20 percent, as above, approximately 31 inches of precipitation would be available as direct recharge in the borrow areas as long as the excavations are unsurfaced and undeveloped. Total additional recharge to the Shallow (Qva) Aquifer associated with these new recharge areas would thereby total approximately 0.32 mgd. The estimated value of additional recharge per borrow area is included in Table 2.

In summary, our study indicates the Alternative 2, 3, and 4 improvements would reduce recharge approximately 0.18 mgd and borrow area development would increase recharge approximately 0.32 mgd. The balance of these effects indicates a net increase in recharge to the Shallow (Qva) Aquifer of approximately 0.14 mgd is likely as long as the borrow areas are undeveloped or unsurfaced.

3.2.2 *Aquifer Discharge Volume*

Discharge volumes from study area aquifers will increase in direct proportion to the increase in net recharge discussed above in Section 3.2.1. This increase will be expressed partly as greater discharge to Miller and Des Moines Creeks, and partly as greater underflow to Puget Sound and the Green River Valley. Greater discharge to the creeks would occur shortly after development; greater underflow would likely not be detectable for many years, perhaps centuries.

Greater discharge to area streams would be observable primarily near the proposed borrow areas, where increased recharge would cause the water table (Shallow [Qva] Aquifer) to rise. The rising water table would extend the area of perennial flow upstream and increase the volume of seepage into the stream.

The decrease in recharge associated with fill placement for the airport improvements might also have a localized effect on aquifer discharge. In the new fill areas the reduction in recharge could cause the water table to drop slightly, thus reducing seepage into either Des Moines or Miller Creeks. These effects should be offset by the greater discharge discussed above.

One other possible impact of increased recharge in the borrow areas is increased discharge if the water table rises to land surface and then flows out of the borrow area. This could only occur if the borrow area was excavated to below the seasonal high water table and an outlet was created for overflows.

3.2.3 *Groundwater Flow*

Regional groundwater flow directions are not likely to change as a result of the increased recharge associated with the Master Plan Update improvements. Small changes in local groundwater flow, however, could occur in the borrow areas through increased recharge. Elevation of the water table in these areas could result in higher hydraulic gradients than existing conditions, and therefore increase local groundwater velocities. Similarly, changes in groundwater discharge, particularly along segments of Des Moines and Miller Creeks, may temporarily change local flow directions toward the creeks. These effects are likely to occur primarily in the Shallow (Qva) Aquifer. Hydraulic gradients and groundwater velocity may also be reduced slightly below the proposed construction fill areas due to reductions in recharge.

3.3 GROUNDWATER QUALITY

Groundwater quality in the Shallow (Qva) Aquifer could be impacted by the proposed Alternative 2, 3, and 4 improvements through either infiltration of contaminated surface water associated with construction activities or with later airport operations or borrow area development. However, all of the potential impacts can be mitigated through proper planning and management.

Construction-Related Impacts : Potential construction-related impacts to groundwater quality associated with the airport runway and ancillary improvements would depend on local construction area size, the amount of exposed soil, topography, proximity to water bodies, and the effectiveness of erosion and sediment controls implemented. In the borrow areas, groundwater quality may be impacted by construction-related contaminants introduced by infiltrating surface water. In both the borrow and the airport improvement areas, the potential for construction impacts should be low based on the relatively short construction period and the restrictions likely to be applied by the permitting agencies.

Potential construction impacts on water quality include a range of substances used during construction, including fuels, lubricants, and other petroleum products, and construction waste such as concrete wash water. The Draft EIS identifies the potential for pollution resulting from accidental spills of these substances, from leaking storage containers, from refueling, and from construction equipment maintenance activities. The potential for these impacts should be minimized in areas of new impervious surfaces associated with the Alternative 2, 3, and 4 improvements.

Operations-Related Impacts : Operational impacts on groundwater quality in the proposed runway and ancillary improvement areas are related to new impervious surface area and associated stormwater runoff. The EIS reports that drainage from the new runway and taxiways would be detained on site and then conveyed to Des Moines Creek and Miller Creek. Potential impacts to surface water quality are discussed in Chapter IV of the Draft EIS. Essentially all of the new surface water runoff will leave the airport and not be available for infiltration. Thus, the potential for groundwater contamination from this source is low.

Potential groundwater quality impacts due to future airport operations in the improvement areas include those resulting from the use or leakage of hazardous materials (e.g., fuels and other petroleum products) stored at the airport. These contaminants could create conditions similar to those discussed in Section 2.5.2. However, the airport is currently undertaking studies aimed at reducing the potential for future groundwater quality impacts from this source.

In the borrow areas, operational impacts will depend on future development. The EIS reports the borrow areas may be cleared, graded, or surfaced; however, plans for the areas are currently undetermined. Because of the direct recharge to the Shallow (Qva) Aquifer from the borrow areas, future development in unsurfaced borrow areas could present significant water quality impacts to the groundwater system.

3.4 SUMMARY OF POTENTIAL IMPACTS

Potential impacts associated with Alternative 2, 3, and 4 improvements are summarized as follows:

Groundwater Recharge and Discharge Volumes

- In areas where fill will be placed and compacted, including the runway and airport facility improvements, direct surface water runoff will be increased and recharge reduced. This reduction in recharge to the Shallow (Qva) Aquifer is estimated to be approximately 0.18 mgd.
- In borrow areas where the till will be removed to expose the Esperance Sand, Shallow (Qva) Aquifer recharge will be increased. Total additional recharge to the Shallow Aquifer associated with these new recharge areas is estimated to total approximately 0.32 mgd.
- Alternative 2, 3, and 4 improvements may result in a net increase in recharge to the Shallow (Qva) Aquifer of approximately 0.14 mgd.
- Elevation of the Shallow (Qva) Aquifer water table in the borrow areas due to increased recharge may result in temporarily increased discharge to nearby streams, and to upstream expansion of zones of perennial flow in Des Moines or Miller Creeks, where they intersect the Shallow (Qva) Aquifer.
- A possibility exists for groundwater discharge directly out of the borrow areas if they are excavated below the seasonal high water table and an outlet is created for overflow.
- Borrow Areas 1 and 5 are in areas where the Lawton Clay is absent. Recharge in these areas may therefore directly affect the Intermediate (Qc[3]) Aquifer.

Groundwater Flow and Quality

- Regional groundwater flow directions are not likely to change as a result of the Master Plan Update improvements. Small changes in local groundwater flow, however, could occur in the borrow areas as a result of the possible elevation of the water table in these areas. These changes are likely to occur primarily in the Shallow (Qva) Aquifer.
- Groundwater quality in the Shallow (Qva) Aquifer could potentially be impacted by the proposed Alternative 2, 3, and 4 improvements through either infiltration of contaminated surface water associated with construction activities or with later airport operations or borrow area development.
- Potential construction impacts on water quality include a range of pollutants used during construction, including fuels, lubricants, and other petroleum products, and construction waste such as concrete wash water. The Draft EIS states pollution could result from accidental spills of these substances, from leaking storage containers, from refueling, and from construction equipment maintenance activities. The potential for construction impacts is considered low due to the short period of construction and implementation of best management practices.

- Operational impacts on groundwater quality in the proposed runway and ancillary improvement areas are related to new impervious surface area and associated stormwater runoff. This potential is also considered low because most stormwater will be transported off the airport and not be available for infiltration.

- Potential groundwater quality impacts due to future airport operations in the improvement areas are primarily those resulting from the use or leakage of hazardous materials (e.g., fuels and other petroleum products) stored at the airport. These contaminants could infiltrate similar to the existing contaminants discussed in **Section 2.5.2**. The potential for this to occur is considered low as described above.

- Because of the direct recharge to the Shallow (Qva) Aquifer from the borrow areas, future development in unsurfaced borrow areas could potentially present significant water quality impacts to the groundwater system.

- Application of proper management techniques can reduce or eliminate all the potential impacts listed above as sources of groundwater contamination.

4.0 MITIGATION MEASURES

Mitigation measures for impacts from construction and operation-related activities are discussed in the EIS, except where they relate to groundwater recharge or discharge. Mitigation measures identified by our study for potential impacts to groundwater are presented below.

4.1 AQUIFER RECHARGE AND DISCHARGE

Our study indicates a net increase in recharge to the study area groundwater system may result from the proposed Alternative 2, 3, and 4 improvements. Little or no mitigation will likely be needed under these circumstances. However, where Shallow (Qva) Aquifer discharge may result from seasonal water table elevations rising above the base of borrow area excavations, containment could be constructed such that this water is detained within the borrow area, or the base of the borrow pit could be kept above the seasonally highest water table.

4.2 GROUNDWATER QUALITY

Most potential impacts to groundwater quality associated with the airport improvements will likely be prevented by continued implementation of existing management plans and techniques, and those that will be adopted for the improvements.

For construction of the airport improvements and the borrow areas, potential contamination spills can be mitigated by implementation of best management practices such as construction waste handling plans and fueling and vehicle maintenance plans, and strict contractual requirements of contractors. Use of best management practices such as spill containment areas, phasing of construction activities (to minimize the amount of disturbed and exposed areas), and conducting activities during the dry season (April through September) also should prevent or reduce potential impacts on surface water and groundwater quality (Port of Seattle, 1995).

As indicated in the EIS, various mitigation requirements stipulated by federal, state, and applicable local laws, policies, and design standards, will be applicable to construction and operation of the new parallel runway development at the airport. It is assumed that construction and operational impacts on water quality will be mitigated through implementation of National Pollutant Discharge Elimination System (NPDES) permit requirements, detention requirements, and compliance with state waste and materials management requirements, water quality standards, and stormwater management guidelines (Port of Seattle, 1995).

Specific plans required as part of compliance with the Port's NPDES permit will need to be implemented to identify and control pollutants coming from the airport, and to prevent and control potential operational impacts on groundwater from industrial wastewater system (IWS) and storm drainage system (SDS) discharges.

In the event of future development of the borrow areas, mitigation against potential groundwater quality impacts to the Shallow (Qva) and Intermediate (Qc[3]) Aquifers will be necessary. This mitigation could include preventing surface water run-on into the borrow areas from outside areas, reserving the borrow areas for activities with little or no potential for groundwater contamination, or developing the borrow areas with appropriate controls.

5.0 REFERENCES

Applied Geotechnology Inc. (now AGI Technologies). 1988. Hydrogeology Technical Memorandum - Midway Landfill Remedial Investigation, Seattle Engineering Department - Solid Waste Utility. (Appendix A of Groundwater Technical Report.)

Armstrong, J.E.; D.R. Crandell; D.J. Easterbrook; and J.M. Noble. 1965. Late Pleistocene Stratigraphy and Chronology in Southwestern British Columbia and Northwestern Washington. Geol. Society of America Bulletin, V. 76, pp. 321-330, March 1965.

Burns and McDonnell Waste Consultants, Inc. 1993. Phase III Site Assessment Report for the United Airlines UST Fuel Farm at Seattle-Tacoma International Airport, Volume I, Seattle, Washington.

Burns and McDonnell Waste Consultants, Inc. 1993. Phase III Site Assessment Report for the United Airlines UST Fuel Farm at Seattle-Tacoma International Airport, Volume II, Seattle, Washington.

CH2M Hill. 1990. Pumping Test of the Intermediate Aquifer, Beverly Observation Well Location, Seattle, Washington.

CH2M Hill Northwest, Inc. 1989. Construction and Testing Report, Riverton Well No. 2, Seattle, Washington.

CH2M Hill. 1986. Highline Well Field, Glacier Site Development, Seattle, Washington.

Converse Consultants NW. Concourse D Groundwater Remedial Investigation, Seattle-Tacoma International Airport, Seattle, Washington.

Crandell, D.R.; D.R. Mullineaux; and H.H. Waldron. 1965. The Age and Origin of the Puget Sound Trough in Western Washington, in Geological Survey Research 1965: U.S. Geol. Survey Prof. Paper 525-B.

Crandell, D.R. 1963. Surficial Geology and Geomorphology of the Lake Tapps Quadrangle, Washington: U.S. Geol. Survey Prof. Paper 388-A, 84 p.

Crandell, D.R.; F.R. Mullineaux; and H.H. Waldron. 1958. Pleistocene Sequence in Southeastern Part of the Puget Sound Lowland, Washington; American Journal of Science, V. 256, June 1958, pp. 384-397.

Dames & Moore. 1993. Soils and Ground Water Investigation, Former PanAM Fueling Facility Sea-Tac Airport, Seattle, Washington.

Dames & Moore. 1993. Remedial Investigation of Soils and Ground Water B-Concourse, Sea-Tac Airport, Seattle, Washington.

Easterbrook, Don J. 1994. Chronology of Pre-Late Wisconsin Pleistocene Sediments in the Puget Lowland, Washington; Washington Division of Geology and Earth Resources, Bulletin 80, p. 191-206.

Easterbrook, D.S.; D.R. Crandell; and E.B. Leopold. 1967. Pre-Olympia Pleistocene Stratigraphy and Chronology in the Central Puget Lowland, Washington. Geol. Soc. of America Bulletin. V. 78. No. 1, pp 13-20.

Economic Engineering Services, Inc., Herrera Environmental Consultants. 1985. Highline Well Field Project Water Quality and Treatment Program, Technical Memorandum #4, Seattle, Washington.

Economic and Engineering Services, Inc. 1985. Highline Well Field Project Preliminary Water Quality Analyses of Shallow Aquifer, Supplement to Technical Memo #2 (Task 3B--Water Quality Sampling), Seattle, Washington.

ENSR Consulting and Engineering. 1994. Remedial Investigation and Feasibility Study at Former Continental Tank Farm, Sea-Tac Airport, Volume I, Seattle, Washington.

ENSR Consulting and Engineering. 1994. Remedial Investigation and Feasibility Study at Former Continental Tank Farm, Sea-Tac Airport, Volume II-Appendices, Seattle, Washington.

Freeze, R.A.; and J.A. Cherry. 1979. Groundwater. Prentice Hall Inc., Englewood Cliffs, N.J. 604 pages.

Hart Crowser, Inc. 1993. Further Site Characterization, Former Autogas Tank Cluster, Sea-Tac International Airport, Seattle, Washington.

Hart Crowser, Inc. 1989. Site Assessment, Sea-Tac International Airport, Seattle, Washington.

Hart Crowser. 1988. Updated Analysis of Highline Aquifer Monitoring Data, Seattle, Washington.

Hart Crowser & Associates, Inc. 1986. Hydrogeologic Evaluation of Riverton Heights Production Well No. 1, Highline Well Field Study, Phase I, Seattle, Washington.

Hart Crowser & Associates, Inc. 1985a. Summary Letter for Exploratory Drilling and Testing, Highline Well Field Study, Seattle, Washington.

Hart Crowser & Associates, Inc. 1985b. Technical Memorandum No. 4, Hydrogeological Assessment with Results of Exploratory Drilling and Testing Program, Highline Well Field Study, Seattle, Washington.

Hart Crowser & Associates, Inc. 1984a. Technical Memorandum No. 1: Summary of Data Review, Highline Well Field Study, Seattle, Washington.

Hart Crowser & Associates, Inc. 1984b. Technical Memorandum No. 2, Field Investigations, Highline Well Field Study, Seattle, Washington.

Hong West & Associates, Inc. 1994. Port of Seattle Former PanAM Facilities Remedial Investigation Draft Report, Volume 1 of 2, Seattle, Washington.

Luzier, J.E. 1969. Geology and Ground Water Resources of Southwestern King County, Washington; Washington Department of Water Resources Water Supply Bulletin No. 28.

Mullineaux, D.R.; H.H. Waldron; and M. Rubin. 1965. Stratigraphy and Chronology of Late Interglacial and Early Vashon Glacial Time in the Seattle Area, Washington; Contributions to Stratigraphy: U.S. Geol. Survey Bulletin 1194-O.

Noble, J.B. 1990. Proposed Revision of Nomenclature for the Pleistocene Stratigraphy of Coastal Pierce County, Washington. Washington Dept. of Natural Resources, Open File Report 90-4.

Port of Seattle, U.S. Department of Transportation, Federal Aviation Administration. 1995. Draft Environmental Impact Statement for Proposed Master Plan Update Development Actions at Seattle-Tacoma International Airport. Volumes 1, 2, and 3.

Richardson, D.; J.W. Bingham; and R.J. Madison. 1968. Water Resources of King County, Washington. USGS Water Supply Paper 1852.

RZA AGRA, Inc. 1993. Northwest Airlines Fuel Farm and South Satellite Tunnel Complex, Sea-Tac International Airport, Sea-Tac, Washington.

RZA AGRA, Inc. 1992. Sea-Tac Airport Facilities Bulk Fuel Farm, Sea-Tac International Airport, Seattle, Washington.

Seattle Water Department, CH2M Hill, Economic and Engineering Services, Inc., and Herrera Environmental Consultants. 1990. Highline Well Field Evaluation; Volume 1.

Shannon & Wilson, Inc. 1991. Supplemental Agreement 1 Environmental Investigation Draft Report, Pan AM Fuel Farm, Sea-Tac International Airport, Seattle, Washington.

South King County Ground Water Advisory Committee; Economic and Engineering Services Inc., Hart-Crowser, Inc., Pacific Groundwater Group, and Robinson and Noble, Inc. 1989. South King County Ground Water Management Plan. Volumes I and II.

Viessman, W.; G.L. Lewis; and J.W. Knapp. 1989. Introduction to Hydrology, 3rd ed., Harper and Row Publishers Inc., New York. 780 pp.

Waldron, Howard H. 1962. Geology of the Des Moines Quadrangle, Washington, U.S.G.S. Geologic Map GQ-158.

Waldron, Howard H. 1961. Geology of the Poverty Bay Quadrangle, Washington, United States Geological Survey Quadrangle Map GQ-158.

Waldron, H.H.; D.R. Mullineaux; and D.R. Crandell. 1957. Age of the Vashon Glaciation in the Southern and Central Parts of the Puget Sound Basin, Washington [abs.]: Geol. Soc. America Bull., v.68, no. 12, pt. 2, pp. 1849-1850.

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for Mackey Smith, C.E.G.
Executive Vice President

AGI TECHNOLOGIES

Table 1
General Groundwater Chemistry
Shapiro/Sea-Tac EIS
SeaTac , Washington

Aquifer	Specific Conductance (µmhos/cm)	TDS	Alakalinity (mg/L)	Color	Iron (mg/L)	Manganese (mg/L)	Hardness (mg/L)	Sodium (mg/L)	Sulfate (mg/L)	Chloride (mg/L)	Nitrate as N (mg/L)
Shallow Aquifer Hellums, Ellaston, Washington Memorial Park, Brittenbach, & Gingrich Wells (E.E.S., 1985)	244 - 358	N/A	79.2 - 142	N/A	0.01 - 0.3	0.0 - 0.23	21.7 - 156	5.9 - 10.8	N/A	N/A	0.3 - 4.2
Intermediate Aquifer Boulevard Park and Riverton Heights Wells (Seattle Water Department, 1990)	153 -289	160 -195	N/A	3 - 5	0.04 - 0.6	0.012 - 0.065	N/A	4.78 - 8.12	4.7 - 35	3.0 - 7.55	0.02 - 3.3
Expected water quality based on historical analysis of inorganic parameters (E.E.S., 1984)	150	N/A	80	N/A	0.1 - 1.0	0.03 - 0.10	70	6	8	5	0.1
Deep Aquifer Angle Lake & Des Moines Wells (7-26-95) (Personal Communication - Highline Water District)	154 - 259	N/A	N/A	<5	<0.03	0.067 - 0.093	61 - 91	9 - 10	N/A	2 - 7	<0.02

Notes:

mg/L - Milligrams per liter.
N/A - Not available.
µmhos/cm - Micromhos per centimeter.

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Table 2

Summary of Borrow Area Acreage, Fill Volume, and Estimated Recharge

Shapiro/Sea-Tac EIS

SeaTac, Washington

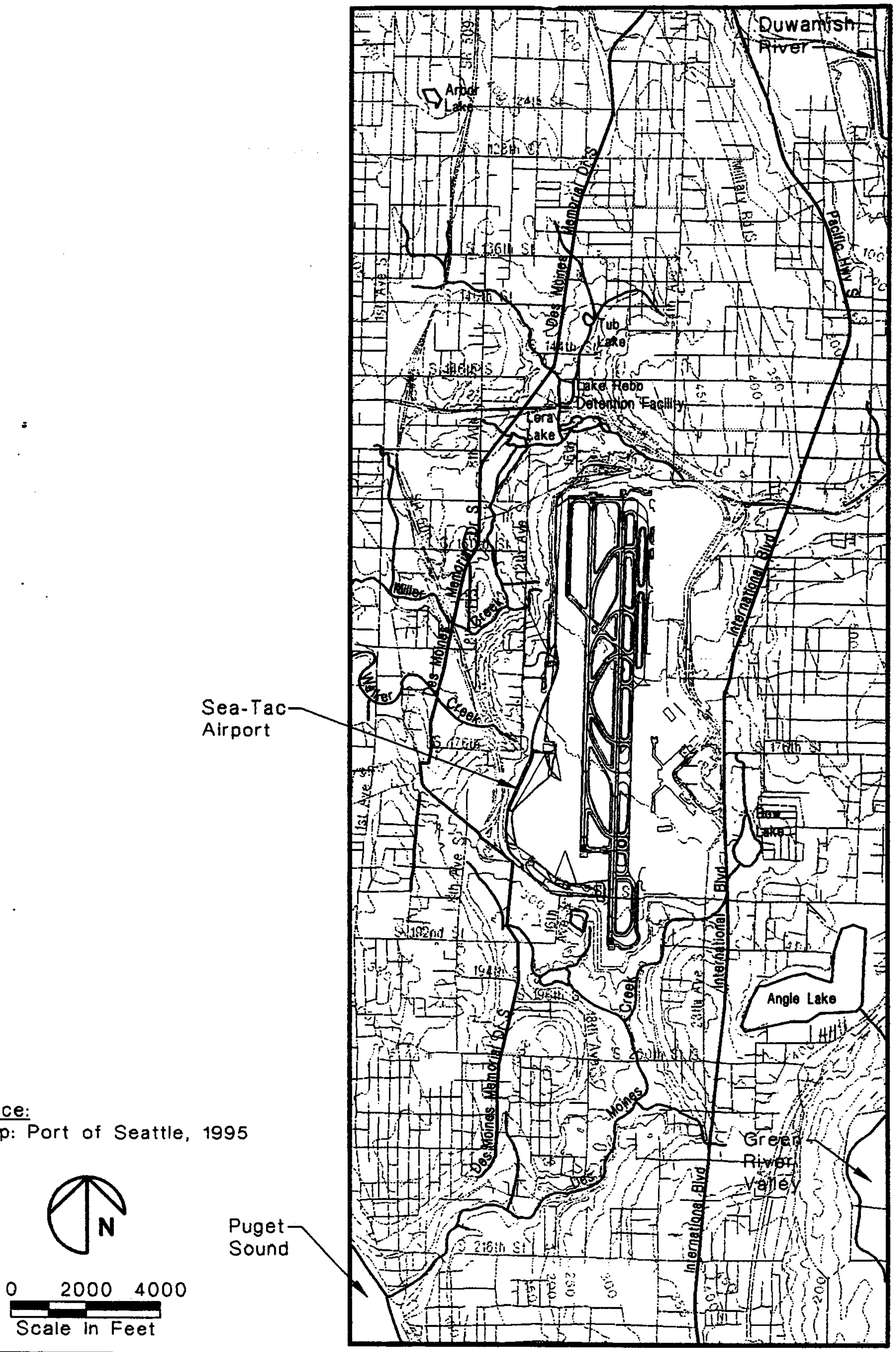
Area	Approximate Ground Surface Elevation Range[a] (feet)	Approximate Acreage	Maximum Volume of Soil Available for Excavation[b] (million cubic yards)	Approximate Area of Till (Qvt) to be Removed (square feet)	Additional Recharge to Groundwater Due to Removal of Till[c] (gallons/day)
1	250 to 350	110	0.5	3,537,215	188,471
2	175 to 275	20	0.65	450,552	24,006
3	250 to 350	60	2.90	0	0
4	290 to 395	40	2.20	447,595	23,849
5	275 to 475	60	1.75	1,523,534	81,177
TOTAL		290	8.0	5,958,896	317,503

Notes:

a) NGVD 1929 Datum.

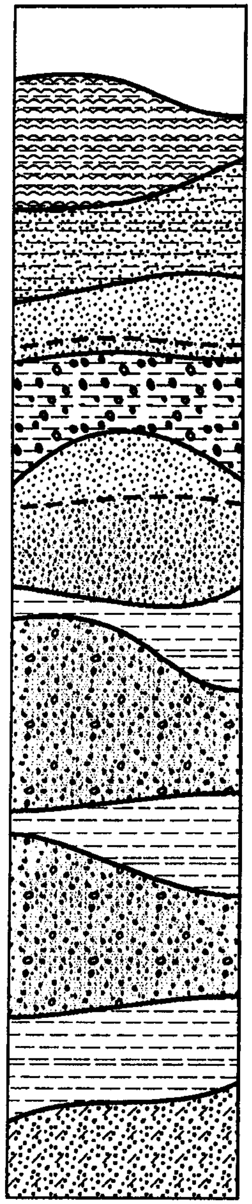
b) From Shapiro, 1995.

c) Based on 39 inches precipitation, minus 10% evaporation and 10% runoff.



Reference:
Basemap: Port of Seattle, 1995

AGI TECHNOLOGIES	**Study Area** Shapiro/Sea-Tac EIS Seatac, Washington			**AR 003398**			FIGURE **1**
studyar.dwg	PROJECT NO. 14,887.003	DRAWN BJA	DATE Nov 95	APPROVED	REVISED		DATE



Symbol	Stratigraphic Unit As Identified in SKCGMP	Geologic Description and Regional Correlation	Hydrostratigraphic Unit Modified from HWFASR
Qaf	Fill	Miscellaneous surficial fills	**Perched Zone** Seasonally perched groundwater occurs at base of fills on top of till.
Qal	Recent Alluvium	Primarily fine grained sand, silt, clay, and peat deposited along stream channels and valley bottoms.	Seasonally perched groundwater occurs at base of alluvium at top of till.
Qvr	Vashon Recessional Outwash	Scattered deposits of well sorted sand and gravel. Typically include outwash deposits.	Seasonally perched groundwater occurs at base of Recessional Outwash on top of till.
Qvt	Vashon Till	Compact mixture of gravel in a gray clayey, silty sand matrix, with occasional boulders and lenses of sand and gravel. Typically mantles older Vashon glacial and nonglacial deposits in study area.	**Aquitard** Primarily of low permeability except for isolated lenses of sand which may contain water seasonally. Typically averages 10 to 50 feet thick.
Qva	Vashon Advance Outwash	Predominantly sand in study area. Locally may include very fine sand and silt (Esperance Sand of Mullineaux, 1965; Colvos Sand of Molenaar, 1963).	**Shallow [Qva] Aquifer** Moderately permeable aquifer with typically abundant water; primarily unconfined. Typically 50 to 150 feet thick.
Qvl	Lawton Clay	Glacio-lacustrine deposit primarily composed of laminated clayey silt, silty clay, silt, and fine sand (Mullineaux, 1965).	**Aquitard** Low permeability deposit which impedes downward flow to Qc(3) Aquifer. Typically 50 to 100 feet thick.
Qc(3)	Third Coarse-Grained Deposit	Typically oxidized glacial outwash sand and gravel (Salmon Springs Drift of Crandell, et al.,1958).	**Intermediate [Qc(3)] Aquifer** Typically saturated, high permeability aquifer, primarily confined. Regionally important for water supply. Supplies Seattle Water Department Highline Well Field.
Qf(3)	Third Fine-Grained Deposit	Composed primarily of fine to medium silty sand. Contains andesite grains imparting characteristic lavender hue (Puyallup Formation of Crandell, et al., 1958).	**Aquitard** Typically low permeability relative to overlying and underlying aquifers. Impedes vertical flow between Qc(3) and Qc(4) Aquifers. Thickness typically 50 to 250 feet.
Qc(4)	Fourth Coarse-Grained Deposit	Coarse grained deposits (formation uncertain).	**Deep [Qc(4)] Aquifer** High permeability, confined aquifer. Thickness uncertain. Regionally used for water supply. Supplies Highline Water District wells.
Qf(4)	Fourth Fine-Grained Deposit	Primarily silty clay (formation uncertain).	**Aquitard** Low permeability deposit which impedes downward flow. Thickness uncertain.
Tbr	Tertiary Bedrock	Principally arkosic, micaceous sandstone and interbedded shale and coal. Locally includes thick sequence of volcanic sandstone and conglomerate, tuffaceous siltstone, tuff-breccia, and lava flows (Puget Group of Waldron, et al., 1962).	**Bedrock** Hydraulic characteristics uncertain.

Notes:
SKCGMP - South King County Groundwater Management Plan (South King County Groundwater Advisory Committee, 1989)

HWFASR - Highline Well Field Aquifer Storage and Recovery Project (Seattle Water Department, 1994)

☐ Water Bearing Deposits

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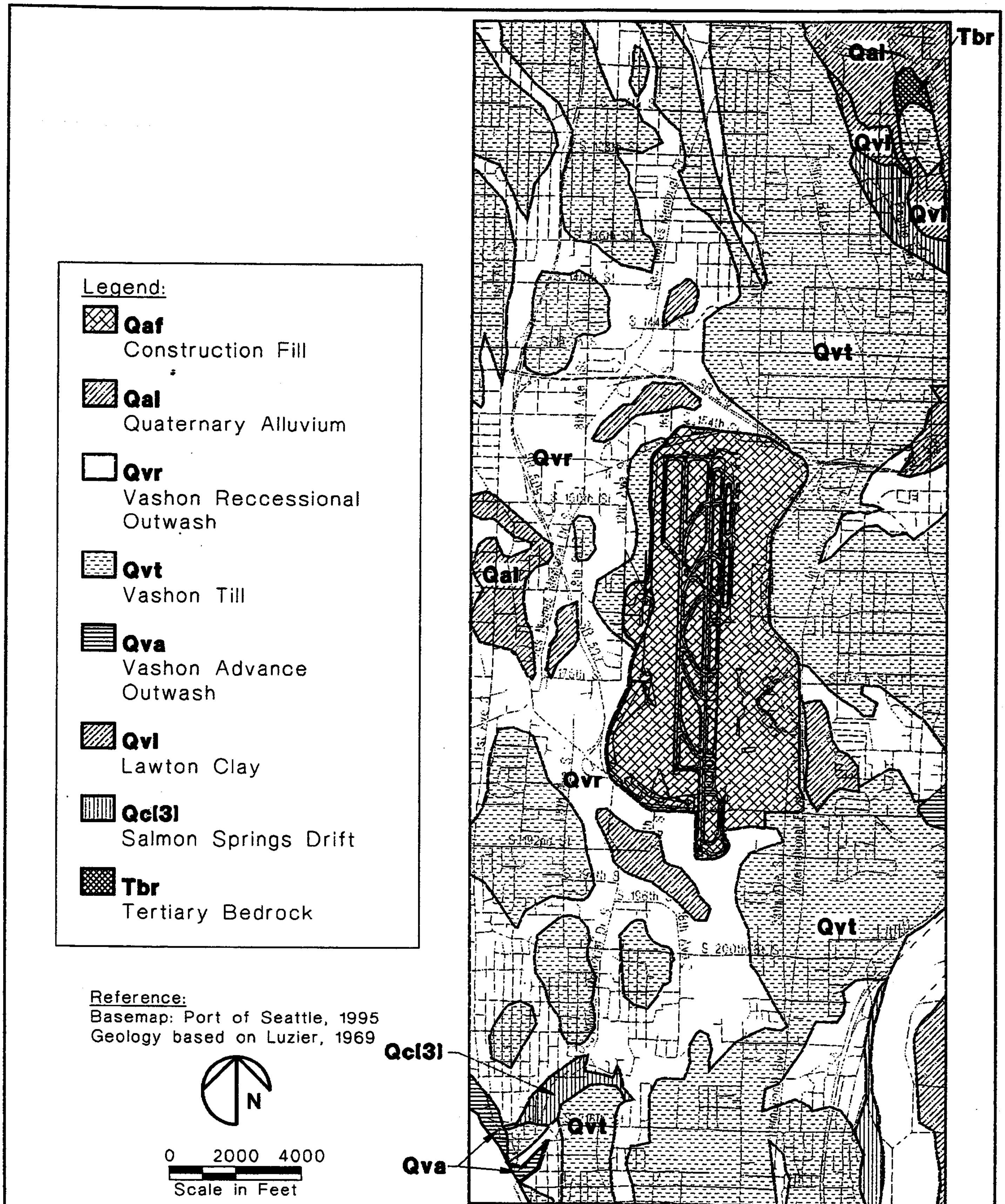
Generalized Stratigraphic Column
Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE **2**

	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
stratcol.cdr	14,887.003	BJA	Nov 95	IWWS		



AGI TECHNOLOGIES

geology.dwg

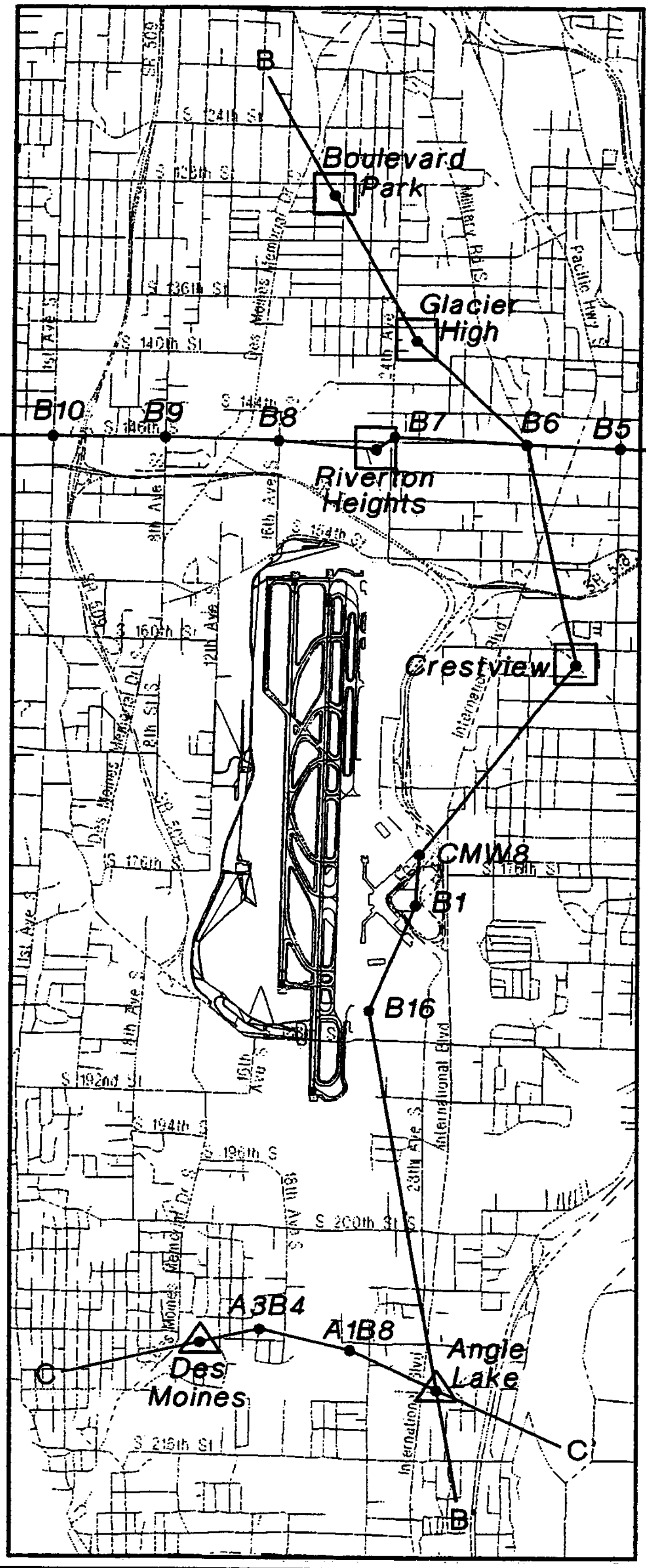
Surficial Geology

Shapiro/Sea-Tac EIS
Seatac, Washington

AR 003400

FIGURE **3**

PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
14,887.003	BJA	Nov 95			



Legend:

- ☐ Seattle Water Department Water Supply or Test Well (Highline Well Field)
- △ Highline Water District Water Supply Well
- • Boring/Well from Previous Investigations

Reference:
Basemap: Port of Seattle, 1995

N

0 2000 4000
Scale in Feet

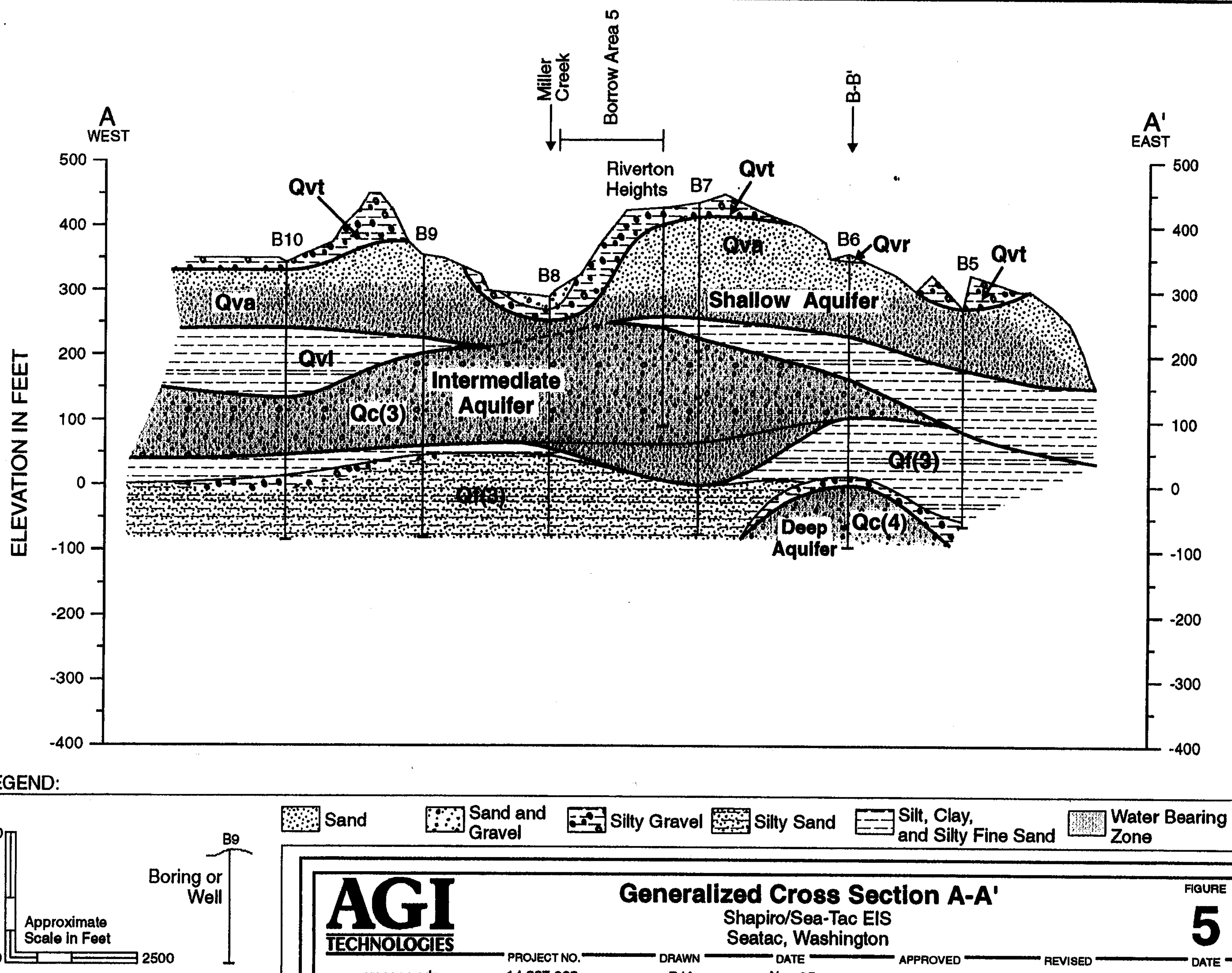
AGI TECHNOLOGIES

Well Locations and Cross Section Location Map

Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE 4

crosssec.dwg	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
	14,887.003	BJA	Nov 95			



AGI TECHNOLOGIES

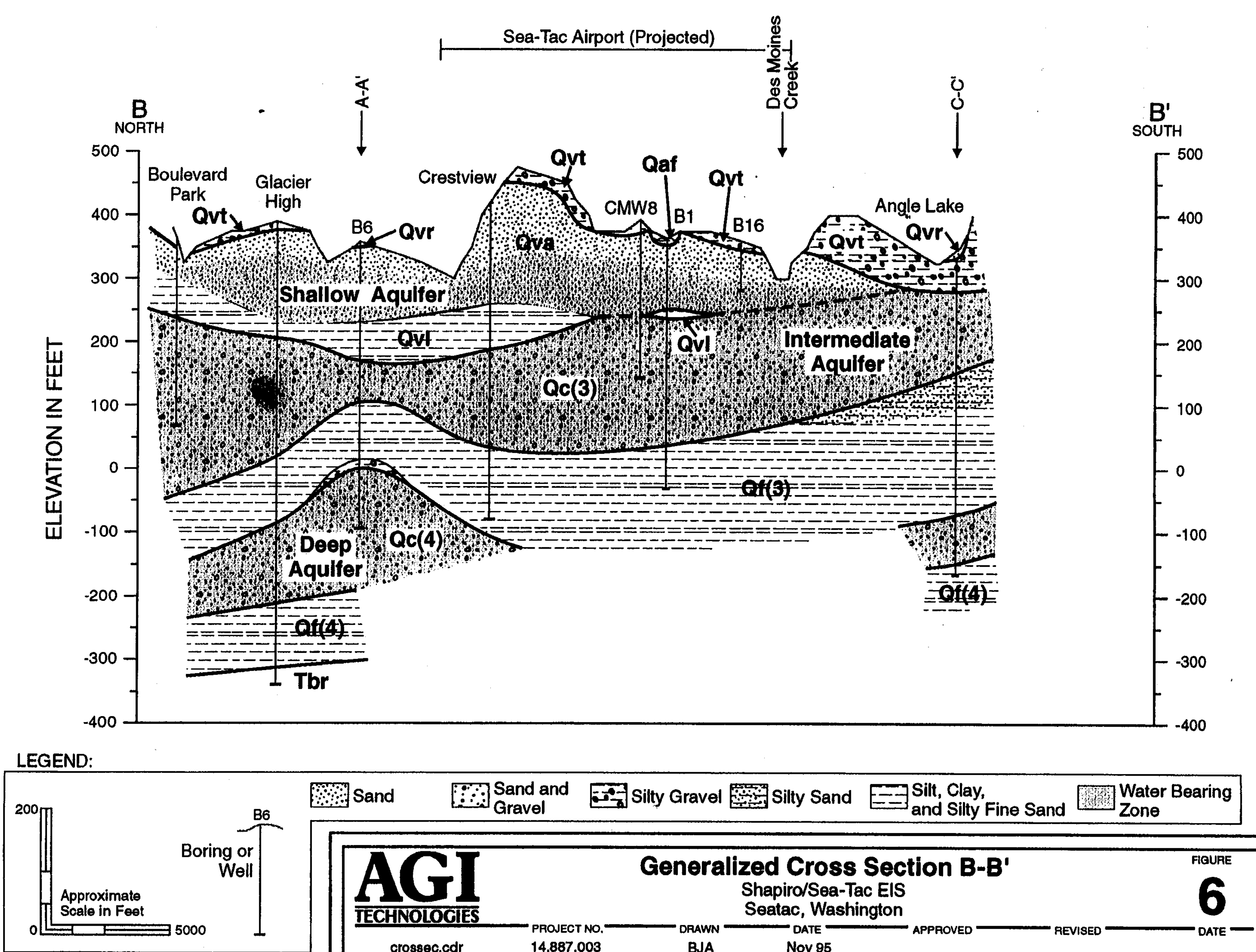
Generalized Cross Section A-A'

Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE **5**

	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
crossec.cdr	14,887.003	BJA	Nov 95			

AR 003402



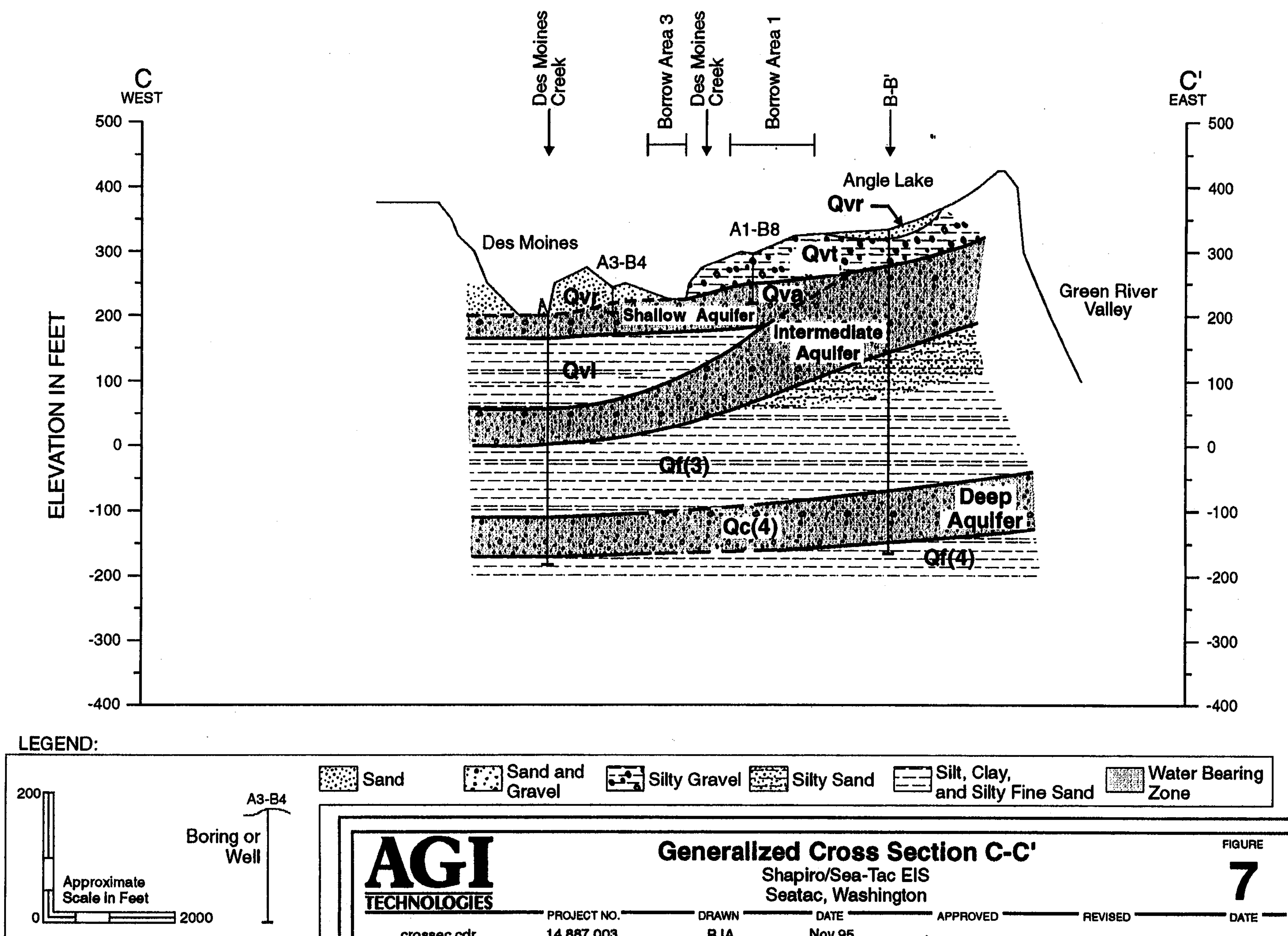
AGI TECHNOLOGIES

Generalized Cross Section B-B'
Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE 6

	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
crossec.cdr	14,887.003	BJA	Nov 95			

AR 003403

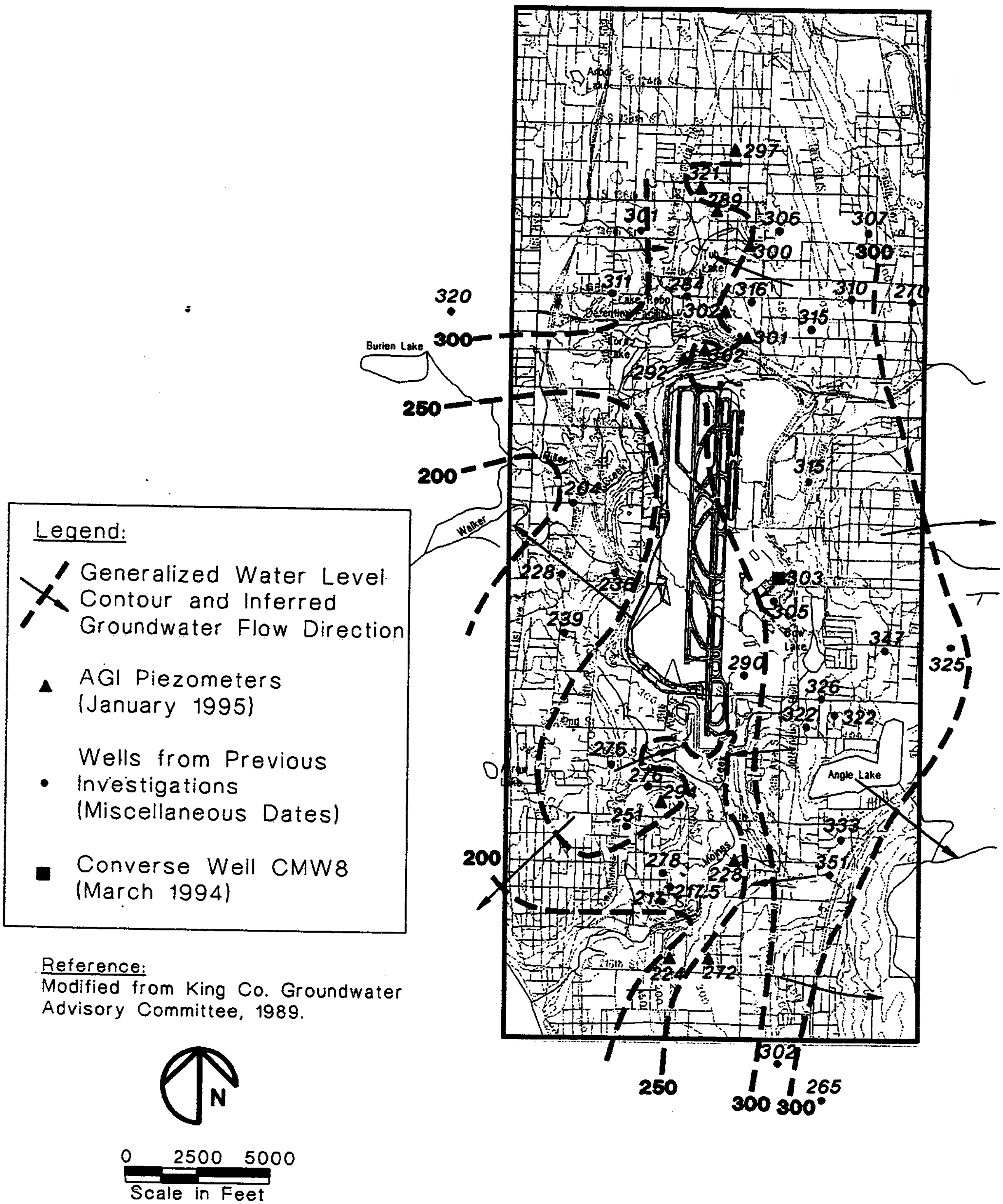


AGI TECHNOLOGIES

Generalized Cross Section C-C'
Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE 7

	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
crossec.cdr	14,887.003	BJA	Nov 95			



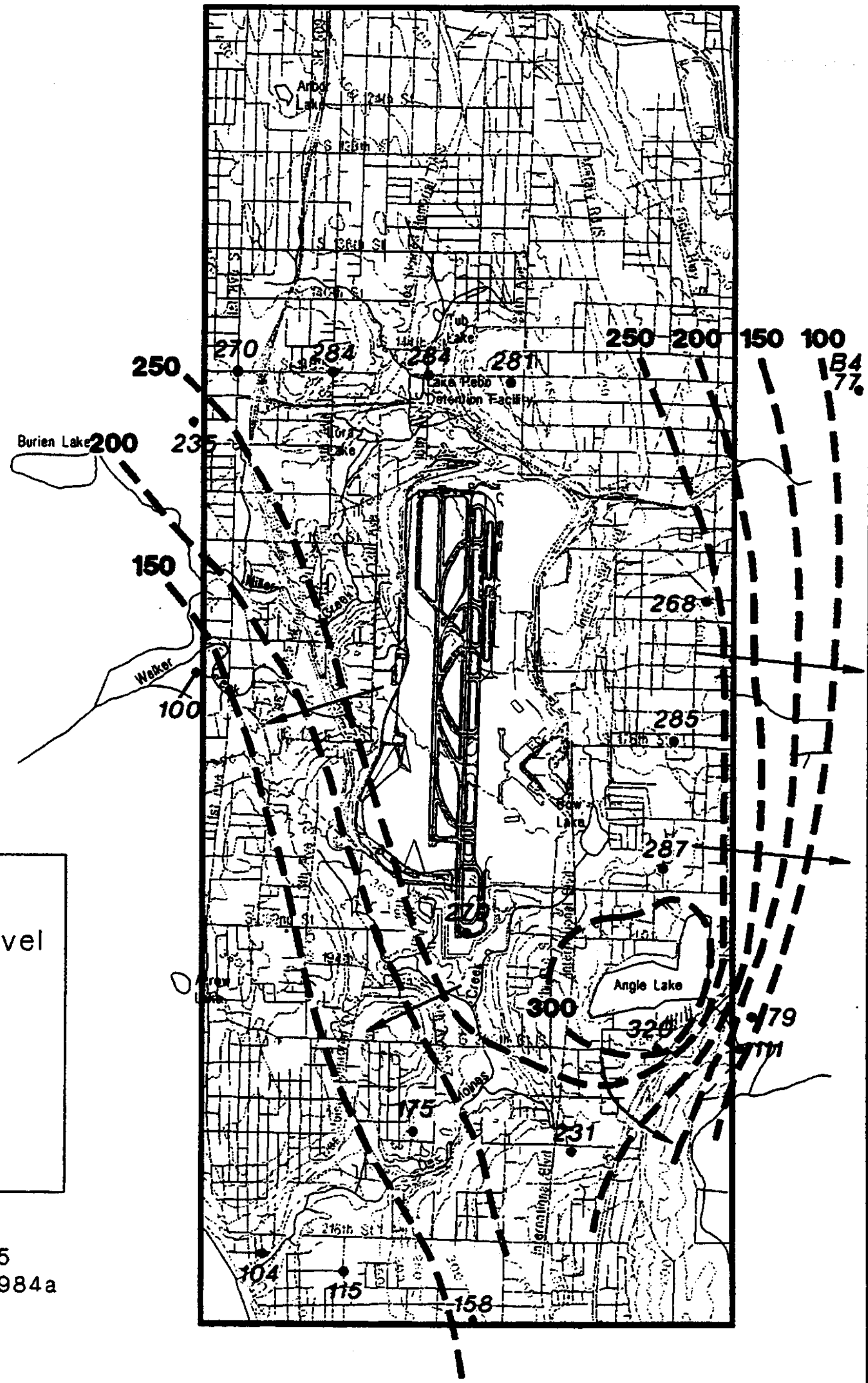
Generalized Groundwater Elevations-Shallow (Qva) Aquifer

Shapiro/Sea-Tac EIS
Seatac, Washington

AGI TECHNOLOGIES

FIGURE **8**

wells2.dwg	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
	14,887.003	BJA	Nov 95			



Legend:

Generalized Water Level Contour and Inferred Flow Direction

• Wells from Previous Investigations (Miscellaneous Dates)

Reference:
Basemap: Port of Seattle, 1995
Modified from Hart Crowser, 1984a

N

0 2500 5000
Scale in Feet

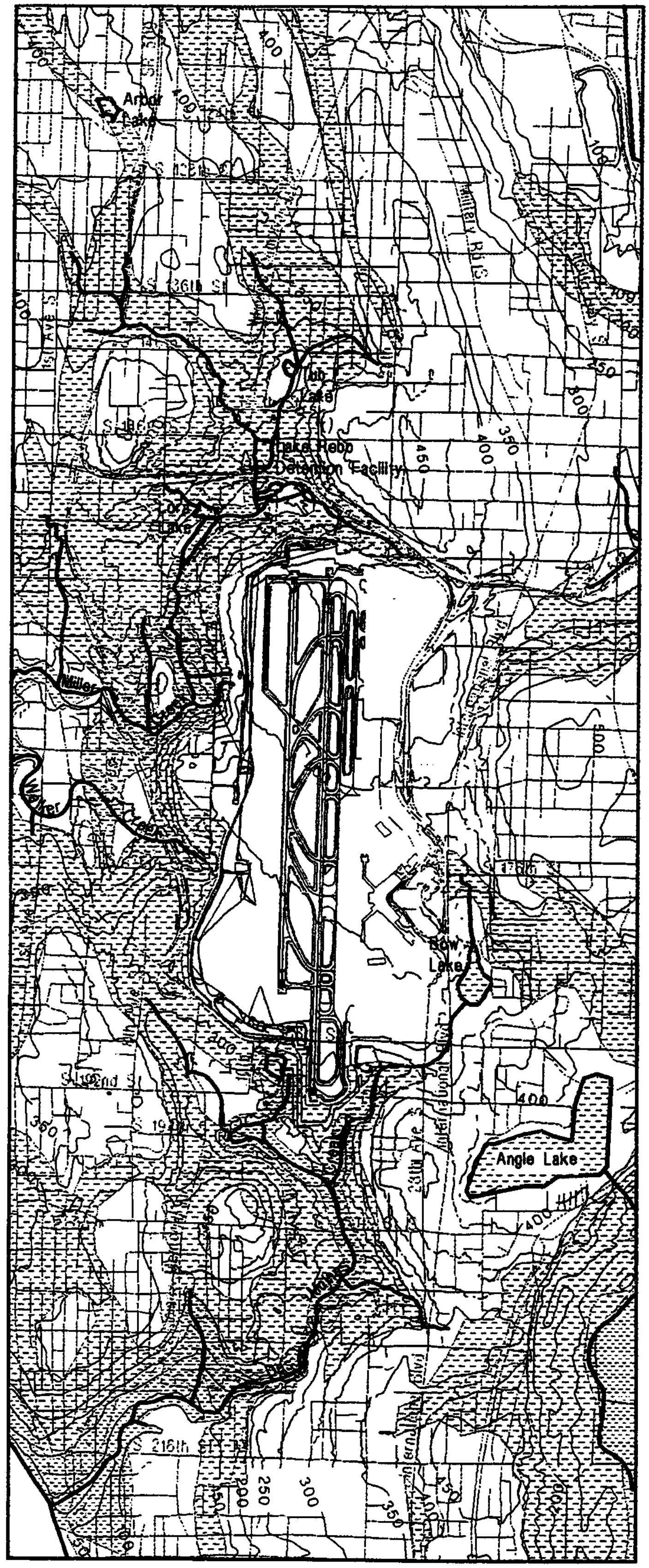
AGI TECHNOLOGIES

Generalized Groundwater Elevations-Intermediate Aquifer

FIGURE 9

Shapiro/Sea-Tac EIS
Seatac, Washington

	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
points.dwg	14,887.003	BJA	Nov 95			



Legend:

Recharge Area

Discharge Area

Reference:
Basemap: Port of Seattle, 1995

N

0 2000 4000

Scale in Feet

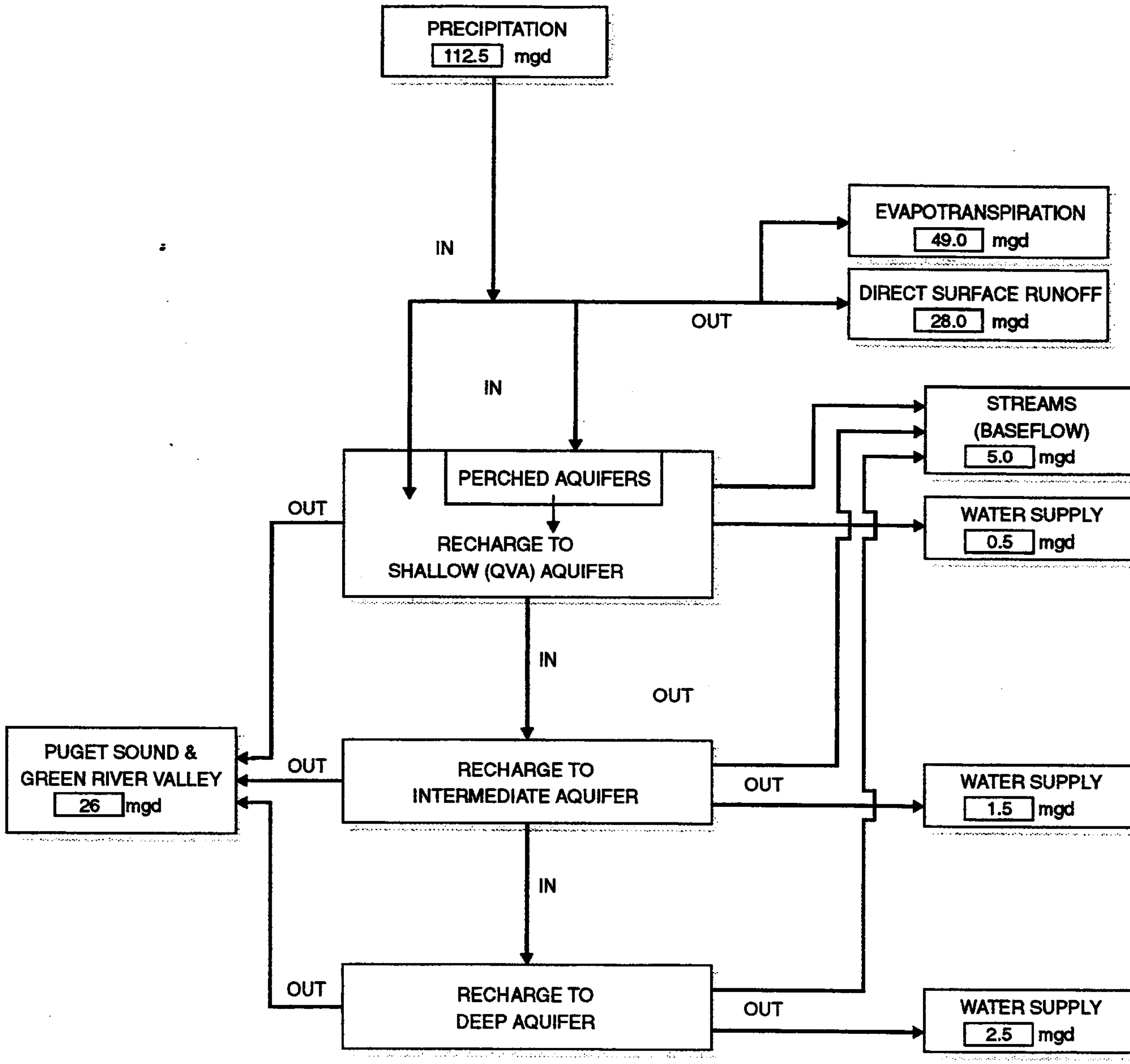
AGI TECHNOLOGIES

Aquifer Recharge/Discharge - Existing Conditions

Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE **10**

	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
redis.dwg	14,887.003	BJA	Nov 95			



AGI TECHNOLOGIES

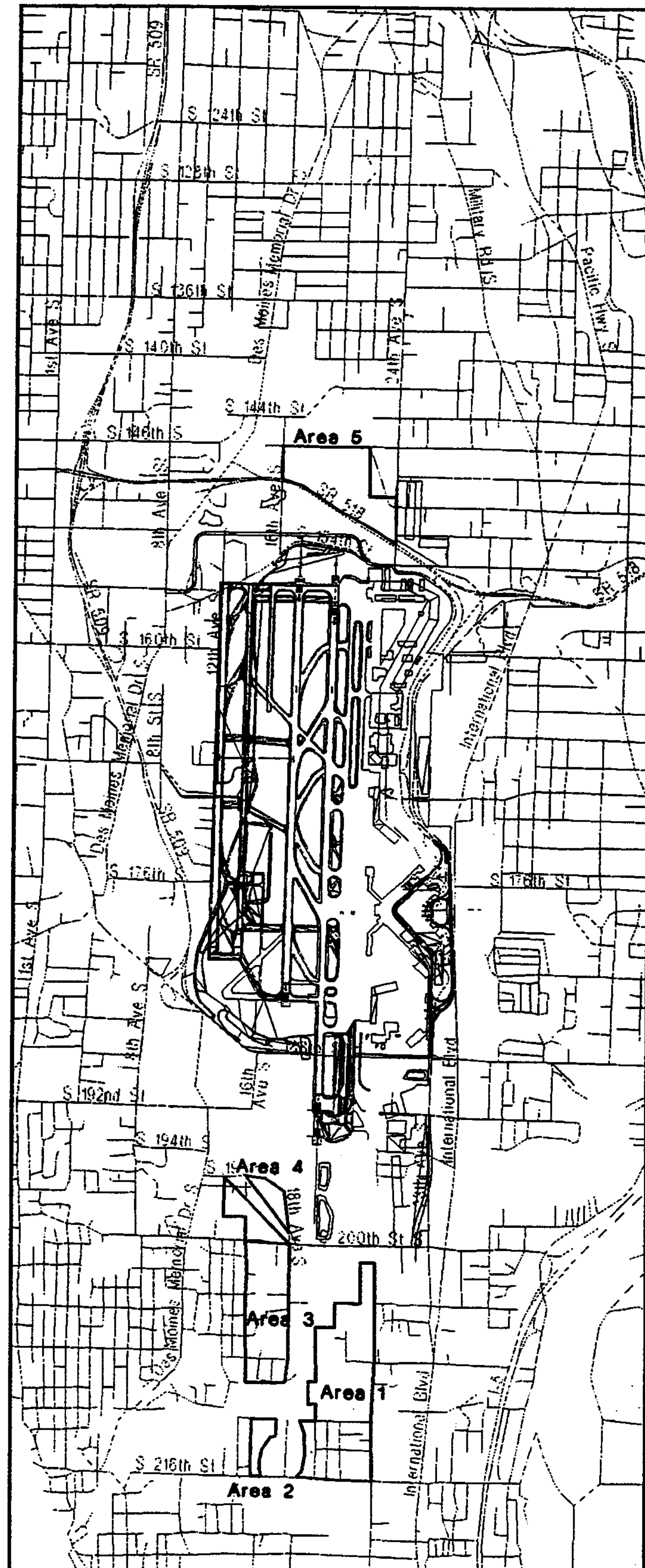
Groundwater Balance - Existing Conditions

Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE **11**

waterbal.cdr	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
	14,887.003	BJA	Nov 95			

AR 003408



Legend:

Proposed Future Facilities (Existing Facilities with Proposed Improvements)

Borrow Area

Reference:
Basemap: Port of Seattle, 1995

N

0 2000 4000
Scale in Feet

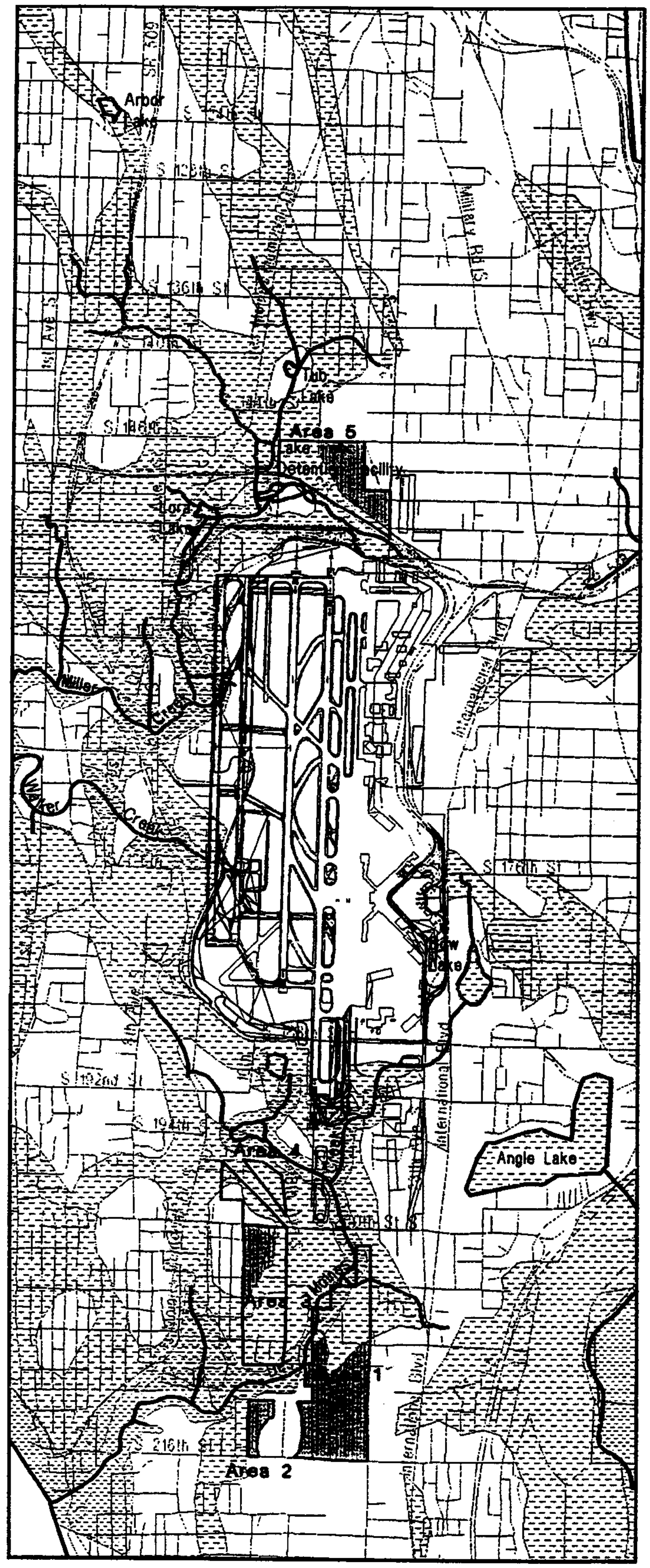
AGI TECHNOLOGIES

Existing Facilities with Improvements

Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE **12**

	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
Improv.dwg	14,887.003	BJA	Nov 95			



Legend:

- Proposed Future Facilities (Existing Facilities with Proposed Improvements)
- Borrow Area
- Filled Recharge Area
- Recharge Area Created by Till Removal
- Recharge Area
- Discharge Area

Reference:
Basemap: Port of Seattle, 1995

N

0 2000 4000
Scale in Feet

AGI TECHNOLOGIES

Aquifer Recharge/Discharge with Improvements

Shapiro/Sea-Tac EIS
Seatac, Washington

FIGURE **13**

	PROJECT NO.	DRAWN	DATE	APPROVED	REVISED	DATE
redisimp.dwg	14,887.003	BJA	Nov 95			

AR 003410

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APPENDIX G

HYDROLOGIC MODELING STUDY
For SeaTac Airport
Master Plan Update EIS

April 7, 1995
Revised November 16, 1995

MONTGOMERY WATER GROUP, INC.
Water Resources•Environmental•Civil Engineering

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(206) 827-3243 (206) 827-3509 FAX

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TABLE OF CONTENTS

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1.0 INTRODUCTION

This report summarizes the hydrological modeling analyses of Miller Creek and Des Moines Creek using the Hydrological Simulation Program - Fortran (HSPF) model, a continuous simulation rainfall runoff and streamflow routing model. The analyses was conducted by Montgomery Water Group, Inc., under subcontract to Shapiro and Associates as part of the Environmental Impact Statement (EIS) for the Seattle-Tacoma International Airport (SeaTac Airport) Master Plan Update.

This report was first prepared for the Draft EIS submission on April 7, 1995. For the Final EIS, several modifications were made to the HSPF modeling analysis. One involved a revision of the pervious and impervious areas within the SeaTac Airport drainage subbasins. Recently revised estimates of these areas, developed during ongoing studies of the SeaTac Airport stormwater drainage system (SDS), were incorporated into the HSPF models. This revision resulted in significant changes to the Des Moines Creek model, but had little effect on the Miller Creek model. A second revision involved a reanalysis of the stormwater detention requirements. A more in-depth analysis of the offsite stormwater flows was performed to estimate the total detention volume needed to completely meet the stormwater detention criteria. Finally, the analysis of the effects of the proposal on low flows was expanded to include a separate verification of the HSPF modeling results.

1.1 PURPOSE

The HSPF modeling analyses were performed to evaluate the effect of land use changes proposed in the Master Use Plan Update on streamflow characteristics in Miller and Des Moines Creek. Those streams are the receiving waters for stormwater runoff from SeaTac Airport. Land use changes proposed in the Master Use Plan Update include an increase in the impervious surface for the proposed SeaTac Airport 3rd runway, new areas of compacted fill which cover areas that are currently occupied by low density housing or open space, and expansion of terminal facilities. Potential effects to streamflows in Miller and Des Moines Creek were evaluated by comparing the hydrologic regime of the creeks under proposed land use conditions to their current hydrologic regime. The comparison was performed using statistical measures of flood frequency, annual flow duration, flow volumes, and monthly flow exceedence.

The HSPF modeling analyses focused on assessing the effects of the proposed project on offsite streams. Detailed modeling of the existing SDS within SeaTac Airport was not conducted because the HSPF model is not suitable for modeling structural elements of complex storm drainage systems. A separate stormwater system modeling analysis using WATERWORKS, a hydraulic analysis computer program capable of modeling complex storm drainage systems, is currently underway by the Port of Seattle. For this study, a representation of the proposed airport expansion was created in the HSPF model using available data on existing drainage patterns, available streamflow data for calibrating an existing conditions model, overall proposed land use changes, and likely offsite stormwater discharge limitations. These data allowed for an accurate assessment of the effect of current and proposed SeaTac Airport stormwater drainage on the two receiving streams.

1.2 METHODOLOGY

Hydrologic modeling of Miller Creek and Des Moines Creek was performed using the HSPF Version 10 (USEPA, 1993) continuous simulation model. The HSPF model is accepted by local agencies and is the preferred method for evaluating effects of stormwater runoff on receiving streams. The hydrologic simulation of both basins used the 47-year record of historical SeaTac hourly precipitation (October 1948 to July 1994).

HSPF models which simulate existing hydrologic conditions were created and calibrated for the Miller Creek and Des Moines Creek Watersheds. Figure 1 shows the location of Miller and Des Moines Creek relative to SeaTac Airport. Current land use characteristics within these watersheds are summarized in Table 1-1. The land use within the Miller Creek Basin is largely residential, whereas the majority of the land use within the Des Moines Creek Basin is airport and other commercial uses.

The Miller Creek HSPF model covers the entire drainage basin. It was adapted from an earlier HSPF model developed by Northwest Hydraulic Consultants (NHC) for the feasibility analysis of the Lake Reba Regional Detention Pond (NHC, 1989). For this study, the NHC model was revised using updated stream and watershed data and then recalibrated using five years of recorded streamflow data (July 1989 to June 1994). The streamflow data was collected by King County from sites at the Lake Reba Detention Pond discharge and at lower Miller Creek near the mouth.

TABLE 1-1
SUMMARY OF CURRENT LAND USE IN MILLER CREEK
AND DES MOINES CREEK WATERSHEDS

Land Use	Des Moines Creek		Miller Creek	
	Area, acres	Percent of Total	Area, acres	Percent of Total
Commercial - Airport	983	27%	193	4%
Commercial	814	23%	727	14%
Multi-family	197	5%	250	5%
Residential	855	24%	2988	57%
Open	735	21%	720	14%
Forest/wetland	0[a]	0%	305	6%
Total	**3585**	**100%**	**5183**	**100%**

[a] Forested and wetland areas in Des Moines Creek are included in the other land use classifications.

The Des Moines Creek HSPF model was assembled from data and information compiled in previous hydraulic modeling studies of the stream. The Des Moines Creek HSPF model covers 2,700 acres, which is about 75 percent of the 3,585 acre Des Moines Creek Watershed. The model extends from the headwaters of the basin to South 208th Street. The Des Moines Creek HSPF model was also calibrated using five years of streamflow data collected from a site at the inflow to Tyee Detention Pond.

The proposed condition models evaluated the potential effects of the changed land use on the hydrologic regimes of Miller and Des Moines Creek. The proposed action will change the amounts and types of impervious and pervious surface area in the subbasins draining from SeaTac Airport. Land use changes incorporated into the proposed condition models included increased impervious surface area from the 3rd runway and changes in pervious surface area from new fill. To mitigate increased stormwater runoff, stormwater detention storage was also included in the proposed conditions models. Detention storage requirements were calculated using detention criteria from the Stormwater Management Manual for Puget Sound (Ecology, 1992). The stormwater detention criteria included detention of the 2-year storm and release at a rate no greater than 50 percent of the existing 2-year runoff rate, and detention of the 10-year and 100-year storms and release at existing rates for those storms.

1.3 DATA SOURCES

The HSPF models of Miller and Des Moines Creek were created or adapted using existing hydraulic models and other relevant data. The data sources are listed below.

1.3.1 Miller Creek

Miller Creek Regional Stormwater Detention Facilities Design Hydrologic Modeling (NHC, 1990). A HSPF model of Miller Creek was prepared for that study to evaluate the Lake Reba Regional Detention Facility. Land use characteristics, subbasin delineation and some structural data from the NHC model was the framework for the revised HSPF model of Miller Creek described in this study.

Miller Creek Regional Stormwater Detention Facilities Draft Feasibility Report (Parametrix, 1990). That report evaluates Lake Reba detention alternatives. Additional details on the selected alternative are included in this study.

Miller Creek, Normandy Park, Washington, Limited Map Maintenance Study (NHC, 1991). That study was prepared for the Federal Emergency Management Agency to delineate the 100-year floodplain along Miller Creek. The study produced a HEC-2 model for Miller Creek, beginning at Lake Reba and extending to Puget Sound. Stream hydraulic modeling results from the HEC-2 model were incorporated into the revised HSPF model of Miller Creek prepared for this study.

Lake Reba Regional Detention Facility Dam Safety Analysis (Parametrix, 1992). Storage-elevation-discharge rating curves for Lake Reba were obtained from that report and used in the revised HSPF model of Miller Creek.

Lake Reba Regional Pond, Miller Creek, Design Drawings (KCSWM, 1992). Design elevations for the Lake Reba outlet works were obtained from the design drawings to verify storage-elevation-discharge rating curves.

Lake Reba Operation and Maintenance Manual (KCSWM, no date). That brief document provided miscellaneous operational data for Lake Reba.

Brief Description Report of Ambaum Regional Water Quality Detention Pond (KCSWM, 1989). That report and the accompanying TR-20 modeling files describe the operational characteristics of the Ambaum Pond. The HSPF model of Miller Creek was prepared for this study and incorporated in the Ambaum Pond facility.

Ambaum Regional Pond, Miller Creek, Design Drawings (KCSWM, 1991). Design elevations for the Ambaum Pond outlet works were obtained from the design drawings to verify storage-elevation-discharge rating curves. The Ambaum Regional Pond was built in 1992.

1.3.2 Des Moines Creek

Des Moines Creek Watershed Management Plan (Herrera, 1989). That report provides documentation of a hydrologic model of Des Moines Creek that was previously created by King County Surface Water Management Division, (SWM). The model, based on the Soil Conservation Service's TR-20 model, was used to evaluate various detention pond alternatives for Des Moines Creek. One of the alternatives, Tyee Pond (termed Pond C in the report), was eventually built by King County. Subbasin model structure and stream hydraulic data from the TR-20 model were incorporated into the new HSPF model of Des Moines Creek.

TR-20 Model Files for Des Moines Creek Pond C (Tyee Pond) (KCSWM, 1989). Computer model files for Des Moines Creek were obtained from King County. The model of Des Moines Creek was revised by King County during the Tyee Pond design to evaluate various outlet works options. A storage-elevation-discharge rating curve for Tyee Pond was obtained from these files.

Des Moines Creek Regional Pond, Tyee Valley Golf Course, As-Built Design Drawings (KCSWM, 1992). Design elevations for the Tyee Pond outlet works were obtained from the design drawings to verify storage-elevation-discharge rating curves.

Tyee Regional Pond Operations and Maintenance Manual (KCSWM, no date). That brief document provided miscellaneous operational data for Tyee Pond.

Final Environmental Impact Statement (Parametrix, 1994). A drainage analysis of the South Aviation Support Area (SASA) was conducted for that EIS. The drainage analysis adapted and used the King County TR-20 model of Des Moines Creek. Assumptions on total impervious area and drainage to the Industrial Wastewater System (IWS) were incorporated into the HSPF model of Des Moines Creek.

Des Moines Creek GIS Study (Gambrell Urban, 1994). Land use data for Des Moines Creek were obtained from these maps which are being prepared for the SeaTac Airport Master Use Update.

Geologic Map of the Des Moines Quadrangle, Washington (Waldron, 1962). Soil mapping units representing till, outwash, and wetland soils were derived from this map.

1:25,000-Scale Metric Topographic-Bathymetric Map of Burien, Washington (USGS, 1983). Delineation of the Des Moines Creek subbasins were largely based on this map.

1.3.3 Sea-Tac Airport

Sea-Tac International IWS and Storm Water Systems, August 1992 (Anne Symonds, 1992). The 1" = 200' maps of the airport prepared by Anne Symonds provide a detailed inventory of the SDS and IWS conveyance systems, drainage subbasin boundaries, and outfall locations. Onsite drainage boundaries and pathways used for this study were based on information contained on these maps.

WATERWORKS Model Data for SeaTac Airport (Anne Symonds, 1994). Preliminary WATERWORKS model files of the SDS system developed by Anne Symonds were used to determine total impervious surface area within each of the eight SeaTac Airport stormwater drainage systems. That data was used to describe SeaTac subbasin land use in the HSPF models prepared for this study. The WATERWORKS model of SeaTac Airport was still under development and was not yet available at the time this study was performed. Updated estimates of impervious and pervious areas were obtained for the Final EIS analysis (Minton, G., personal communication, October 5, 1995).

AR 003422

Preliminary Maps of SeaTac Master Use Plan Alternatives (P&D Aviation, 1994). A delineation of the 3rd runway and other proposed facilities was obtained from AutoCAD maps supplied by P&D Aviation. The plans for the 8,500-foot runway with the central terminal were used to determine new impervious areas in the HSPF models prepared for this study.

1.3.4 Precipitation And Streamflow

SeaTac Precipitation and Evaporation WDM File (KCSWM, 1994). A HSPF Watershed Data Management (WDM) file containing precipitation and evaporation data for the period October, 1948 to July 1993 was obtained from King County Surface Water Management. The precipitation data, which were collected at the SeaTac Airport Weather Station, were recorded at hourly intervals. Daily evaporation rates were derived from historical monthly pan evaporation rates recorded at Puyallup.

Hourly Precipitation Data, Seattle-Tacoma Airport (NCDC, 1994). SeaTac precipitation records for the period of July 1993 to July 1994, used to update the WDM file, were obtained from the National Climatic Data Center.

King County Surface Water Management Stream Gauge Data (KCSWM, 1994). Provisional stream gauging data for various locations in Miller and Des Moines Creek were obtained from King County Surface Water Management. Sites used for calibrating the HSPF models included Gauge 42A, Miller Creek at Southwest 175th Place in Normandy Park; Gauge 42B, Lake Reba outflow (Miller Creek); and Gauge 11A, Tyee Pond discharge (Des Moines Creek). The period of record for the gauges generally runs from 1988 to 1944; a comparison of the available records is provided in Figure 2.

AR 003423

1.4 MODELING ASSUMPTIONS

As with any hydrologic model, assumptions were made regarding use of existing data, how basin features were incorporated into the model, and how proposed conditions were simulated. The following paragraphs describe the primary assumptions used in the modeling process.

- Modeling data obtained from existing models were still valid. For example, land use data for Miller Creek developed in 1989 were still considered valid for our modeling effort.

- A single HSPF model using land use characteristics of the 8500-foot runway proposal was developed to evaluate Alternatives 2-4 relative to current conditions (Alternative 1). The 8500-foot runway alternative has the greatest area of new impervious surface and therefore represents the largest potential increase in storm stormwater runoff of all alternatives.

- The existing stormwater drainage system (SDS) at SeaTac Airport was not incorporated into the HSPF models because of its complexity. Flow from the SDS was modeled by simulating runoff from pervious and impervious surfaces that are quantified in the current WATERWORKS model of the SDS.

- For the same reason, the existing IWS was also not incorporated into the HSPF models. The IWS collects runoff from industrial areas (e.g, fueling, maintenance and de-icing locations) for treatment at the lagoon treatment system. A pipeline drains the effluent to Puget Sound. The IWS has a hydraulic capacity of between the 10- and 25-year storm events and overflows to the SDS during larger storm events. It was assumed that all runoff from the IWS drains to Puget Sound. Therefore, these areas were removed from the HSPF models.

- Land use changes associated with the proposal were simulated by replacing affected areas with impervious surface and pervious fill area, and by adding detention storage to mitigate the increased stormwater runoff. Detention facilities were located at locations where stormwater is likely to discharge offsite and enter the mainstem channels of Miller and Des Moines Creek. The effects of changed land use in the proposed borrow source areas were not considered in the HSPF modeling analysis.

- Stormwater drainage will following existing drainage pathways under proposed conditions, with no transfer of drainage area between Miller and Des Moines Creeks.

AR 003424

2.0 CURRENT CONDITIONS ANALYSES

A discussion of the methodology and results of the HSPF modeling for current conditions is contained in the following sections.

2.1 MILLER CREEK MODEL

The Miller Creek current conditions HSPF model was adapted from a previous HSPF model that was developed for the *Miller Creek Regional Stormwater Facilities Design Hydrologic Modeling* (NHC, 1990). The NHC model was modified and improved for this study using updated stream watershed data from recent studies providing better information on SeaTac Airport subbasin characteristics, and more extensive streamflow data for calibration.

The Miller Creek Watershed subbasins used in the current conditions HSPF model are shown in Figure 3. A more detailed depiction of drainage basin boundaries in the vicinity of SeaTac Airport is shown in Figure 4.

Land cover data used in the Miller Creek HSPF model for current conditions are summarized in Table A-1 (Appendix A). Table A-1 gives the acreage of land use types, soil types and slope combinations within each subbasin. A schematic of the HSPF model of Miller Creek for current conditions, illustrating the arrangement of subbasins and stream reaches in the model, is shown in Figure 5.

Land cover data for the SeaTac Airport SDS and IWS drainage basins are summarized in Table 2-1. These estimates of drainage areas were developed during current modeling studies of the SDS and IWS. Since the IWS discharges directly to Puget Sound, the IWS area were assumed to not contribute any runoff to Miller and Des Moines Creeks.

Most subbasins are represented by a single set of land cover data within the HSPF models. However, the SeaTac Airport subbasins each have two sets of land cover data. These basins are identified by a pair of numbers, such as 20/25 in Figures 3 and 4. Land cover data in the first subbasin number represents the area drained by the SDS system for that subbasin (i.e., the areas in Table 2-1), and land cover data in the second subbasin number represents the remaining area in the

TABLE 2-1
SEATAC AIRPORT SDS AND IWS AREAS

Subbasin	SDS (acres) Pervious	Impervious	Total	IWS (acres)	Drains to:
SDE-4	28	92	120	--	Des Moines Creek
SDN-1	0	14	14	--	Miller Creek
SDN-2	7	27	34	--	Miller Creek
SDN-3	43	16	59	--	Miller Creek
SDN-4[a]	7	3	10	--	Miller Creek
SDS-1	0	40	40	--	Des Moines Creek
SDS-2[b]	13	0	13	--	Des Moines Creek
SDS-3	221	209	430	--	Des Moines Creek
SDS-4	26	18	44	--	Des Moines Creek
SDW-3	14	10	24	--	Des Moines Creek
IWS Air Cargo /Runway	--	--	--	106[c]	Puget Sound
IWS Terminal	--	--	--	148[d]	Puget Sound
TOTAL	359	429	788	254	--

[a] Included in SDN-3 in HSPF model.
[b] Included in Subbasin 11 in HSPF model.
[c] Located in Subbasins SDE-4 (56 acres) in Des Moines Creek model, and SDN-1 (25 acres) and SDN-2 (25 acres) in Miller Creek model.
[d] Located in Subbasin SDS-1 in Des Moines Creek model.

subbasin (e.g., not drained by the SDS system). The model was constructed this way to allow the SDS components to be modeled individually if necessary.

For this study, several changes were made to the HSPF model prepared by NHC. One change was made to correct an error in the stream network. The NHC model showed the SeaTac "I" Pond subbasin (Subbasin No. 20) draining to Walker Creek, located below the stream gauge site on lower Miller Creek. Subbasin 20 actually joins Miller Creek a short distance below 1st Avenue South, above the stream gauge site. Another change to the HSPF model prepared by NHC was a revision of the subbasin boundaries in the vicinity of SeaTac Airport to those shown on the SDS drainage basin maps recently prepared by Anne Symonds (1992). Current drainage basin boundaries are shown in Figure 4. Changing the boundaries required modification of land use data in subbasins adjoining SeaTac Airport. Land use and soils data from NHC (1990) were used for this purpose.

Other changes made to the HSPF model included revisions to stream reach data for Miller Creek below Lake Reba. New routing data was derived using the results of the FEMA HEC-2 hydraulic model of Miller Creek (NHC, 1991). The change resulted in a more accurate representation of streamflow routing in the HSPF model. Elevation-storage-discharge relationships for the Lake Reba and Ambaum Detention Facilities were also incorporated into the HSPF model based on information contained in the King County TR-20 models, as-built drawings, and dam safety reports.

2.2 DES MOINES CREEK MODEL

The Des Moines Creek HSPF model prepared for this study used hydraulic modeling data contained in the TR-20 model that was originally developed by King County. The TR-20 model was used in several studies including the Des Moines Creek Watershed Plan and the Tyee Pond Detention Facility design. The TR-20 model included the east and west branches of Des Moines Creek (draining from Tyee Pond and Northwest Ponds, respectively), and extended downstream to South 208th Street.

A map of the subbasins used in the Des Moines Creek HSPF model is shown in Figure 6. A more detailed description of the drainage basin boundaries in the vicinity of SeaTac Airport is shown in Figure 4.

Land cover data for the SeaTac Airport SDS drainage subbasins are summarized in Table 2-1. Table A-2 in Appendix A summarizes the land cover parameters used in the HSPF model for Des Moines Creek under current conditions. Table A-2 gives the acreage of land use types, soil types and slope combinations within each subbasin. Subbasin land use data were derived from a geographical information system (GIS) analysis of land use and soil maps which was performed by Gambrell Urban. A schematic of the HSPF model of Des Moines Creek for current conditions, illustrating the arrangement of subbasins and stream reaches in the model, is shown in Figure 7.

AR 003427

To create the Des Moines Creek HSPF model, the following adaptations from the TR-20 model were performed:

- The number of subbasins from the TR-20 model was reduced to group similar land use areas together and to simplify the stream network.
- Stream cross-sections from the TR-20 model (XSECTN) were modified and combined to create FTABLES stream reach data in the HSPF model.

2.3 CALIBRATION

Calibration is the process whereby model parameters are adjusted to achieve a close match between recorded streamflows and simulated streamflows over a time period when streamflow data are available. Nearly five years of recorded streamflow, from October 1989 to July 1994, were used in the calibration process.

Hydrologic modeling using HSPF requires refinement of many different parameters that describe different streamflow-producing processes. These processes are based on the concepts of the Stanford Watershed Model. The dominant processes in HSPF include rainfall runoff from pervious and impervious surfaces, infiltration of rainfall to shallow and deep soils, soil moisture accounting, flow of groundwater from shallow soils to streams (i.e., interflow), flow of groundwater from deep soils to streams, and loss of groundwater to deep aquifers. Each of these physical processes are controlled by several parameters. Typically, standard parameters that have been developed for the Puget Sound lowland region (Dinicola, 1990), are used as the initial starting point in the calibration process. This is followed by parameter adjustments to achieve a better match between simulated and recorded streamflows.

As a general guide, the following objectives were established for calibration:

- Achieve a good match of peak flow magnitudes and hydrograph recession characteristics for the larger storms of record, particularly the January 1990, November 1990, and early 1991 events.
- Achieve a good match between recorded and simulated average monthly flows (i.e., runoff volume).

AR 003428

- Achieve a good match between recorded and simulated flow duration curves.

A goal of matching peak flows and volumes to within plus or minus 10 percent (on average) was the target for calibration. However, greater emphasis was placed on accurate calibration of the largest recorded storms to achieve accurate flood frequency estimates.

The final HSPF parameters arrived at in the calibration process for the Miller Creek and Des Moines Creek HSPF models are summarized in Tables 2-2 and 2-3, respectively. Model parameters listed in Tables 2-2 and 2-3 were adjusted on a watershed-wide basis, rather than by individual subbasins during the calibration process. Thus, in the Miller Creek HSPF model the parameters in Table 2-2 were used for the drainage areas above each of the two stream gauges. Total impervious area and active groundwater outflow (in the pervious runoff function) were also adjusted during the calibration process to match peak runoff and baseflow rates, respectively. These parameters were adjusted separately for each drainage area that is tributary to a stream gauge to improve the match of simulated flows to recorded flows.

TABLE 2-2
MILLER CREEK HSPF MODEL PARAMETERS

Land Segment[a]	LZSN (in)	INFILT (in/hr)	LSUR (ft)	SLSUR	KVARY (1/in)	AGWRC (1/day)	INFEXP	INFILD	BASETP	AGWETP	CEPSC (in)	UZSN (in)	NSUR	INTFW	IRC (1/day)	LZETP	RETSC (in/hr)
TFF	4.5	0.10	400	0.05	0.5	0.980	2.5	2.0	0.0	0.0	0.10	0.25	0.35	1.7	0.12	0.70	N/A
TFM	4.5	0.10	400	0.11	0.5	0.980	2.0	2.0	0.0	0.0	0.10	0.25	0.35	1.7	0.12	0.70	N/A
TFS	4.5	0.10	200	0.20	0.5	0.980	1.5	2.0	0.0	0.0	0.10	0.25	0.35	1.7	0.12	0.70	N/A
TGF	4.5	0.10	400	0.05	0.5	0.980	3.5	2.0	0.0	0.0	0.10	0.25	0.25	1.7	0.12	0.25	N/A
TGM	4.5	0.10	400	0.10	0.5	0.980	2.0	2.0	0.0	0.0	0.10	0.25	0.25	1.7	0.12	0.25	N/A
TGS	4.5	0.10	200	0.20	0.5	0.980	1.5	2.0	0.0	0.0	0.10	0.15	0.25	1.7	0.12	0.25	N/A
OF	5.0	1.50	400	0.05	0.3	0.980	2.0	2.0	0.0	0.0	0.10	0.25	0.35	1.7	0.12	0.7	N/A
OG	5.0	0.70	400	0.05	0.3	0.980	2.0	2.0	0.0	0.0	0.10	0.25	0.25	0.0	0.12	0.25	N/A
SA	4.0	1.00	100	0.001	0.5	0.980	10.0	2.0	0.0	0.7	0.10	3.00	0.50	1.0	0.12	0.80	N/A
EIA	N/A	N/A	500	0.05	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.100	N/A	N/A	N/A	0.10

a Land Segments:
TFF - Till, forested, flat
TFM - Till, forested, moderate
TFS - Till, forested, steep
TGF - Till, grass, flat
TGM - Till, grass, moderate
TGS - Till, grass, steep
OF - Outwash forest
OG- Outwash grass
SA - Wetlands
EIA - Effective impervious area

N/A - Not applicable

TABLE 2-3
DES MOINES CREEK HSPF MODEL PARAMETERS

Land Segment[a]	LZSN (in)	INFILT (in/hr)	LSUR (ft)	SLSUR	KVARY (1/in)	AGWRC (1/day)	INFEXP	INFILD	BASETP	AGWETP	CEPSC (in)	UZSN (in)	NSUR	INTFW	IRC (1/day)	LZETP	RETSC (in/hr)
TFF	4.5	0.30	400	0.05	0.5	0.996	3.5	2.0	0.0	0.0	0.20	1.00	0.35	3.0	0.3	0.70	N/A
TFM	4.5	0.30	400	0.10	0.5	0.996	2.0	2.0	0.0	0.0	0.20	0.50	0.35	3.0	0.3	0.70	N/A
TFS	4.5	0.30	200	0.20	0.5	0.996	1.5	2.0	0.0	0.0	0.20	0.30	0.35	3.0	0.3	0.70	N/A
TGF	4.5	0.30	400	0.05	0.5	0.996	3.5	2.0	0.0	0.0	0.20	1.00	0.25	3.0	0.7	0.25	N/A
TGM	4.5	0.30	400	0.10	0.5	0.996	2.0	2.0	0.0	0.0	0.20	0.50	0.25	6.0	0.5	0.25	N/A
TGS	4.5	0.30	200	0.20	0.5	0.996	1.5	2.0	0.0	0.0	0.20	0.30	0.25	7.0	0.3	0.25	N/A
OF	5.0	2.00	400	0.05	0.3	0.996	2.0	2.0	0.0	0.0	0.20	0.50	0.35	2.0	0.8	0.7	N/A
OG	5.0	0.80	400	0.05	0.3	0.996	2.0	2.0	0.0	0.0	0.10	0.50	0.25	2.0	0.8	0.25	N/A
SA	4.0	2.00	100	0.001	0.5	0.996	10.0	2.0	0.0	0.7	0.20	3.00	0.50	1.0	0.8	0.80	N/A
EIA	N/A	N/A	100	0.05	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.15	N/A	N/A	N/A	0.20

a Land Segments:
TFF - Till, forested, flat
TFM - Till, forested, moderate
TFS - Till, forested, steep
TGF - Till, grass, flat
TGM - Till, grass, moderate
TGS - Till, grass, steep
OF - Outwash forest
OG- Outwash grass
SA - Wetlands
EIA - Effective impervious area

N/A - Not applicable

Results from the HSPF model calibration process for the 5-year period of recorded streamflows are shown in Figures 8 and 9. Figure 8 is a plot comparing recorded and simulated monthly peak flows at two locations: Miller Creek near its mouth and Des Moines Creek at South 208th Street. Figure 9 is a plot comparing recorded and simulated monthly average flows, which are representative of flow volume, for the same two locations. Data used to create the plots are contained in Tables B-1 to B-3 (Appendix B).

In the study for the Lake Reba Detention Facility (NHC, 1990), the HSPF model was calibrated using streamflow data from a single gauging station at lower Miller Creek for the period of 1988-1989. For this study, the HSPF model for Miller Creek was calibrated at two locations: Miller Creek below Lake Reba (King County Gauge 42B) and Miller Creek at S.W. 175th Pl. in Normandy Park (King County Gauge 42A), which is located near the mouth of the creek. Model parameters were modified to calibrate the model to five years of recorded streamflow data (1989-1994), including the three largest storms on record which occurred in 1990 and 1991. Because the Lake Reba and Ambaum detention facilities were built in 1992, model calibration was performed for pre- and post-detention conditions. All subsequent simulation runs were based on post-detention conditions.

The Des Moines Creek HSPF model was calibrated to streamflow data collected from the stream gauge located at the east branch of Des Moines Creek above Tyee Pond (King County Gauge 11C). Model parameters were modified to calibrate the model to five years of recorded streamflow data (1989-1994). Stream gauge data for the Tyee Pond outlet were not used because King County noted that stage readings from this gauge were affected by debris. Also, unknown amounts of streamflow bypassed the outlet gauge during the January and November 1990 storms when water flowed over the emergency spillway. Stream gauge data from a gauge located near the mouth of the stream in Des Moines were available but were not used for calibration because the basin model did not extend downstream to that location.

In general, good calibration results were achieved at the lower Miller Creek gauge for both peak flows (Figure 8) and flow volume (Figure 9). The simulated monthly peak flows averaged 89 percent of the corresponding recorded flows (Table B-1). The simulated flow volume achieved better results, with the simulated volume equal to 99 percent of recorded volume. However, data in Table B-1 show that simulation accuracies varied from year to year. The variations may be caused by several factors, including inaccurate peak flow estimates in the stream gauge data, variable precipitation patterns in the watershed (HSPF assumes a uniform distribution using data recorded at the airport), or inaccuracy in subbasin characterization in the HSPF model. Calibration results at the Des Moines Creek inflow to Tyee Pond were similar, with simulated peak flows averaging 101 percent of the recorded flows and a simulated flow volume averaging 114 percent of the recorded volume (Table B-3). The simulated peak flows were also quite variable, probably due to the difficulty of simulating runoff from a basin that has a high percentage of impermeable surface area. The Des Moines Creek basin has approximately 50 percent of its area in commercial and airport uses.

Calibration results for Miller Creek below Lake Reba are summarized in Table B-2 (Appendix B). A good match of peak flows was obtained, but the simulated flow volumes were only 60 percent of the recorded flow volumes. The calibration problem was caused by difficult modeling conditions in the upper Miller Creek Watershed. Streamflow monitoring data for this section of the Miller Creek Basin (recorded at SR 518 above Lake Reba) showed very low peak flow magnitudes for the relatively large basin size. That may be a result of a combination of highly permeable soils, groundwater that drains to deep aquifers rather than to Miller Creek, several natural lakes that may retain large amounts of stormwater runoff, and inaccurate stream gauge data. Runoff volumes were underestimated at Miller Creek below Lake Reba because impervious areas had to be significantly reduced to achieve a good match of peak flows.

In addition to a calibration of the models using the parameters listed in Tables 2-2 and 2-3, test runs were made using the parameters from the NHC model for Miller Creek and the USGS regional parameters for Des Moines Creek. The test runs were made to compare the performance of our calibration parameters to parameters used in previous modeling studies.

A comparison of flow duration characteristics between recorded and simulated flows is shown in Figure B-1 (Appendix B). Two duration curves for simulated flows are shown: one for the calibrated model and one for the test runs using either the USGS regional parameters or the NHC parameters. The duration curves illustrate that the model calibration using our parameters resulted in a closer match to recorded flows.

Hydrograph plots of simulated versus recorded flows for major storms are shown in Figures B-2 to B-4 (Appendix B) for each of the stream gauge locations (Miller Creek near its mouth, Miller Creek below Lake Reba, and Des Moines Creek above Tyee Pond) during the storms of January 1990, November 1990, and April 1991. In addition, a fourth set of hydrograph plots are shown (Figure B-5) comparing simulated Des Moines Creek flows at South 208th Street with recorded flows near the mouth. The stream gauge on lower Des Moines Creek was established in late 1991, and therefore the hydrograph plots are for storms that occurred after this time. Although these two locations are somewhat separated from each other, good matches of storm peak and shape were achieved.

Hydrograph plots of the entire 5-year calibration period are also included in Figures B-6 and B-7 (Appendix B). These plots show daily peak hourly flows for recorded and simulated records at Miller Creek near its mouth and at the Des Moines Creek inflow to Tyee Pond.

2.4 LONG TERM SIMULATION RESULTS

Following calibration of the HSPF models, long-term hydrologic simulations of current conditions for Miller and Des Moines Creek were performed. These simulations were run for the 1948-1994 (47-year) period using hourly SeaTac Airport precipitation and daily Puyallup evaporation data as input. HSPF input data files, called user control input (UCI) files, for the current conditions simulation are contained in Appendix D. The UCI files that were used for the calibration runs are identical to the long-term simulation UCI files contained in Appendix D, except that simulation time and output file specifications were changed.

The results of the current conditions analyses were summarized using flood frequency and flow duration statistical measures. The summaries were completed for selected locations along Miller and Des Moines Creeks. The locations, which spatially represent streamflow conditions along the creeks, are as follows:

- Miller Creek:
  - Below Lake Reba
  - At 1st Avenue South
  - Near mouth

- Des Moines Creek:
  - below Confluence (of east and west branches)
  - at South 208th Street

Flood Frequency

Flood frequency estimates for current conditions are summarized in Table 2-4. Flood frequency estimates were calculated using annual peak flows produced by the HSPF simulation and the Corps of Engineers computer program HEC-FFA, Flood Frequency Analysis (COE, 1992). Exceedence probability graphs of flood frequency data are included as Figures C-1 to C-5 (Appendix C).

TABLE 2-4
FLOOD FREQUENCY ESTIMATES FOR
CURRENT CONDITIONS

Probability	Return Period (years)	Flow (cfs)				
		Miller Creek			Des Moines Creek	
		below Lk. Reba	at 1st Avenue	near Mouth	below Confluence	at S. 208th
0.01	100	171	293	468	232	280
0.02	50	158	259	412	207	247
0.05	20	140	217	343	176	206
0.10	10	125	185	293	154	178
0.20	5	108	154	243	132	150
0.50	2	80	109	173	103	112
0.80	1.25	57	77	124	83	86
0.90	1.11	47	64	104	74	76
0.95	1.05	40	55	91	69	69
0.99	1.01	28	40	69	60	58

AR 003435

The estimated 100-year flood for lower Miller Creek is 468 cfs. This compares to an estimate of 562 cfs by NHC using the previous HSPF model of Miller Creek (NHC, 1990) for the scenario of current land use without the Lake Reba Detention Facility, and 479 cfs for the scenario of future land use with the Lake Reba Detention Facility. NHC did not report an estimate of flood flows for the scenario of current land use with the Lake Reba Detention Facility. That scenario was modeled in this study.

In the proposed conditions analysis described in Section 3, representative 2-year, 10-year, and 100-year storms were selected for use in calculating total detention storage requirements. Those storms were selected from the historical simulation period by choosing storms whose peak flows most closely matched the flood frequency estimates. The selected storms were December 2-5, 1975 for the 2-year event; February 26-29, 1972 for the 10-year event; and January 8-11, 1990 for the 100-year event.

Flow Duration

Flow duration analyses under current conditions were also completed. The results are summarized in a comparative analysis to proposed land use conditions in Section 4.

2.5 MODELING AND CALIBRATION CHALLENGES

Several difficulties in model development and calibration were encountered during the study. The following paragraphs briefly describe difficulties encountered during the modeling process.

- The primary challenge to model calibration was the difficulty simulating the very rapid hydrograph recession in Miller Creek flows. This hydrograph characteristic is probably the result of the large amount of impervious surface in the watershed. During the calibration process, the interflow and recession parameters were extended beyond their "normal" range. This resulted in improved simulation. Future modeling efforts should consider reanalyses of land use characteristics for the entire watershed and more emphasis on adjustment of effective impervious areas in the calibration process.

- A large groundwater loss was needed to simulate recorded flow volumes in both Miller and Des Moines Creek. This was achieved by increasing infiltration and deep-groundwater recharge rates. Recorded peak runoff rates from upper Miller Creek (above SR-518) were particularly low relative to the basin area. The low runoff rates may be due to very pervious soil conditions and lake retention. A large reduction in effective impervious area was made in the HSPF model input to properly simulate the recorded peak flow rates.

- Much difficulty was encountered while trying to establish the current outlet rating curves for the Lake Reba, Ambaum, and Tyee Pond detention facilities, and for Bow Lake and Pond B. Accurate elevation-storage-discharge relationships for these facilities and lakes should be developed for future modeling efforts using as-built drawings and flow monitoring. Relationships that were derived during pre-design studies are apparently still being used by King County for their stream gauging.

- Streamflow data used for calibration were provided by King County with the qualifier "provisional data - do not distribute". The records had numerous gaps due to gauge failure and many high flow readings appeared erroneous, particularly the Lake Reba outflow data. The data should be further reviewed and checked by King County.

3.0 PROPOSED CONDITIONS ANALYSES

The tasks performed in the proposed conditions analyses included modifying the Miller Creek and Des Moines Creek models to reflect additional impervious area and other changed land use in the SeaTac Airport subbasins, calculating detention storage required to meet offsite discharge criteria, running the 47-year simulations, and comparing the resulting streamflow to existing condition streamflow.

A detailed representation of proposed stormwater facilities (i.e., conveyance pipes and stormwater ponds) was not incorporated into the HSPF models for two primary reasons. First, the design process for the runway drainage system had not begun, and no details of a potential system were available for incorporation into this study. Second, such a system would have been too complex to incorporate into an HSPF model. The only significant change made to the modeling network was the addition of detention storage ponds at certain subbasin nodes. Other minor changes were made to the network structure to help simplify the modeling of detention storage facilities. This included joining adjacent subbasins at a single node so that the combined runoff entered a single detention pond.

3.1 LAND USE CHANGES

Figure 10 illustrates the land use changes associated with the proposal. Changes in land use are based on the alternative that includes a 8,500-foot runway located 1,700 feet west of existing Runway 16R-34L. Also included with this alternative is the proposed South Aviation Support Area (SASA) in the southeast corner of the airport, new expansion of parking and cargo areas north of

SR-518, and new areas of fill. Other facilities such as terminal facilities and central parking were not incorporated into the HSPF models because they do not add new impervious area or they currently drain to the IWS. Also, future conversions of SDS areas to connect to the IWS (for water quality improvements) were not incorporated into the HSPF models because study efforts regarding these potential actions are still in progress.

Table 3-1 summarizes the current and proposed land use in the SeaTac Airport subbasins. Land use changes are detailed by subbasin in Table 3-2. New impervious area was assumed to be 100 percent effective, and new fill was assumed to have runoff characteristics equal to flat till. Subbasin areas and parameters for the proposed conditions model are summarized in Table A-3 (Appendix A).

Under the proposal, impervious surface area will increase by 95.4 acres in the Des Moines Creek Watershed and by 97.4 acres in the Miller Creek Watershed, for a total of 192.8 acres. Another 65.7 acres of impervious surface area will be located in the SASA area, but will drain to Puget Sound via the IWS. That area was excluded from the model. In addition, the total area of fill is increased by about 550 acres over both watersheds.

3.2 DETENTION REQUIREMENTS

The Stormwater Management Manual for the Puget Sound Basin (Ecology, 1992) was used as a guide to determine stormwater detention volumes and release rates. The detention criteria used for this analysis included detention of the 2-year storm and release at 50 percent of the existing 2-year runoff rate, and detention of the 10-year and 100-year storms and release at 100 percent of existing runoff rates for those storm events.

Detention storage required to meet Ecology's standards was estimated through the use of design storms hydrographs produced by the HSPF model. As described in Section 2.4, design storms were selected from the historical simulation period to represent 2-year, 10-year, and 100-year recurrence interval storms. The selected storms were December 2-5, 1975 for the 2-year event; February 26-29, 1972 for the 10-year event; and January 8-11, 1990 for the 100-year event. The recorded flows at lower Miller Creek during these design storms are 172 cfs, 277 cfs, and 433 cfs, respectively, compared to the flood frequency estimates of 173 cfs, 293 cfs, and 468 cfs, respectively (for the

TABLE 3-1
SUMMARY OF LAND USE CHANGES IN SEATAC SUBBASINS
ASSUMED FOR PROPOSAL

	Des Moines Creek (acres)	Miller Creek (acres)	Total
CURRENT LAND USE			
- SDS impervious area[a]	369	60	429
- IWS[a]	204	50	254
- Fill and other[a]	410	83	493
- Non-airport[b]	204	326	530
Total	**1,187**	**519**	**1,706**
PROPOSED CHANGES			
- New SDS impervious area	95.4	97.4	192.8
- New IWS	65.7	0	65.7
- New fill	282.5	262.3	544.8
PROPOSED LAND USE			
- SDS impervious area	464.4	157.4	621.8
- IWS	269.7	50	319.7
- Fill and other	452.9	311.6	764.5
Total	**1,187**	**519**	**1,706**

[a] Includes Subbasins 19 and 24 (SDW-3), 20 and 25 (SDS-3), 21 and 26 (SDS-1), and 23 and 28 (SDE-4) in Des Moines Creek, and Subbasins 23 and 27 (SDN-1), 24 and 28 (SDN-2), and 25 and 29 (SDN-3 and SDN-4) in Miller Creek.

[b] Areas in other subbasins affected by airport expansion.

2-year, 10-year, and 100-year return intervals). Since the January 1990 runoff event was less than the estimated 100-year flow, the hourly precipitation amounts in this storm were proportionately increased by a factor of 1.10, which raises the total runoff volume from that event to an amount equal to the average of the January and November 1990 runoff events (the two largest events on record).

TABLE 3-2
DETAIL OF LAND USE CHANGES IN SEATAC SUBBASINS
ASSUMED FOR PROPOSAL

Location	Subbasin Number	Modified Land Use (acres)	
		Compacted Fill	Impervious Surface
MILLER CREEK			
- Drainage to Lake Reba (node 40)	6	32.7	37.9
	7	24.5	0
	25/29	16.3	3.6
- Drainage to middle creek (node 47)	8	67.3	13.7
	26	71.0	24.5
- Drainage to lower creek (node 46)	20	50.5	17.7
Subtotal		**262.3**	**97.4**
DES MOINES CREEK			
- Drainage to Northwest Pond (node 66)	19/24	26.6	1.0
	20/25	144.3	55.0
- Drainage to Tyee Pond (node 50)	4	7.7	0.0
	5	26.8	27.2
	6	27.0	42.7
- Drainage below confluence (node 67)	12	12.6	0.0
	13	13.2	28.1
	21/26	24.3	7.1
- Less drainage to IWS		0.0	(65.7)
Subtotal		**282.5**	**95.4**
TOTAL		**544.8**	**192.8**

The first step in determining detention volumes was to calculate existing runoff rates from the area in each subbasin that were affected by proposed land use changes (i.e., the areas in Table 3-2). Those rates were determined by performing an HSPF model run with the areas removed. Existing rates of runoff were calculated by comparing these flows to the existing conditions model. The second step was to calculate the allowable runoff rate from each subbasin as defined by the stormwater detention criteria. For the 2-year event, this was calculated as the existing subbasin flow minus one-half of the flow calculated in the first step. For the 10-year and 100-year events, the allowable runoff rate is equal to the existing subbasin flow. The last step was to modify the subbasin land use parameters to reflect the proposed land use and to add the required detention storage.

Detention pond sizes were estimated using the King County Surface Water Management Division computer program RDFAC. Ninety-six hour hydrographs were extracted from the HSPF models to perform a routing analysis with RDFAC. A "generic" configuration of a stormwater pond was assumed in all cases. The stormwater ponds were assumed to be 6-foot deep basins with two outlet orifices. The bottom orifice was sized for the allowable release rate during a 2-year event, and the top orifice was sized for the allowable release rates during 10- and 100-year events. This analysis resulted in an initial estimate of stormwater detention volumes. To derive the final stormwater detention volumes, a series of full HSPF simulations were conducted, with each HSPF simulation using an incrementally larger storage volume. After each simulation a flood frequency analysis was conducted at each of the evaluation points along Miller and Des Moines Creek to verify whether the detention criteria were met. The final storage volumes in the Miller Creek model were increased by 50 percent over those initially estimated using the design storms, and the storage volumes in the Des Moines Creek model were increased by 20 percent. The Miller Creek model needed a significantly larger increase in storage volumes. The reason for this may be the presence of a large amount of existing storage along this stream (mostly behind roadway culverts), which alters the flood routing characteristics of this stream, and the timing of runoff from the three SeaTac subbasins that enter Miller Creek at different points in the basin.

The volume of stormwater detention used in the proposed conditions analysis is summarized in Table 3-3. We assumed that three detention ponds would be placed within each watershed. The locations were selected to represent the likely locations of stormwater discharge after the change in land use at the airport.

AR 003441

TABLE 3-3
ASSUMED ONSITE DETENTION VOLUMES

Location	Total Detention Volume[a] (ac-ft)
MILLER CREEK	
- Drainage to Lake Reba	14.9
- Drainage to middle creek	35.3
- Drainage to lower creek	10.4
Subtotal	**60.6**
DES MOINES CREEK	
- Drainage to Tyee Pond	4.6
- Drainage to Northwest Ponds	24.4
- Drainage below confluence	2.4
Subtotal	**31.4**
TOTAL	**92.0**

[a] Includes active storage volume only (i.e., to 100-year storm level).

3.3 RESULTS

The estimated discharge from airport areas under existing and proposed conditions is summarized in Table 3-4. The results are from HSPF runs using the design storm events discussed in Section 2.4. In all cases, the discharge under proposed conditions is lower than the current discharge. In fact, in order to meet the stormwater detention criteria at offsite stream locations, in many cases runoff rates are significantly lower under proposed conditions compared to existing conditions. The discharges listed in Table 3-4 include runoff from areas not affected by proposed land use changes in addition to the areas with land use changes. Thus, a full 50 percent reduction in the 2-year discharge from all airport areas would not be realized.

The performance of the detention ponds in attenuating stormwater runoff is illustrated by hydrographs in Figure 11. The hydrographs are of stormwater discharge from a representative subbasin, Miller Creek Subbasin 24, under three scenarios: existing conditions, proposed conditions without detention, and proposed conditions with detention. Stormwater detention reduces the peak flow rate but increases the rate of flow during the period of hydrograph recession. Stormwater detention has the greatest relative effect during the 2-year storm because detention used in the analyses called for a 50 percent reduction in peak flows during that event.

TABLE 3-4
DESIGN STORM DISCHARGES FROM ONSITE SUBBASINS

Location	Flow Event[a]	Discharge (cfs) Existing	Proposed
MILLER CREEK			
- Lake Reba inflow	2-Year	22.4	16.1
	10-Year	55.9	39.2
	100-Year	93.2	82.4
- Middle creek	2-Year	13.7	6.4
	10-Year	51.4	23.8
	100-Year	94.5	72.8
- Lower creek	2-Year	18.7	12.4
	10-Year	46.8	33.1
	100-Year	78.8	69.6
DES MOINES CREEK			
- NW Pond inflow	2-Year	53.4	44.8
	10-Year	89.5	80.5
	100-Year	145.3	138.5
- Tyee Pond inflow	2-Year	25.7	22.3
	10-Year	44.5	37.6
	100-Year	63.8	60.2
- Below confluence	2-Year	8.0	6.2
	10-Year	17.8	13.4
	100-Year	26.2	24.5

[a] 2-Year event: December 2, 1975
10-Year event: February 27, 1972
100-Year event: January 9, 1990 (with precipitation factored by 1.10)

Simulated peak flow rates in Miller and Des Moines Creeks under current and proposed land use conditions for the three design storm events are summarized in Table 3-5. The simulated peak flows under proposed conditions were found to be less than current peak flows. HSPF input data files for the proposed conditions analyses are contained in Appendix E.

TABLE 3-5
DESIGN STORM DISCHARGES AT OFFSITE STREAMFLOW LOCATIONS

Location	Flow Event[a]	Flow (cfs) Existing	Flow (cfs) Proposed
MILLER CREEK			
- Near mouth (node 17)	2-Year	172	169
	10-Year	298	282
	100-Year	434	414
- at 1st Ave. S. (node 33)	2-Year	106	103
	10-Year	188	169
	100-Year	247	233
- Lake Reba outflow (node 7)	2-Year	78	76
	10-Year	124	123
	100-Year	147	144
DES MOINES CREEK			
- At S. 208th St. (node 18)	2-Year	99	98
	10-Year	171	162
	100-Year	259	248
- Below confluence (node 13)	2-Year	90	88
	10-Year	145	139
	100-Year	222	213

[a] 2-Year event: December 2, 1975
10-Year event: February 27, 1972
100-Year event: January 9, 1990

4.0 ANALYSIS OF IMPACTS

Potential effects of the proposed land use changes on streamflows in Miller and Des Moines Creeks were determined by comparing HSPF model simulation results from the proposed conditions model, which included increased impervious surfaces and stormwater detention, with results from the current conditions model. Statistical measures of flood frequency, flow duration, and flow exceedence were used in the comparative analysis.

4.1 FLOOD FREQUENCY

A 47-year simulation using the historical precipitation record was run with the proposed condition models. A flood frequency analysis of the results of that simulation can determine whether the detention ponds, whose design was based on a representative design storm, can have similar performance over other individual or sequences of storms contained in the historical period of the precipitation record.

The flood frequency analysis was performed using annual peak flows derived from the 47-year simulation period and the HEC-FFA software package. Estimated peak flows for various flood frequencies are summarized in Tables 4-1 and 4-2 for Miller Creek and Des Moines Creek, respectively. The tables also present the difference between estimates of peak flows under proposed and current conditions. The analysis using a full 47-year period of record indicated that peak flows under proposed conditions will not exceed those predicted for existing conditions. As discussed in Section 3.2, the HSPF models were used to verify the amount of stormwater detention volume that would be required under the proposed development scenario. The flood frequency values for proposed conditions that are contained in Tables 4-1 and 4-2 reflect the amount of stormwater detention volume needed to limit offsite flood peaks to no greater than existing conditions.

4.2 ANNUAL FLOW DURATION AND VOLUME

A flow duration analysis quantifies changes in streamflow rates at incremental flow intervals over the entire range of streamflow. Table 4-3 summarizes a comparison of flow duration characteristics between current and proposed conditions. The flow duration analyses were prepared using simulated hourly flows and the USGS SWSTAT computer program which is a surface water statistical analysis program (USGS, 1993).

TABLE 4-1
FLOOD FREQUENCY ESTIMATES FOR
PROPOSED CONDITIONS - MILLER CREEK

Probability	Return Period (years)	Location					
		Below Lake Reba		At 1st Avenue		Near Mouth	
		Flow (cfs)	Difference[a] (cfs)	Flow (cfs)	Difference[a] (cfs)	Flow (cfs)	Difference[a] (cfs)
0.01	100	166	-5	292	-1	454	-14
0.02	50	152	-6	256	-3	400	-12
0.05	20	134	-6	212	-5	334	-9
0.10	10	119	-6	181	-4	285	-8
0.20	5	103	-5	150	-4	238	-5
0.50	2	76	-4	105	-4	170	-3
0.80	1.25	55	-3	75	-2	122	-2
0.90	1.11	46	-1	63	-1	103	-1
0.95	1.05	39	-1	54	-1	89	-2
0.99	1.01	28	0	41	+1	68	-1

[a] Compared to existing conditions (Table 2-4).

TABLE 4-2
FLOOD FREQUENCY ESTIMATES FOR
PROPOSED CONDITIONS - DES MOINES CREEK

Probability	Return Period (years)	Location			
		Below Confluence		At S. 208th Street	
		Flow (cfs)	Difference[a] (cfs)	Flow (cfs)	Difference[a] (cfs)
0.01	100	232	0	280	0
0.02	50	205	-2	244	-3
0.05	20	172	-4	202	-4
0.10	10	149	-5	173	-5
0.20	5	127	-5	145	-5
0.50	2	96	-7	108	-4
0.80	1.25	76	-7	84	-4
0.90	1.11	68	-6	74	-2
0.95	1.05	62	-7	68	-1
0.99	1.01	54	-6	58	0

[a] Compared to existing conditions (Table 2-4).

TABLE 4-3
FLOW DURATION CHARACTERISTICS AT OFFSITE STREAM LOCATIONS

Miller Creek near Mouth (Node 17)

Flow Interval (cfs)		Existing Conditions: Percent Time in Flow Interval	Existing Conditions: Percent Time Flow Exceeds Lower Limit	Existing Conditions: Annual Flow Volume (ac-ft)	Proposed Conditions: Percent Time in Flow Interval	Proposed Conditions: Percent Time Flow Exceeds Lower Limit	Proposed Conditions: Annual Flow Volume (ac-ft)	Difference: Total Volume (ac-ft)	Difference: Percent Volume	Difference: Duration of Flow (hours)
0	2	34.52	100.00	250	34.25	100.00	248	-2	-1%	-24
2	4	34.89	65.48	758	33.66	65.75	731	-27	-4%	-108
4	6	7.57	30.60	274	7.47	32.09	270	-4	-1%	-9
6	8	4.07	23.03	206	4.18	24.62	212	6	3%	10
8	10	2.89	18.96	188	3.00	20.44	195	7	4%	10
10	15	4.79	16.07	433	5.08	17.44	460	26	6%	25
15	20	2.98	11.28	378	3.25	12.36	412	34	9%	24
20	25	2.00	8.30	326	2.21	9.11	360	34	11%	18
25	30	1.43	6.30	285	1.56	6.91	311	26	9%	11
30	35	1.08	4.87	254	1.16	5.34	273	19	7%	7
35	40	0.80	3.79	217	0.87	4.19	236	19	9%	6
40	45	0.62	2.99	191	0.67	3.31	206	15	8%	4
45	50	0.48	2.37	165	0.54	2.64	186	21	13%	5
50	60	0.62	1.89	247	0.68	2.10	271	24	10%	5
60	70	0.38	1.27	179	0.44	1.42	207	28	16%	5
70	80	0.25	0.89	136	0.28	0.98	152	16	12%	3
80	90	0.16	0.64	98	0.18	0.70	111	12	12%	2
90	100	0.11	0.48	76	0.12	0.52	83	7	9%	1
100	120	0.14	0.37	111	0.16	0.40	127	16	14%	2
120	140	0.09	0.23	85	0.09	0.23	85	0	0%	0
140	160	0.04	0.14	43	0.05	0.14	54	11	25%	1
160	180	0.02	0.10	25	0.02	0.10	25	0	0%	0
180	200	0.03	0.07	41	0.03	0.07	41	0	0%	0
200	220	0.02	0.05	30	0.02	0.04	30	0	0%	0
220	240	0.01	0.03	17	0.01	0.03	17	0	0%	0
240	260	0.00	0.02	0	0.01	0.02	18	18	0%	1
260	280	0.01	0.02	20	0.01	0.02	20	0	0%	0
280	300	0.01	0.01	21	0.01	0.01	21	0	0%	0
300	320	0.00	0.00	0	0.00	0.00	0	0	0%	0
			Acre-feet/year:	5,054		Acre-feet/year:	5,361	307		
			Average flow:	6.99 (cfs)		Average flow:	7.42 (cfs)	0.43 (cfs)		

Miller Creek at 1st Avenue (Node 33)

Flow Interval (cfs)		Existing Conditions: Percent Time in Flow Interval	Existing Conditions: Percent Time Flow Exceeds Lower Limit	Existing Conditions: Annual Flow Volume (ac-ft)	Proposed Conditions: Percent Time in Flow Interval	Proposed Conditions: Percent Time Flow Exceeds Lower Limit	Proposed Conditions: Annual Flow Volume (ac-ft)	Difference: Total Volume (ac-ft)	Difference: Percent Volume	Difference: Duration of Flow (hours)
0	2	71.99	100.00	521	69.98	100.00	507	-15	-3%	-176
2	4	8.23	28.01	179	8.31	30.02	180	2	1%	7
4	6	4.40	19.78	159	4.63	21.70	168	8	5%	20
6	8	3.04	15.38	154	3.25	17.07	165	11	7%	18
8	10	2.24	12.34	146	2.46	13.82	160	14	10%	19
10	15	3.68	10.10	333	4.11	11.36	372	39	12%	38
15	20	2.13	6.42	270	2.36	7.25	299	29	11%	20
20	25	1.37	4.29	223	1.55	4.90	252	29	13%	16
25	30	0.89	2.92	177	1.01	3.34	201	24	13%	11
30	35	0.56	2.03	132	0.64	2.33	151	19	14%	7
35	40	0.39	1.47	106	0.44	1.69	119	14	13%	4
40	45	0.27	1.08	83	0.32	1.25	98	15	19%	4
45	50	0.18	0.81	62	0.21	0.92	72	10	17%	3
50	60	0.22	0.63	88	0.25	0.71	100	12	14%	3
60	70	0.12	0.41	56	0.14	0.46	66	9	17%	2
70	80	0.08	0.29	43	0.10	0.32	54	11	25%	2
80	90	0.06	0.21	37	0.07	0.22	43	6	17%	1
90	100	0.04	0.15	28	0.05	0.15	34	7	25%	1
100	120	0.04	0.10	32	0.05	0.10	40	8	25%	1
120	140	0.03	0.06	28	0.02	0.05	19	-9	-33%	-1
140	160	0.01	0.03	11	0.01	0.03	11	0	0%	0
160	180	0.01	0.02	12	0.01	0.02	12	0	0%	0
180	200	0.00	0.01	0	0.00	0.01	0	0	0%	0
200	220	0.00	0.00	0	0.00	0.00	0	0	0%	0
			Acre-feet/year:	2,880		Acre-feet/year:	3,124	244		
			Average flow:	3.99 (cfs)		Average flow:	4.32 (cfs)	0.34 (cfs)		

TABLE 4-3 (CONTINUED)
FLOW DURATION CHARACTERISTICS AT OFFSITE STREAM LOCATIONS

Miller Creek below Lake Reba (Node 7)

Flow Interval (cfs)		Existing Conditions: Percent Time in Flow Interval	Existing Conditions: Percent Time Flow Exceeds Lower Limit	Existing Conditions: Annual Flow Volume (ac-ft)	Proposed Conditions: Percent Time in Flow Interval	Proposed Conditions: Percent Time Flow Exceeds Lower Limit	Proposed Conditions: Annual Flow Volume (ac-ft)	Difference: Total Volume (ac-ft)	Difference: Percent Volume	Difference: Duration of Flow (hours)
0	2	81.32	100.00	589	80.12	100.00	580	-9	-1%	-105
2	4	7.64	18.68	166	7.84	19.88	170	4	3%	18
4	6	3.69	11.05	134	3.88	12.04	140	7	5%	17
6	8	2.10	7.36	106	2.25	8.16	114	8	7%	13
8	10	1.30	5.26	85	1.46	5.91	95	10	12%	14
10	15	1.81	3.96	164	1.97	4.46	178	14	9%	14
15	20	0.86	2.15	109	0.97	2.49	123	14	13%	10
20	25	0.48	1.29	78	0.54	1.52	88	10	13%	5
25	30	0.26	0.81	52	0.32	0.98	64	12	23%	5
30	35	0.16	0.55	38	0.19	0.66	45	7	19%	3
35	40	0.10	0.39	27	0.13	0.47	35	8	30%	3
40	45	0.07	0.29	22	0.08	0.34	25	3	14%	1
45	50	0.05	0.22	17	0.06	0.26	21	3	20%	1
50	60	0.06	0.17	24	0.07	0.20	28	4	17%	1
60	70	0.03	0.12	14	0.04	0.13	19	5	33%	1
70	80	0.02	0.09	11	0.02	0.09	11	0	0%	0
80	90	0.01	0.07	6	0.01	0.07	6	0	0%	0
90	100	0.02	0.06	14	0.02	0.05	14	0	0%	0
100	120	0.02	0.04	16	0.02	0.03	16	0	0%	0
120	140	0.01	0.01	9	0.01	0.01	9	0	0%	0
140	160	0.00	0.00	0	0.00	0.00	0	0	0%	0
			Acre-feet/year:	1,680		Acre-feet/year:	1,781	101		
			Average flow:	2.32 (cfs)		Average flow:	2.46 (cfs)	0.14 (cfs)		

Des Moines Creek at S 208th Street (Node 18)

Flow Interval (cfs)		Existing Conditions: Percent Time in Flow Interval	Existing Conditions: Percent Time Flow Exceeds Lower Limit	Existing Conditions: Annual Flow Volume (ac-ft)	Proposed Conditions: Percent Time in Flow Interval	Proposed Conditions: Percent Time Flow Exceeds Lower Limit	Proposed Conditions: Annual Flow Volume (ac-ft)	Difference: Total Volume (ac-ft)	Difference: Percent Volume	Difference: Duration of Flow (hours)
0	2	32.07	100.00	232	35.62	100.00	258	26	11%	311
2	4	31.26	67.93	679	29.29	64.38	636	-43	-6%	-173
4	6	13.12	36.67	475	11.58	35.09	419	-56	-12%	-135
6	8	6.55	23.55	332	6.11	23.51	310	-22	-7%	-39
8	10	3.99	17.01	260	3.78	17.40	246	-14	-5%	-18
10	15	5.27	13.01	477	5.32	13.62	481	5	1%	4
15	20	2.51	7.75	318	2.65	8.31	336	18	6%	12
20	25	1.45	5.23	236	1.51	5.65	246	10	4%	5
25	30	0.93	3.78	185	1.01	4.14	201	16	9%	7
30	35	0.73	2.85	172	0.78	3.13	184	12	7%	4
35	40	0.52	2.12	141	0.57	2.35	155	14	10%	4
40	45	0.43	1.61	132	0.47	1.78	145	12	9%	4
45	50	0.31	1.17	107	0.35	1.31	120	14	13%	4
50	60	0.38	0.87	151	0.42	0.96	167	16	11%	4
60	70	0.22	0.49	104	0.25	0.54	118	14	14%	3
70	80	0.11	0.27	60	0.13	0.29	71	11	18%	2
80	90	0.06	0.16	37	0.06	0.17	37	0	0%	0
90	100	0.03	0.10	21	0.03	0.11	21	0	0%	0
100	120	0.03	0.07	24	0.04	0.08	32	8	33%	1
120	140	0.02	0.04	19	0.02	0.05	19	0	0%	0
140	160	0.01	0.02	11	0.01	0.03	11	0	0%	0
160	180	0.01	0.01	12	0.01	0.01	12	0	0%	0
180	200	0.00	0.00	0	0.00	0.01	0	0	0%	0
200	220	0.00	0.00	0	0.00	0.00	0	0	0%	0
			Total acre-feet:	4,184		Total acre-feet:	4,223	39		
			Average flow:	5.79 (cfs)		Average flow:	5.84 (cfs)	0.05 (cfs)		

TABLE 4-3 (CONTINUED)
FLOW DURATION CHARACTERISTICS AT OFFSITE STREAM LOCATIONS

Des Moines Creek below Confluence (Node 13)											
		Existing Conditions			Proposed Conditions			Difference			
Flow Interval (cfs)		Percent Time in Flow Interval	Percent Time Flow Exceeds Lower Limit	Annual Flow Volume (ac-ft)	Percent Time in Flow Interval	Percent Time Flow Exceeds Lower Limit	Annual Flow Volume (ac-ft)	Total Volume (ac-ft)	Percent Volume	Duration of Flow (hours)
0	2	41.32	100.00	299	44.59	100.00	323	24	8%	286
2	4	28.31	58.68	615	25.89	55.41	562	-53	-9%	-212
4	6	10.19	30.37	369	9.06	29.52	328	-41	-11%	-99
6	8	5.68	20.18	288	5.32	20.45	270	-18	-6%	-32
8	10	3.50	14.50	228	3.40	15.13	222	-7	-3%	-9
10	15	4.72	11.00	427	4.84	11.73	438	11	3%	11
15	20	2.18	6.28	276	2.33	6.89	295	19	7%	13
20	25	1.31	4.10	213	1.40	4.56	228	15	7%	8
25	30	0.82	2.79	163	0.93	3.16	185	22	13%	10
30	35	0.59	1.96	139	0.64	2.22	151	12	8%	4
35	40	0.40	1.38	109	0.47	1.58	128	19	17%	6
40	45	0.27	0.97	83	0.32	1.11	98	15	19%	4
45	50	0.19	0.71	65	0.20	0.79	69	3	5%	1
50	60	0.23	0.51	92	0.27	0.59	108	16	17%	4
60	70	0.11	0.29	52	0.12	0.32	56	5	9%	1
70	80	0.06	0.18	33	0.08	0.19	43	11	33%	2
80	90	0.04	0.11	25	0.04	0.12	25	0	0%	0
90	100	0.02	0.07	14	0.02	0.08	14	0	0%	0
100	120	0.02	0.05	16	0.03	0.05	24	8	50%	1
120	140	0.01	0.02	9	0.01	0.03	9	0	0%	0
140	160	0.01	0.01	11	0.01	0.01	11	0	0%	0
160	180	0.00	0.00	0	0.00	0.00	0	0	0%	0
180	200	0.00	0.00	0	0.00	0.00	0	0	0%	0
200	220	0.00	0.00	0	0.00	0.00	0	0	0%	0
			Total acre-feet:	3,525		Total acre-feet:	3,586	61		
			Average flow:	4.88 (cfs)		Average flow:	4.96 (cfs)	0.08 (cfs)		

The flow duration analyses predict that flow volumes in lower Miller Creek will increase by approximately 300 acre-feet per year on average, or an increase of approximately 6 percent. Flow volumes in Des Moines Creek are predicted to increase by approximately 60 acre-feet per year, or an increase of approximately 1 percent. The increase in flow volumes is caused by the additional impervious area, which reduces infiltration and evapotranspiration. The smaller increase found for Des Moines Creek was a result of the diversion of surface water runoff in the South Aviation Support Area to Puget Sound via the IWS.

The increase in flow volumes occur in the lower flow ranges. In Miller Creek near the mouth, the increase occurs mostly below the 120 cfs flow level, which is equal to about the 1.25-year storm frequency. (The 1.25-year flood occurs on average about every 4 out of 5 years). In Des Moines Creek at South 208th Street, the increase occurs below the 80 cfs flow level, which is also equal to about the 1.25 year flow magnitude. This is the range of flow that occurs when the detention ponds discharge their storage to the streams after a storm event. For storms larger than the 2-year event, the increased volume of flow is not significant.

Under low flow conditions, the HSPF model predicts that streamflows will decrease. The reduction is due to less interflow and groundwater recharge that could occur when impervious surface area is increased. In Miller Creek, streamflows below the 6 cfs magnitude, which occur about 77 percent of the time over the year, would be reduced by about 3 percent as a result of the proposal. The greatest effect to streamflows would be in the 2-4 cfs range, where the flow would be reduced by up to 4 percent.

The effects of the proposal on low flows are more pronounced in Des Moines Creek. Streamflows below the 10 cfs magnitude, which currently occur about 87 percent of the time over the year in Des Moines Creek, would be reduced by about 6 percent as a result of the airport project. The greatest decrease would be in the 4-6 cfs range, where streamflows would be reduced by up to 11 percent. The effects are greater in Des Moines Creek than in Miller Creek because diversion of runoff to the IWS further decreases the amount of water reaching the stream.

4.3 SEASONAL FLOW EXCEEDENCE CHARACTERISTICS

The potential change in seasonal runoff characteristics was assessed using a flow exceedence analysis for different periods of the year. For the analysis, flow exceedence levels of 90 percent, 50 percent and 10 percent were selected to represent low, median, and high streamflow conditions, respectively. To assess seasonal differences, the calendar year was divided into 48 periods, or 4 per month. This analysis allows one, for example, to determine how the proposal will affect August streamflow rates during a low flow year. A computer program developed by the USGS (Program B17) and modified by Bruce Barker of the Department of Ecology Water Resources Division was used for this analysis.

The seasonal flow exceedence analysis is summarized in Table 4-4. The analysis was conducted at each of the five evaluation points on Miller and Des Moines Creek. The results show that, due to the proposed land use change, summer streamflows during low flow years (i.e., exceeded 90 percent of the time) may decrease by up to about 0.1 cfs in Miller Creek, and by up to 0.2 cfs in Des Moines Creek. Summer streamflow is predicted to increase during median and high flow years in Miller Creek, but decrease in Des Moines Creek. The latter is due to diversion of SASA runoff to the IWS.

TABLE 4-4

FLOW EXCEEDENCE CHARACTERISTICS AT OFFSITE STREAM LOCATIONS

Miller Creek near Mouth (Node 17)										
		Average Daily Flow Rate (cfs)								
		Current Conditions			Proposed Conditions			Difference		
Period	Month	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow
1	Jan	3.1	9.4	29.7	3.1	9.8	31.8	0.0	0.4	2.1
2		3.7	10.5	29.6	3.8	11.0	31.4	0.1	0.5	1.8
3		3.2	9.9	33.9	3.2	10.5	36.3	0.0	0.6	2.4
4		3.5	11.3	29.6	3.6	11.8	31.6	0.1	0.5	2.0
5	Feb	2.8	8.1	24.4	2.8	8.4	26.0	0.0	0.3	1.6
6		3.4	8.7	25.0	3.4	9.1	26.7	0.0	0.4	1.7
7		3.4	10.4	27.5	3.4	10.9	29.4	0.0	0.5	1.9
8		3.2	8.1	23.0	3.2	8.4	24.5	0.0	0.3	1.5
9	Mar	3.4	7.4	21.8	3.4	7.7	23.1	0.0	0.3	1.3
10		2.8	7.1	19.6	2.8	7.4	20.9	0.0	0.3	1.3
11		2.7	5.6	14.8	2.7	5.8	15.7	0.0	0.2	0.9
12		2.9	6.2	14.0	2.9	6.4	14.9	0.0	0.2	0.9
13	Apr	2.4	4.5	13.7	2.4	4.6	14.5	0.0	0.1	0.8
14		2.3	4.6	11.2	2.3	4.8	11.9	0.0	0.2	0.7
15		2.0	3.9	11.0	2.0	4.0	11.7	0.0	0.1	0.7
16		2.3	3.6	6.8	2.3	3.7	7.1	0.0	0.1	0.3
17	May	2.1	3.1	6.0	2.1	3.2	6.3	0.0	0.1	0.3
18		1.9	2.8	4.6	1.9	2.8	4.8	0.0	0.0	0.2
19		1.8	2.6	4.5	1.8	2.7	4.7	0.0	0.1	0.2
20		1.8	2.8	5.6	1.8	2.9	5.9	0.0	0.1	0.3
21	Jun	1.7	2.5	6.5	1.6	2.6	6.9	-0.1	0.1	0.4
22		1.6	2.5	5.1	1.6	2.6	5.4	0.0	0.1	0.3
23		1.7	2.2	4.4	1.7	2.2	4.6	0.0	0.0	0.2
24		1.6	2.4	4.2	1.6	2.4	4.4	0.0	0.0	0.2
25	Jul	1.6	2.2	3.7	1.6	2.2	3.8	0.0	0.0	0.1
26		1.5	2.1	3.8	1.5	2.1	3.9	0.0	0.0	0.1
27		1.6	1.8	2.8	1.6	1.9	2.9	0.0	0.1	0.1
28		1.6	1.8	2.6	1.6	1.8	2.6	0.0	0.0	0.0
29	Aug	1.5	1.7	3.0	1.5	1.7	3.1	0.0	0.0	0.1
30		1.5	1.7	3.0	1.5	1.7	3.2	0.0	0.0	0.2
31		1.4	1.8	3.9	1.3	1.9	4.1	-0.1	0.1	0.2
32		1.5	2.0	4.7	1.4	2.0	5.0	-0.1	0.0	0.3
33	Sep	1.3	2.0	4.1	1.3	2.0	4.3	0.0	0.0	0.2
34		1.4	2.1	4.0	1.4	2.2	4.2	0.0	0.1	0.2
35		1.4	2.5	6.1	1.3	2.6	6.6	-0.1	0.1	0.5
36		1.3	2.6	7.2	1.3	2.7	7.8	0.0	0.1	0.6
37	Oct	1.4	2.4	7.4	1.4	2.5	8.0	0.0	0.1	0.6
38		1.5	3.2	9.0	1.5	3.4	9.8	0.0	0.2	0.8
39		1.6	3.5	9.8	1.6	3.7	10.6	0.0	0.2	0.8
40		2.3	5.0	11.6	2.4	5.3	12.7	0.1	0.3	1.1
41	Nov	2.2	5.8	15.5	2.2	6.2	16.9	0.0	0.4	1.4
42		2.6	7.8	22.2	2.7	8.4	24.1	0.1	0.6	1.9
43		2.6	8.5	26.2	2.7	9.0	28.3	0.1	0.5	2.1
44		3.7	10.8	27.9	3.8	11.5	29.9	0.1	0.7	2.0
45	Dec	4.2	11.9	27.6	4.3	12.7	29.8	0.1	0.8	2.2
46		3.8	10.8	28.2	3.9	11.5	30.3	0.1	0.7	2.1
47		3.6	10.3	29.1	3.7	10.9	31.1	0.1	0.6	2.0
48		3.9	10.4	25.3	4.0	11.0	27.2	0.1	0.6	1.9

Notes:
1) Low, median and high flow are defined as flows exceeding 90%, 50% and 10% of the time, respectively.
2) Each month is divided into 4 equal periods, for a total of 48 periods in the year.
3) Flows are based on HSPF model results for a 1948-1994 simulation period.

TABLE 4-4 (CONTINUED)

FLOW EXCEEDENCE CHARACTERISTICS AT OFFSITE STREAM LOCATIONS

Miller Creek above 1st Avenue (Node 33)										
		Average Daily Flow Rate (cfs)								
		Current Conditions			Proposed Conditions			Difference		
Period	Month	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow
1	Jan	1.1	5.0	19.1	1.1	5.4	21.0	0.0	0.4	1.9
2		1.5	5.7	18.8	1.6	6.2	20.4	0.1	0.5	1.6
3		1.2	5.3	21.8	1.3	5.7	24.0	0.1	0.4	2.2
4		1.3	6.2	19.1	1.4	6.7	21.1	0.1	0.5	2.0
5	Feb	0.9	4.2	15.6	1.0	4.4	17.1	0.1	0.2	1.5
6		1.3	4.6	15.7	1.4	5.0	17.2	0.1	0.4	1.5
7		1.2	5.7	17.7	1.3	6.1	19.4	0.1	0.4	1.7
8		1.2	4.2	14.4	1.2	4.5	15.8	0.0	0.3	1.4
9	Mar	1.3	3.8	13.3	1.4	4.1	14.6	0.1	0.3	1.3
10		1.0	3.5	12.2	1.0	3.7	13.4	0.0	0.2	1.2
11		0.9	2.7	8.9	0.9	2.9	9.7	0.0	0.2	0.8
12		1.0	3.0	8.4	1.0	3.2	9.2	0.0	0.2	0.8
13	Apr	0.7	2.0	8.0	0.7	2.1	8.7	0.0	0.1	0.7
14		0.6	2.0	6.5	0.6	2.1	7.2	0.0	0.1	0.7
15		0.5	1.5	6.2	0.5	1.6	6.8	0.0	0.1	0.6
16		0.6	1.4	3.5	0.6	1.5	3.8	0.0	0.1	0.3
17	May	0.5	1.1	2.9	0.5	1.1	3.2	0.0	0.0	0.3
18		0.4	0.8	2.1	0.4	0.9	2.3	0.0	0.1	0.2
19		0.3	0.7	2.0	0.3	0.8	2.2	0.0	0.1	0.2
20		0.3	0.8	2.7	0.3	0.9	3.0	0.0	0.1	0.3
21	Jun	0.2	0.7	3.1	0.2	0.7	3.4	0.0	0.0	0.3
22		0.2	0.6	2.3	0.2	0.6	2.6	0.0	0.0	0.3
23		0.2	0.5	1.8	0.2	0.5	1.9	0.0	0.0	0.1
24		0.2	0.5	1.8	0.2	0.5	2.0	0.0	0.0	0.2
25	Jul	0.2	0.4	1.4	0.2	0.4	1.5	0.0	0.0	0.1
26		0.1	0.4	1.4	0.1	0.4	1.5	0.0	0.0	0.1
27		0.1	0.3	0.8	0.1	0.3	0.9	0.0	0.0	0.1
28		0.1	0.2	0.6	0.1	0.2	0.7	0.0	0.0	0.1
29	Aug	0.1	0.2	0.8	0.1	0.2	0.9	0.0	0.0	0.1
30		0.1	0.2	0.9	0.1	0.2	0.9	0.0	0.0	0.0
31		0.1	0.2	1.3	0.1	0.2	1.4	0.0	0.0	0.1
32		0.1	0.3	1.9	0.1	0.3	2.2	0.0	0.0	0.3
33	Sep	0.1	0.2	1.5	0.1	0.3	1.7	0.0	0.1	0.2
34		0.1	0.3	1.7	0.1	0.3	1.9	0.0	0.0	0.2
35		0.1	0.4	3.1	0.1	0.5	3.7	0.0	0.1	0.6
36		0.1	0.5	4.0	0.1	0.5	4.6	0.0	0.0	0.6
37	Oct	0.1	0.5	4.1	0.1	0.6	4.6	0.0	0.1	0.5
38		0.1	0.9	5.6	0.1	1.0	6.4	0.0	0.1	0.8
39		0.2	1.1	6.0	0.2	1.2	6.8	0.0	0.1	0.8
40		0.5	2.2	6.8	0.6	2.4	7.7	0.1	0.2	0.9
41	Nov	0.4	2.7	10.0	0.5	3.0	11.2	0.1	0.3	1.2
42		0.6	4.2	14.0	0.6	4.7	15.7	0.0	0.5	1.7
43		0.7	4.5	17.0	0.7	5.0	18.9	0.0	0.5	1.9
44		1.2	6.4	17.3	1.3	7.1	18.9	0.1	0.7	1.6
45	Dec	1.6	6.9	17.3	1.7	7.6	19.1	0.1	0.7	1.8
46		1.4	6.1	17.8	1.6	6.6	19.6	0.2	0.5	1.8
47		1.4	5.7	18.5	1.4	6.2	20.4	0.0	0.5	1.9
48		1.6	5.7	15.9	1.7	6.2	17.6	0.1	0.5	1.7

Notes:
1) Low, median and high flow are defined as flows exceeding 90%, 50% and 10% of the time, respectively.
2) Each month is divided into 4 equal periods, for a total of 48 periods in the year.
3) Flows are based on HSPF model results for a 1948-1994 simulation period.

TABLE 4-4 (CONTINUED)

FLOW EXCEEDENCE CHARACTERISTICS AT OFFSITE STREAM LOCATIONS

Miller Creek below Lake Reba (Node 7)										
		Average Daily Flow Rate (cfs)								
		Current Conditions			Proposed Conditions			Difference		
Period	Month	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow
1	Jan	0.7	2.7	10.2	0.7	2.9	10.9	0.0	0.2	0.7
2		0.9	3.1	10.2	0.9	3.3	10.9	0.0	0.2	0.7
3		0.7	2.9	11.9	0.8	3.0	12.7	0.1	0.1	0.8
4		0.8	3.4	10.4	0.8	3.6	11.1	0.0	0.2	0.7
5	Feb	0.6	2.3	8.5	0.6	2.4	9.1	0.0	0.1	0.6
6		0.8	2.5	8.4	0.8	2.6	9.0	0.0	0.1	0.6
7		0.7	3.1	9.6	0.7	3.3	10.3	0.0	0.2	0.7
8		0.7	2.3	7.8	0.7	2.4	8.4	0.0	0.1	0.6
9	Mar	0.8	2.1	7.3	0.8	2.2	7.7	0.0	0.1	0.4
10		0.6	1.9	6.5	0.6	2.0	7.0	0.0	0.1	0.5
11		0.5	1.5	4.7	0.5	1.5	5.1	0.0	0.0	0.4
12		0.6	1.6	4.4	0.6	1.7	4.7	0.0	0.1	0.3
13	Apr	0.4	1.0	4.2	0.4	1.1	4.5	0.0	0.1	0.3
14		0.4	1.0	3.3	0.4	1.1	3.6	0.0	0.1	0.3
15		0.3	0.8	3.2	0.3	0.8	3.4	0.0	0.0	0.2
16		0.3	0.7	1.7	0.4	0.7	1.8	0.1	0.0	0.1
17	May	0.3	0.5	1.3	0.3	0.6	1.5	0.0	0.1	0.2
18		0.3	0.4	0.9	0.3	0.5	1.0	0.0	0.1	0.1
19		0.2	0.4	0.8	0.2	0.4	0.9	0.0	0.0	0.1
20		0.2	0.4	1.1	0.2	0.4	1.2	0.0	0.0	0.1
21	Jun	0.2	0.3	1.3	0.2	0.4	1.5	0.0	0.1	0.2
22		0.2	0.3	1.0	0.2	0.3	1.1	0.0	0.0	0.1
23		0.2	0.3	0.7	0.2	0.3	0.8	0.0	0.0	0.1
24		0.1	0.3	0.7	0.1	0.3	0.8	0.0	0.0	0.1
25	Jul	0.1	0.2	0.5	0.1	0.2	0.6	0.0	0.0	0.1
26		0.1	0.2	0.5	0.1	0.2	0.6	0.0	0.0	0.1
27		0.1	0.2	0.4	0.1	0.2	0.4	0.0	0.0	0.0
28		0.1	0.1	0.3	0.1	0.1	0.3	0.0	0.0	0.0
29	Aug	0.1	0.1	0.3	0.1	0.1	0.4	0.0	0.0	0.1
30		0.1	0.1	0.3	0.1	0.1	0.4	0.0	0.0	0.1
31		0.1	0.1	0.4	0.1	0.1	0.5	0.0	0.0	0.1
32		0.1	0.1	0.6	0.1	0.2	0.8	0.0	0.1	0.2
33	Sep	0.0	0.1	0.5	0.0	0.1	0.6	0.0	0.0	0.1
34		0.1	0.1	0.5	0.1	0.2	0.6	0.0	0.1	0.1
35		0.0	0.2	0.9	0.0	0.2	1.1	0.0	0.0	0.2
36		0.0	0.2	1.2	0.0	0.2	1.5	0.0	0.0	0.3
37	Oct	0.1	0.2	1.3	0.1	0.2	1.6	0.0	0.0	0.3
38		0.1	0.3	1.8	0.1	0.4	2.2	0.0	0.1	0.4
39		0.1	0.4	1.8	0.1	0.5	2.3	0.0	0.1	0.5
40		0.2	0.7	2.4	0.3	0.9	2.9	0.1	0.2	0.5
41	Nov	0.2	1.0	3.8	0.2	1.2	4.5	0.0	0.2	0.7
42		0.3	1.7	6.1	0.3	1.9	6.9	0.0	0.2	0.8
43		0.3	2.0	8.2	0.4	2.2	9.0	0.1	0.2	0.8
44		0.5	3.0	9.3	0.6	3.3	9.9	0.1	0.3	0.6
45	Dec	0.8	3.3	9.0	0.9	3.6	9.7	0.1	0.3	0.7
46		0.8	3.1	9.4	0.8	3.3	10.1	0.0	0.2	0.7
47		0.7	3.0	9.8	0.8	3.2	10.6	0.1	0.2	0.8
48		0.9	3.0	8.7	0.9	3.2	9.3	0.0	0.2	0.6

Notes:

1) Low, median and high flow are defined as flows exceeding 90%, 50% and 10% of the time, respectively.

2) Each month is divided into 4 equal periods, for a total of 48 periods in the year.

3) Flows are based on HSPF model results for a 1948-1994 simulation period.

TABLE 4-4 (CONTINUED)

FLOW EXCEEDENCE CHARACTERISTICS AT OFFSITE STREAM LOCATIONS

Des Moines Creek below Confluence (Node 13)										
		Average Daily Flow Rate (cfs)								
		Current Conditions			Proposed Conditions			Difference		
Period	Month	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow
1	Jan	2.4	6.8	17.4	2.3	6.8	18.3	-0.1	0.0	0.9
2		3.0	7.6	17.6	2.8	7.7	18.4	-0.2	0.1	0.8
3		2.6	7.3	19.4	2.4	7.3	20.2	-0.2	0.0	0.8
4		2.6	8.1	18.1	2.5	8.1	18.8	-0.1	0.0	0.7
5	Feb	2.3	6.5	15.4	2.1	6.3	15.8	-0.2	-0.2	0.4
6		3.1	6.6	15.5	2.9	6.6	15.9	-0.2	0.0	0.4
7		2.8	7.7	16.4	2.6	7.6	17.0	-0.2	-0.1	0.6
8		2.8	6.5	14.3	2.7	6.3	14.6	-0.1	-0.2	0.3
9	Mar	3.1	5.9	13.8	2.9	5.8	14.1	-0.2	-0.1	0.3
10		2.7	5.7	12.9	2.5	5.6	13.1	-0.2	-0.1	0.2
11		2.4	4.9	10.4	2.2	4.7	10.5	-0.2	-0.2	0.1
12		2.8	5.1	9.3	2.5	4.9	9.5	-0.3	-0.2	0.2
13	Apr	2.2	4.0	9.7	2.0	3.9	9.8	-0.2	-0.1	0.1
14		2.0	4.1	8.3	1.8	3.9	8.4	-0.2	-0.2	0.1
15		1.8	3.6	7.8	1.6	3.4	7.9	-0.2	-0.2	0.1
16		1.9	3.4	5.5	1.8	3.3	5.5	-0.1	-0.1	0.0
17	May	2.0	2.9	4.8	1.8	2.7	4.8	-0.2	-0.2	0.0
18		1.6	2.6	4.1	1.5	2.4	4.1	-0.1	-0.2	0.0
19		1.5	2.4	4.0	1.3	2.2	4.0	-0.2	-0.2	0.0
20		1.5	2.5	4.5	1.4	2.4	4.6	-0.1	-0.1	0.1
21	Jun	1.3	2.3	5.2	1.2	2.1	5.4	-0.1	-0.2	0.2
22		1.3	2.1	4.3	1.1	2.0	4.3	-0.2	-0.1	0.0
23		1.2	1.9	3.7	1.1	1.7	3.7	-0.1	-0.2	0.0
24		1.2	1.9	3.6	1.0	1.8	3.7	-0.2	-0.1	0.1
25	Jul	1.2	1.7	3.2	1.0	1.6	3.2	-0.2	-0.1	0.0
26		1.0	1.6	3.2	0.9	1.5	3.2	-0.1	-0.1	0.0
27		1.0	1.4	2.5	0.9	1.3	2.4	-0.1	-0.1	-0.1
28		1.0	1.3	2.1	0.9	1.2	2.0	-0.1	-0.1	-0.1
29	Aug	0.9	1.2	2.4	0.8	1.1	2.3	-0.1	-0.1	-0.1
30		0.9	1.2	2.5	0.8	1.1	2.5	-0.1	-0.1	0.0
31		0.8	1.2	3.2	0.7	1.1	3.2	-0.1	-0.1	0.0
32		0.8	1.4	3.9	0.8	1.3	4.0	0.0	-0.1	0.1
33	Sep	0.8	1.4	3.4	0.7	1.3	3.5	-0.1	-0.1	0.1
34		0.8	1.5	3.1	0.7	1.4	3.3	-0.1	-0.1	0.2
35		0.8	1.8	5.1	0.7	1.7	5.4	-0.1	-0.1	0.3
36		0.8	1.9	6.0	0.7	1.8	6.4	-0.1	-0.1	0.4
37	Oct	0.8	1.8	5.7	0.7	1.7	6.1	-0.1	-0.1	0.4
38		0.9	2.4	7.2	0.8	2.4	7.8	-0.1	0.0	0.6
39		0.9	2.6	8.2	0.8	2.6	8.9	-0.1	0.0	0.7
40		1.6	3.9	9.4	1.5	4.1	10.3	-0.1	0.2	0.9
41	Nov	1.6	4.5	11.5	1.5	4.7	12.6	-0.1	0.2	1.1
42		1.9	6.0	14.6	1.8	6.3	16.1	-0.1	0.3	1.5
43		1.9	6.1	15.9	1.9	6.4	17.3	0.0	0.3	1.4
44		2.6	7.3	16.0	2.6	7.7	17.0	0.0	0.4	1.0
45	Dec	3.1	8.1	15.9	3.0	8.4	17.0	-0.1	0.3	1.1
46		2.8	7.5	16.5	2.8	7.7	17.5	0.0	0.2	1.0
47		2.7	7.0	17.1	2.6	7.1	18.1	-0.1	0.1	1.0
48		3.0	7.2	14.9	2.9	7.3	15.5	-0.1	0.1	0.6

Notes:
1) Low, median and high flow are defined as flows exceeding 90%, 50% and 10% of the time, respectively.
2) Each month is divided into 4 equal periods, for a total of 48 periods in the year.
3) Flows are based on HSPF model results for a 1948-1994 simulation period.

TABLE 4-4 (CONTINUED)

FLOW EXCEEDENCE CHARACTERISTICS AT OFFSITE STREAM LOCATIONS

Des Moines Creek at S 208th Street (Node 18)										
		Average Daily Flow Rate (cfs)								
		Current Conditions			Proposed Conditions			Difference		
Period	Month	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow	Low Flow	Median Flow	High Flow
1	Jan	2.9	8.0	20.2	2.8	8.0	21.0	-0.1	0.0	0.8
2		3.5	9.0	20.5	3.4	9.0	21.3	-0.1	0.0	0.8
3		3.1	8.7	22.5	3.0	8.6	23.3	-0.1	-0.1	0.8
4		3.2	9.6	21.1	3.0	9.6	21.7	-0.2	0.0	0.6
5	Feb	2.8	7.8	17.9	2.6	7.6	18.3	-0.2	-0.2	0.4
6		3.8	7.9	18.0	3.6	7.8	18.5	-0.2	-0.1	0.5
7		3.4	9.1	19.1	3.2	9.0	19.6	-0.2	-0.1	0.5
8		3.5	7.7	16.7	3.3	7.6	17.0	-0.2	-0.1	0.3
9	Mar	3.7	7.0	16.2	3.6	6.9	16.5	-0.1	-0.1	0.3
10		3.3	6.8	15.0	3.1	6.6	15.3	-0.2	-0.2	0.3
11		2.9	5.8	12.3	2.7	5.7	12.3	-0.2	-0.1	0.0
12		3.4	6.0	10.9	3.1	5.9	11.1	-0.3	-0.1	0.2
13	Apr	2.7	4.8	11.3	2.5	4.7	11.5	-0.2	-0.1	0.2
14		2.4	4.9	9.7	2.3	4.7	9.8	-0.1	-0.2	0.1
15		2.2	4.4	9.1	2.1	4.2	9.2	-0.1	-0.2	0.1
16		2.4	4.2	6.5	2.2	4.0	6.5	-0.2	-0.2	0.0
17	May	2.5	3.5	5.7	2.3	3.3	5.7	-0.2	-0.2	0.0
18		2.0	3.1	4.9	1.9	2.9	4.9	-0.1	-0.2	0.0
19		1.8	2.9	4.8	1.7	2.7	4.7	-0.1	-0.2	-0.1
20		1.9	3.1	5.4	1.7	2.9	5.5	-0.2	-0.2	0.1
21	Jun	1.7	2.7	6.2	1.5	2.6	6.3	-0.2	-0.1	0.1
22		1.6	2.6	5.0	1.4	2.5	5.1	-0.2	-0.1	0.1
23		1.6	2.3	4.5	1.4	2.2	4.5	-0.2	-0.1	0.0
24		1.5	2.3	4.3	1.3	2.2	4.4	-0.2	-0.1	0.1
25	Jul	1.4	2.1	3.8	1.3	2.0	3.8	-0.1	-0.1	0.0
26		1.3	2.0	3.8	1.2	1.9	3.8	-0.1	-0.1	0.0
27		1.2	1.8	3.0	1.1	1.6	2.9	-0.1	-0.2	-0.1
28		1.2	1.6	2.5	1.1	1.5	2.4	-0.1	-0.1	-0.1
29	Aug	1.1	1.5	2.9	1.0	1.4	2.8	-0.1	-0.1	-0.1
30		1.1	1.5	3.0	1.0	1.4	3.0	-0.1	-0.1	0.0
31		1.0	1.5	3.8	0.9	1.4	3.8	-0.1	-0.1	0.0
32		1.1	1.7	4.5	1.0	1.6	4.7	-0.1	-0.1	0.2
33	Sep	1.0	1.7	4.0	0.9	1.6	4.1	-0.1	-0.1	0.1
34		1.0	1.8	3.7	1.0	1.7	3.8	0.0	-0.1	0.1
35		1.0	2.1	5.9	0.9	2.1	6.2	-0.1	0.0	0.3
36		1.0	2.3	6.9	0.9	2.2	7.3	-0.1	-0.1	0.4
37	Oct	1.0	2.2	6.7	0.9	2.1	7.1	-0.1	-0.1	0.4
38		1.1	2.8	8.3	1.1	2.9	8.9	0.0	0.1	0.6
39		1.1	3.1	9.4	1.0	3.1	10.1	-0.1	0.0	0.7
40		1.9	4.6	10.8	1.9	4.8	11.6	0.0	0.2	0.8
41	Nov	1.9	5.3	13.3	1.8	5.5	14.3	-0.1	0.2	1.0
42		2.2	7.0	16.8	2.2	7.3	18.2	0.0	0.3	1.4
43		2.3	7.1	18.2	2.2	7.4	19.7	-0.1	0.3	1.5
44		3.0	8.5	18.5	3.0	8.9	19.5	0.0	0.4	1.0
45	Dec	3.7	9.4	18.3	3.6	9.7	19.4	-0.1	0.3	1.1
46		3.4	8.8	19.0	3.3	9.0	20.0	-0.1	0.2	1.0
47		3.2	8.2	19.9	3.1	8.4	20.8	-0.1	0.2	0.9
48		3.6	8.4	17.3	3.5	8.5	17.9	-0.1	0.1	0.6

Notes:
1) Low, median and high flow are defined as flows exceeding 90%, 50% and 10% of the time, respectively.
2) Each month is divided into 4 equal periods, for a total of 48 periods in the year.
3) Flows are based on HSPF model results for a 1948-1994 simulation period.

4.4 LOW FLOWS

The discussion of HSPF modeling results in Sections 4.2 and 4.3 included an assessment of how the proposed development would affect low flows in Miller Creek and Des Moines Creek. In the analysis of annual flow duration, the total low flow volume below the 6 cfs magnitude was predicted to decrease by about 3 percent in Miller Creek, and the total low flow volume below the 10 cfs magnitude was predicted to decrease by about 6 percent in Des Moines Creek. In the seasonal flow exceedence analysis, the summer stream flow rates were predicted to decrease by up to 0.1 cfs in Miller Creek and by up to 0.2 cfs in Des Moines Creek.

To verify these HSPF-modeled estimates, a separate analysis was conducted to review the potential effects of land use changes on low flows. The basis of this analysis was an assumption that land use changes that increase the impervious area within a basin will result in a proportional reduction in rainfall infiltration to groundwater aquifers. Since summer low flows are supplied by groundwater sources, a change in groundwater recharge will most likely have a similar effect on the magnitude of low flows in the streams. The evaluation of effects on low flows was made at one point on each stream, at the lower-most points in the modeled systems.

Changes to Groundwater Recharge Potential

In this analysis, groundwater recharge refers to water that reaches deeper aquifers. It does not include recharge of the interflow zone. The interflow zone is the shallow soil layer near the surface that typically supplies water to streams for short to intermediate periods of time following a rainfall event. In contrast, discharge from aquifers is the predominant source of water for streams during extended periods of dry weather, which typically occurs during late summer and early fall. Interflow (also called subsurface flow) is the predominant runoff mechanism in areas of glacial till deposits and groundwater flow is the predominate runoff mechanism on glacial outwash deposits (Dinicola, 1990). Thus, development in areas of outwash soils would have a much greater potential for affecting groundwater recharge and low streamflows than development in areas of till soils. The change in groundwater recharge potential was calculated from the change in land use.

Table 4-5 summarizes areas of differing soil types and land use under existing and proposed conditions, and the net change between the two. Areas that describe the recharge potential of different soils and land cover in the Miller and Des Moines Creek basins were categorized as till soil, outwash soil, wetland (or saturated) soil, and impervious area. Outwash soil has the greatest infiltration capacity because it consists of unconsolidated sand and gravel that are highly permeable. Till has very little infiltration capacity because it consists of compacted silt and clay (hardpan) that have low permeability. The change in land areas (as detailed in Table 3-2) results from replacement of pervious areas with impervious pavement, or from replacement of highly permeable soils such as outwash soil with less permeable soil such as till or compacted soil. The area of impervious surface within each basin was based on the percent impervious values listed in the land use summary tables in Appendix A. The change in impervious area in Table 4-5 is smaller than that shown in Tables 3-1 and 3-2 because it includes a small loss of existing impervious area in existing developed areas.

TABLE 4-5
CHANGE IN LAND USE COVERAGE

Affected Land Areas		Miller Creek			Des Moines Creek		
Soil Type	Infiltration Rate (in/hr)	Existing (acres)	Proposed (acres)	Net Change (acres)	Existing (acres)	Proposed (acres)	Net Change (acres)
Till or Compacted	0.06	2005.6	2070.4	+64.8	1208.1	1112.8	-95.3
Outwash	1.4	1851.4	1692.3	-159.1	415.9	358.1	-57.8
Wetland	2.0	101.5	101.5	0	65.9	65.9	0
Impervious	0.0	1224.2	1318.4	+94.2	1010.2	1163.3	153.1
Total	--	**5182.7**	**5182.7**	**0**	**2700.1**	**2700.1**	**0**

Infiltration rates listed in Table 4-5 are based on the regional parameters developed by the USGS, which were derived from soil survey data published by the Soil Conservation Service. All classifications for a particular soil type (e.g., forest and open) were grouped together under an average infiltration rate to simplify the calculation. This resulted in average infiltration rates for till, outwash, and wetland soils of 0.06, 1.4, and 2.0 inches per hour, respectively. As in the HSPF analysis, new fill associated with the airport expansion was assumed to be hydraulically equivalent to till soil.

In the Miller Creek basin the change in land use will result in an increase of 64.8 acres of low permeable till soil and 94.2 acres of impervious surface, and a decrease of 159.1 acres of higher permeable outwash soil. In the Des Moines Creek basin, the change in land use will result in an increase of 153.1 acres of impervious surface, and a decrease of 95.3 acres of till soil and 57.8 acres of outwash soil.

To determine how these land changes could affect infiltration to groundwater aquifers, the areas in Table 4-5 were multiplied by their respective soil infiltration rates and then added together to derive an index that describes the potential for groundwater recharge before and after construction of the project. Although this method cannot be used to quantify the total amount of recharge occurring, it can be used to estimate the relative change in infiltration rates that is caused by changed land use. The groundwater recharge indices for Miller and Des Moines Creek are as follows:

For Miller Creek:

$$Q_{exist} = C1 * [(2005.6*.06)+(1851.4*1.4)+(101.5*2.0)] = C1*2915$$
$$Q_{prop} = C1 * [(2070.6*.06)+(1692.3*1.4)+(101.5*2.0)] = C1*2696$$
$$Q_{prop} = Q_{exist} * 0.93$$

For Des Moines Creek:

$$Q_{exist} = C1 * [(1208.1*.06)+(415.9*1.4)+(65.9*2.0)] = C1*787$$
$$Q_{prop} = C1 * [(1112.8*.06)+(358.1*1.4)+(65.9*2.0)] = C1*670$$
$$Q_{prop} = Q_{exist} * 0.89$$

Where

Q_{exist} is the groundwater recharge rate under existing conditions

Q_{prop} is the groundwater recharge rate under proposed conditions

We conclude that, due to the proposed land use changes, potential groundwater recharge rates will decrease by approximately 7 percent in the Miller Creek basin and by 11 percent in the Des Moines Creek basin. These estimates should be considered approximate because groundwater recharge and discharge processes are more complex than accounted for in this analysis.

Changes to Summer Low Flows Due to Land Use Change

To estimate the effect of a reduction of groundwater recharge on low flows, it was assumed that low flows would be reduced in direct proportion to the reduction in potential groundwater recharge. The percentage decrease in potential groundwater recharge that was calculated above was applied to monthly low flows to estimate the net reduction in streamflow. Monthly low flows for existing conditions were obtained from the flow exceedence analysis that was summarized in Table 4-4. The low flows predicted in the HSPF modelling process were compared to historical flow monitoring data and it was found that the modelled flows generally corresponded to the historical flow monitoring data. For example, recorded flows from the late summer in the 1988-1994 time period at the mouth of Miller Creek typically ranged from 1.5-2.0 cfs. Those flow rates agree with the flow exceedence values from the HSPF simulation. The 1992-1994 monitoring data for Des Moines Creek near the mouth has a very similar range of summer low flow rates that, when translated upstream to the South 208th Street evaluation point, also generally agree with the simulation results.

The groundwater recharge analysis predicts that flows will reduce by about 0.1 cfs in both streams during late summer. The estimates for the predicted reduction of summer flows closely agree with the HSPF analysis summarized that was summarized in Table 4-4.

TABLE 4-6
LOW FLOW CHANGES ASSUMING DIRECT INFLUENCE
OF LAND USE CHANGE ON GROUNDWATER RECHARGE

	Miller Creek near mouth			Des Moines Creek at S.208th		
Month	Existing Low Flows (cfs)	Low Flows under Proposed Conditions (cfs)	Change (cfs)	Existing Low Flows (cfs)	Low Flows under Proposed Conditions (cfs)	Change (cfs)
July	1.5	1.4	-0.1	1.0	0.9	-0.1
August	1.4	1.3	-0.1	0.8	0.7	-0.1
September	1.3	1.2	-0.1	0.8	0.7	-0.1
October	1.4	1.3	-0.1	0.8	0.7	-0.1

5.0 SUMMARY AND CONCLUSIONS

This hydrologic modeling study for the SeaTac Airport Master Plan Update EIS accomplished the following tasks and analyses:

- Hydrologic models of Miller and Des Moines Creek were assembled using the HSPF model from available stream and watershed data. The models were calibrated using recorded streamflow data from the period 1989-1994.

- The calibrated models were run for a 47 year simulation period (1948-1994) using hourly precipitation data from SeaTac Airport.

- Current flow regimes of Miller Creek and Des Moines Creek were derived from the results of the HSPF current conditions models. Flow statistics of flood frequency, annual flow duration, and seasonal flow exceedence were derived.

- Proposed condition models that incorporated proposed features of the Master Plan Update were created by modifying land uses to reflect the addition of the 8,500-foot 3rd runway and expansion of terminal facilities.

- Detention storage volumes and release rates for stormwater runoff were calculated and the performance of the detention storage facilities were simulated in the HSPF models. Detention criteria from the Stormwater Management Manual for the Puget Sound Basin were used.

- The proposed conditions models were run for the 47-year simulation period. Flow statistics of the resulting streamflow regimes in Miller Creek and Des Moines Creek were derived.

- Streamflow characteristics in Miller Creek and Des Moines Creek under proposed conditions were then compared to current conditions to determine the effect of stormwater discharge on the receiving streams.

The comparison of current and proposed streamflow regimes in Miller Creek and Des Moines Creek resulted in the following conclusions:

- The current 100-year flow magnitudes for Miller Creek near the mouth (at the sewage treatment plant) and Des Moines Creek at South 208th Street are estimated to be 468 cfs and 280 cfs, respectively.

- Peak flows in Miller Creek and Des Moines Creek will not increase if adequate stormwater detention storage is provided. Approximate detention storage volumes of 61 acre-feet and 31 acre-feet were calculated for Miller and Des Moines Creek, respectively. With those detention volumes the HSPF simulation showed that peak flows in the streams will not increase.

- The total flow volume in Miller Creek near the mouth is predicted to increase by approximately 300 acre-feet per year, or 6 percent of the average annual flow. Flow volumes in Des Moines Creek will increase by approximately 60 acre-feet per year, or 1 percent. The increases are caused by the additional impervious area, which reduces infiltration of rainfall.

- The flow duration analysis showed that the increase in runoff occurs below the 1.25-year return period flow rate, which is in the 80-120 cfs range for both streams. The increase in runoff volume above those flow rates is not significant.

- The seasonal flow exceedence analysis showed that summer streamflows during low flow years (i.e., exceeded 90 percent of the time) could decrease by up to about 0.1 cfs in Miller Creek, and by up to about 0.2 cfs in Des Moines Creek. A water balance analysis based on an analysis of land use changes resulted in a similar estimate of potential changes to low flows. Summer streamflow could increase during median and high flow years in Miller Creek, but decrease in Des Moines Creek. Average monthly flows in the winter could increase by up to 2.1 cfs on Miller Creek and 1.3 cfs on Des Moines Creek during wetter, higher flow years.

6.0 REFERENCES

Anne Symonds, 1992. Sea-Tac International IWS and Storm Water Systems, August 1992. Prepared for the Port of Seattle. Anne Symonds, Inc., Seattle, Washington.

Anne Symonds, 1994. WATERWORKS Model Data for SeaTac Airport. Prepared for the Port of Seattle. Anne Symonds, Inc., Seattle, Washington.

COE, 1992. HEC-FFA, Flood Frequency Analysis Users Manual. U.S. Army Corps of Engineers. Davis, California.

Dinicola R.S., 1990. Characterization and Simulation of Rainfall Runoff Relations for Headwater Basins in Western King and Snohomish Counties, Washington. U.S. Geological Survey, Tacoma, Washington.

Gambrell Urban, 1994. Des Moines Creek GIS Study, Technical Memo. Gambrell Urban, Inc., Seattle, Washington.

Herrera, 1989. Des Moines Creek Watershed Management Plan. Herrera Environmental Consultants, Seattle, Washington.

KCSWM, 1989. Brief Description Report of Ambaum Regional Water Quality Detention Pond. King County Surface Water Management Division, Seattle, Washington.

KCSWM, 1989. TR-20 Model Files for Des Moines Creek Pond C (Tyee Pond). King County Surface Water Management Division, Seattle, Washington.

KCSWM, 1991. Ambaum Regional Pond, Miller Creek, Design Drawings. King County Surface Water Management Division, Seattle, Washington.

KCSWM, 1992. Des Moines Creek Regional Pond, Tyee Valley Golf Course, As-Built Design Drawings. King County Surface Water Management Division, Seattle, Washington.

KCSWM, 1992. Lake Reba Regional Pond, Miller Creek, Design Drawings. King County Surface Water Management Division, Seattle, Washington.

KCSWM, 1994. SeaTac Precipitation and Evaporation WDM File. King County Surface Water Management Division, Seattle, Washington.

KCSWM, 1994. King County Surface Water Management Stream Gauge Data. King County Surface Water Management Division, Seattle, Washington.

KCSWM, no date. Lake Reba Operation and Maintenance Manual. King County Surface Water Management Division, Seattle, Washington.

KCSWM, no date. Tyee Regional Pond Operations and Maintenance Manual. King County Surface Water Management Division, Seattle, Washington.

Minton, G. 1995. Personal communication (conversation with K. Ritland, Montgomery Water Group) regarding pervious and impervious areas in SeaTac Drainage Basins. October 5, 1995.

NCDC, 1994. Hourly Precipitation Data, Seattle-Tacoma Airport. National Climatic Data Center, Ashville, North Carolina.

NHC, 1990. Miller Creek Regional Stormwater Detention Facilities Design Hydrologic Modeling. Northwest Hydraulic Consultants, Kent, Washington.

NHC, 1991. Miller Creek, Normandy Park, Washington, Limited Map Maintenance Study. Prepared for Federal Emergency Management Agency Region X. Northwest Hydraulic Consultants, Kent, Washington.

Parametrix, 1990. Miller Creek Regional Stormwater Detention Facilities Draft Feasibility Report. Prepared for King County Surface Water Management Division. Parametrix, Inc, Sumner, Washington.

Parametrix, 1992. Lake Reba Regional Detention Facility Dam Safety Analysis. Prepared for King County Surface Water Management Division. Parametrix, Inc, Sumner, Washington.

Parametrix, 1994. South Aviation Support Area Final Environmental Impact Statement. Prepared for U.S. Department of Transportation and Federal Aviation Administration. Parametrix, Inc., Sumner, Washington.

P&D Aviation, 1994. Preliminary Maps of SeaTac Master Use Plan Alternatives. P&D Aviation, Inc., Sacramento, California.

Waldron, H., 1962. Geologic Map of the Des Moines Quadrangle, Washington. U.S. Geological Survey, Department of the Interior, Washington, D.C.

USEPA, 1993. Hydrological Simulation Program FORTRAN, Users Manual for Release 10. Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Athens, GA.

USGS, 1983. 1:25,000-Scale Metric Topographic-Bathymetric Map of Burien, Washington. U.S. Geological Survey, Department of the Interior, Washington, D.C.

USGS, 1993. User Manual for SWSTAT, a Computer Program for Interactive Computation of Surface-Water Statistics. U.S. Geological Survey, Water-Resources Investigations Report 94-xxx. 10/14/93 draft.

WADOE, 1992. Stormwater Management Manual for the Puget Sound Basin, The Technical Manual. Washington Department of Ecology, Olympia, Washington.

FIGURES

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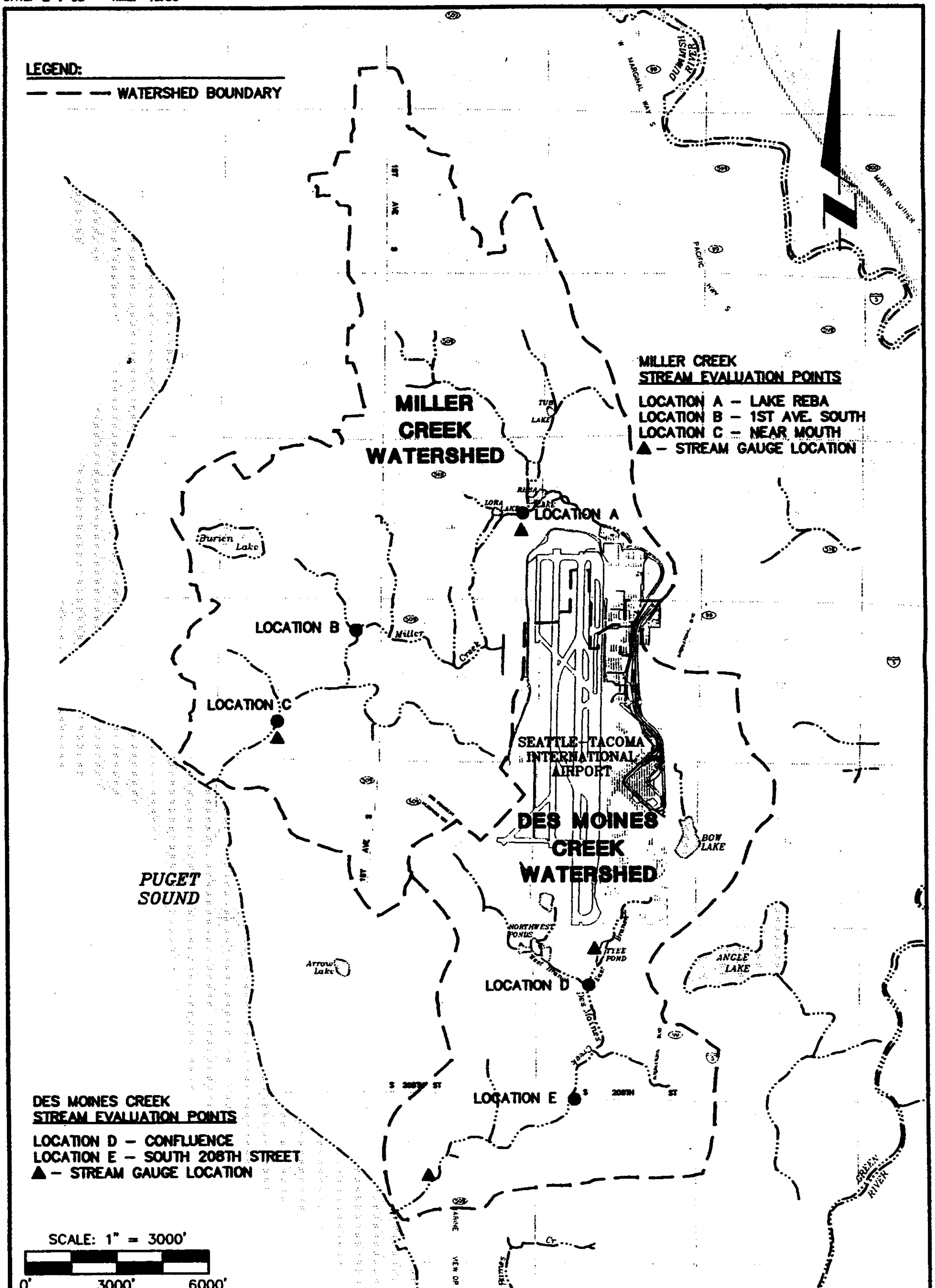

PROJECT NUMBER 94-121	DESIGNED / DRAWN / CHECKED / DATE 1-25-95 / R.A. MONTGOMERY PROJECT MANAGER	**FIGURE 1** MILLER CREEK AND DES MOINES CREEK WATERSHEDS

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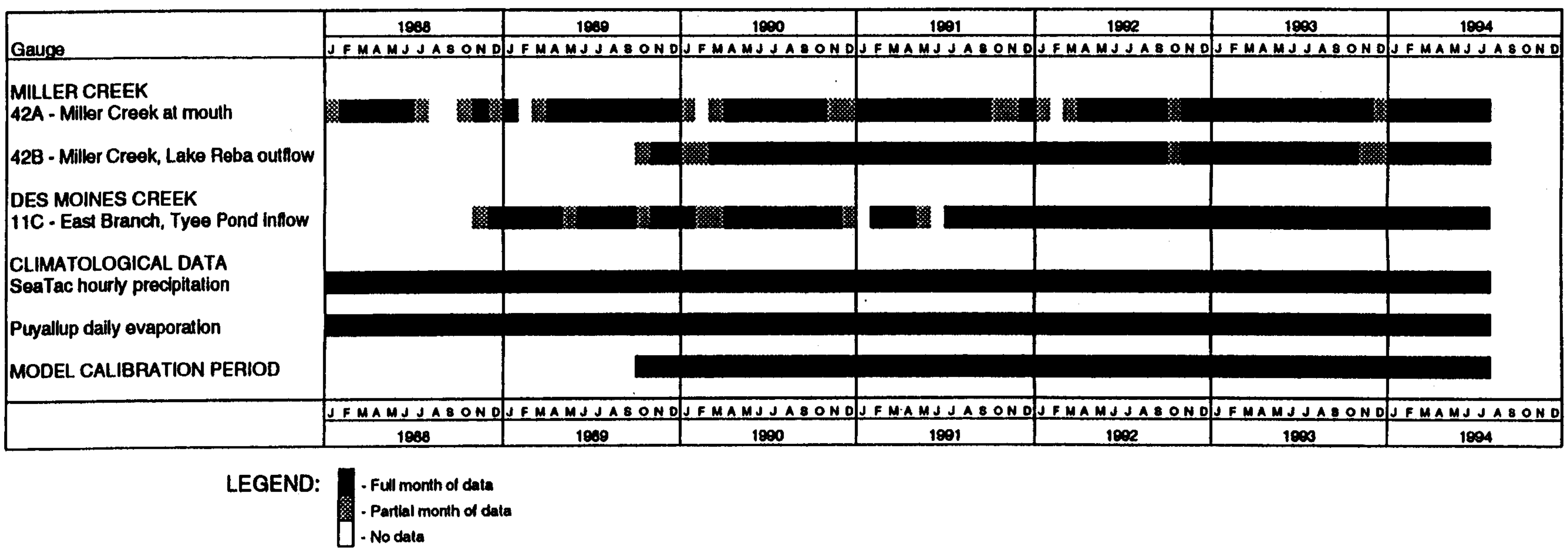

Availability of Recorded Streamflow Data

MONTGOMERY WATER GROUP, INC. Water Resources • Environmental • Civil Engineering	SeaTac Airport Master Plan Update EIS Hydrologic Modeling Study	Figure 2

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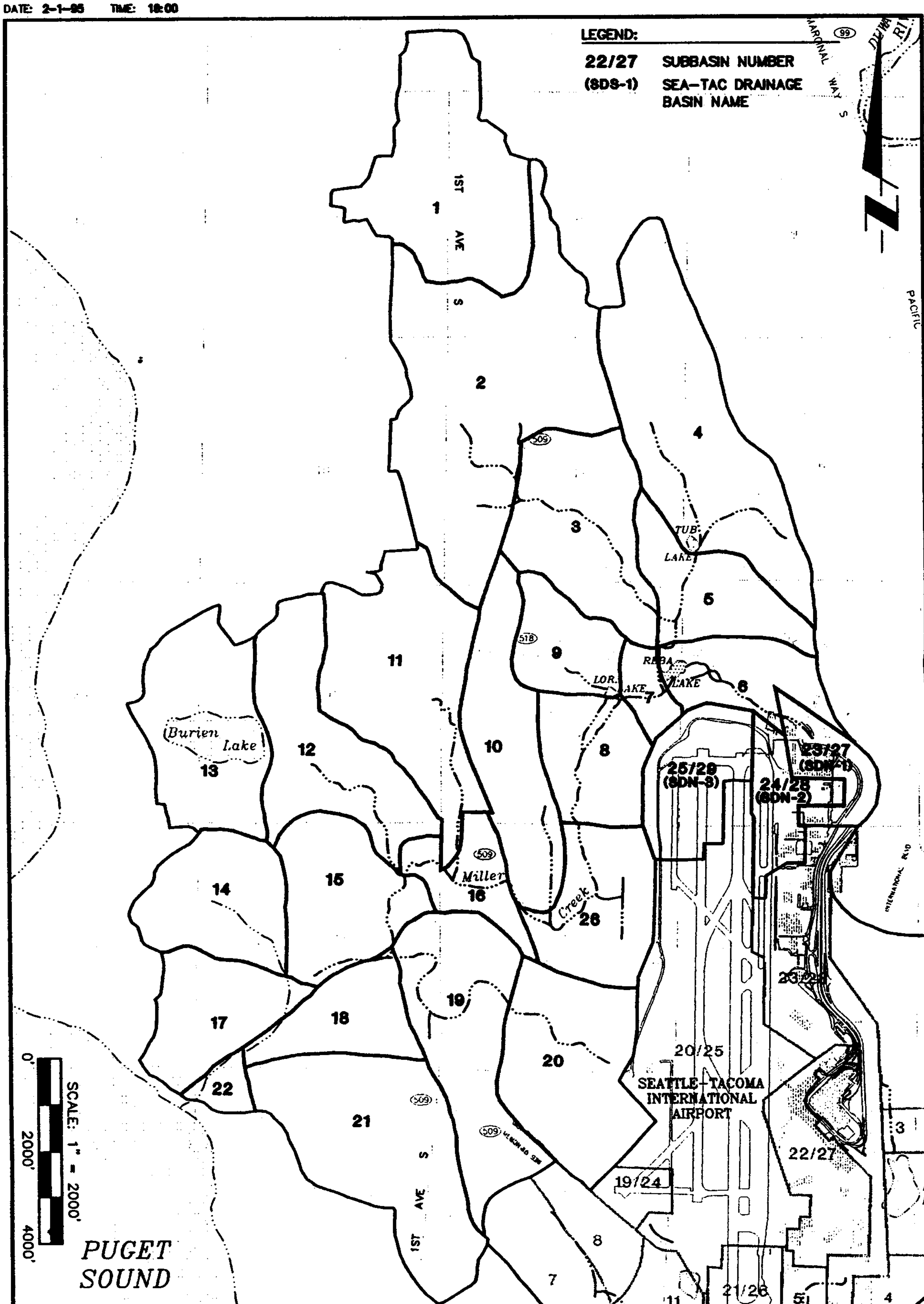

PROJECT NUMBER 94-121	FIGURE 3 MILLER CREEK WATERSHED SUBBASINS	MONTGOMERY WATER GROUP, INC. Water Resources · Environmental · Civil Engineering

DATE 1-20-95
R.A. MONTGOMERY
PROJECT MANAGER

AR 003469

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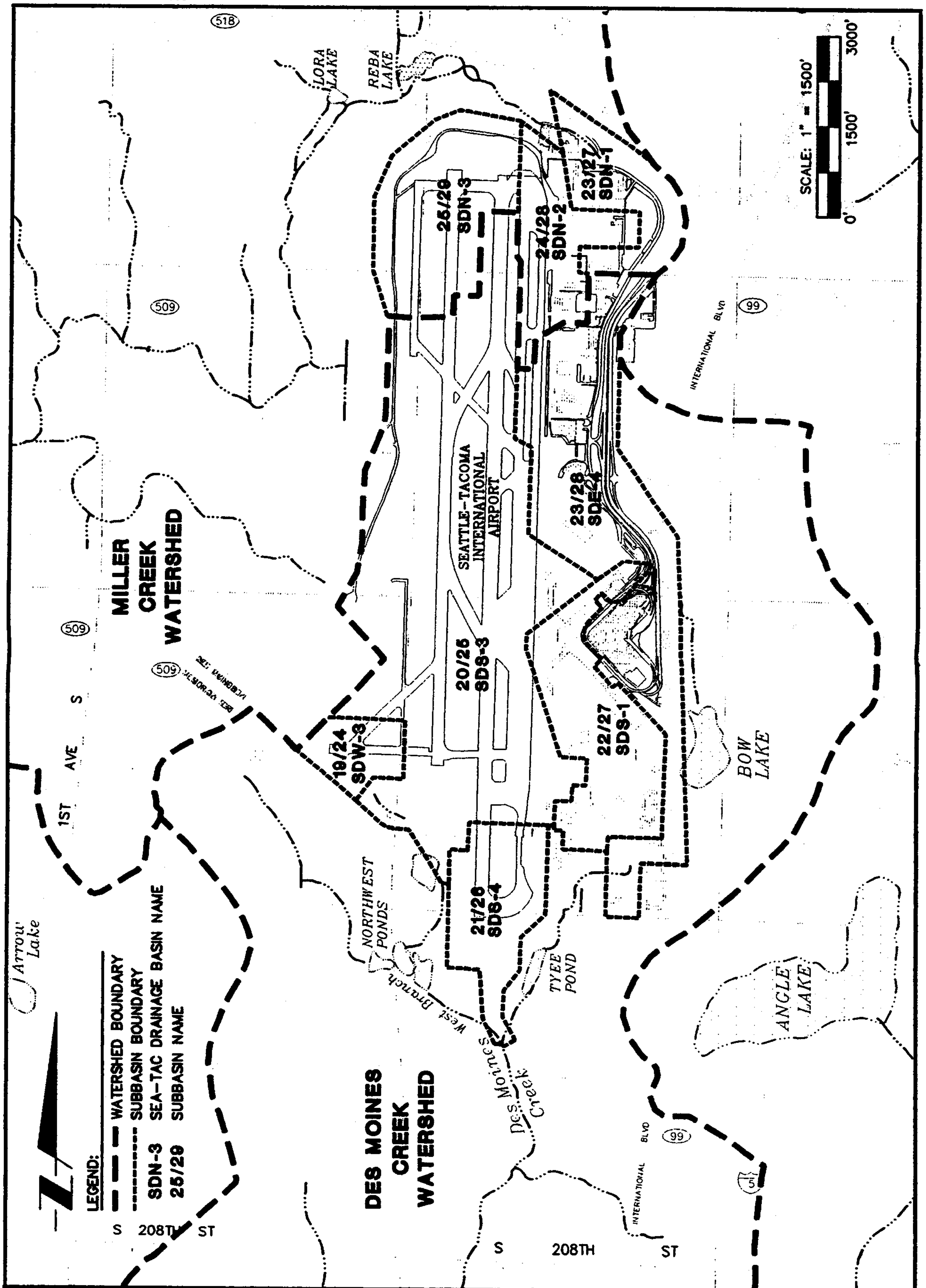

PROJECT NUMBER 94-121	DESIGNED K.W.K. / DRAWN N.B. / CHECKED R.A.M. / DATE 1-25-95 / R.A. MONTGOMERY PROJECT MANAGER	FIGURE 4 SEA-TAC AIRPORT SUBBASINS

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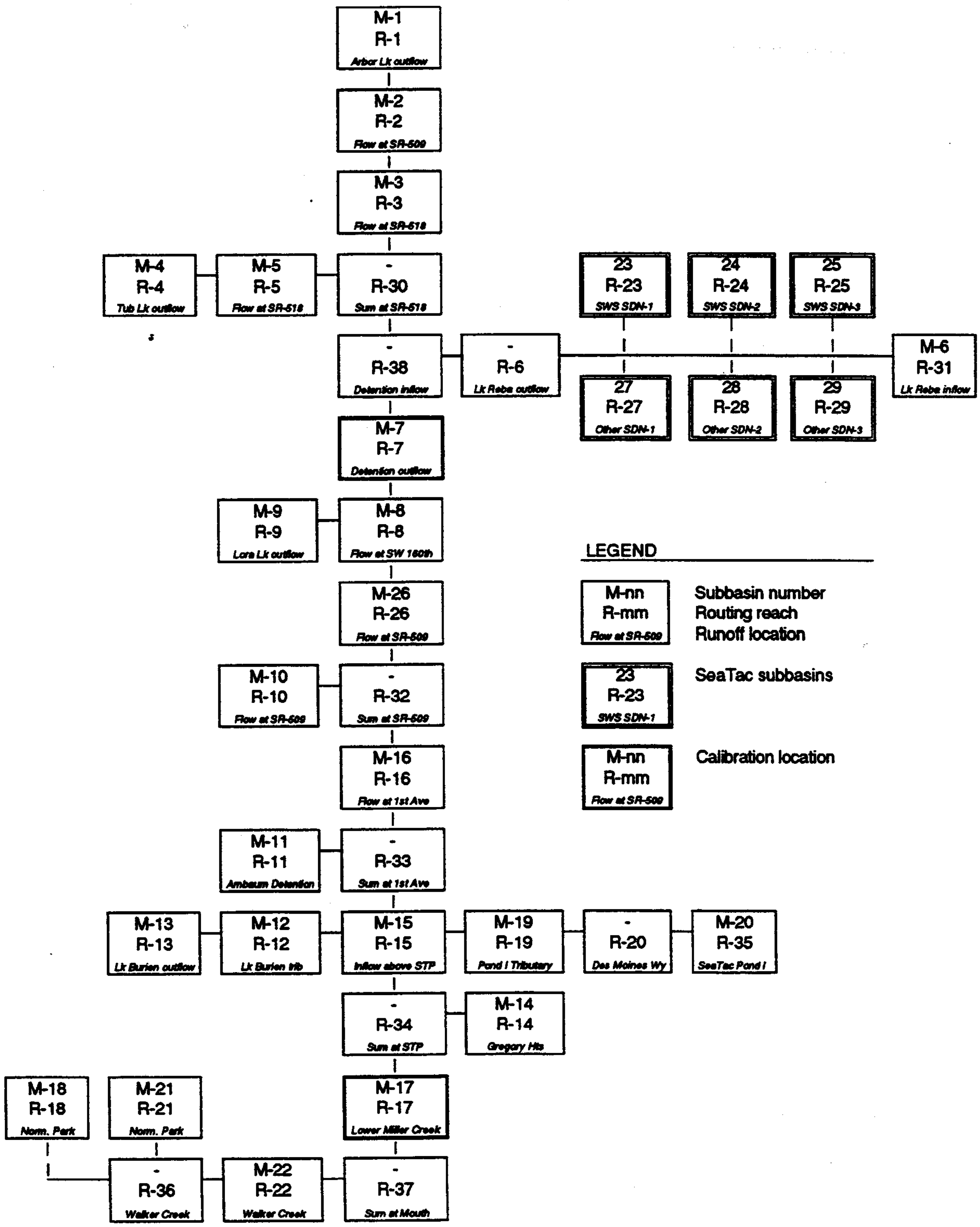

Miller Creek HSPF Model Schematic

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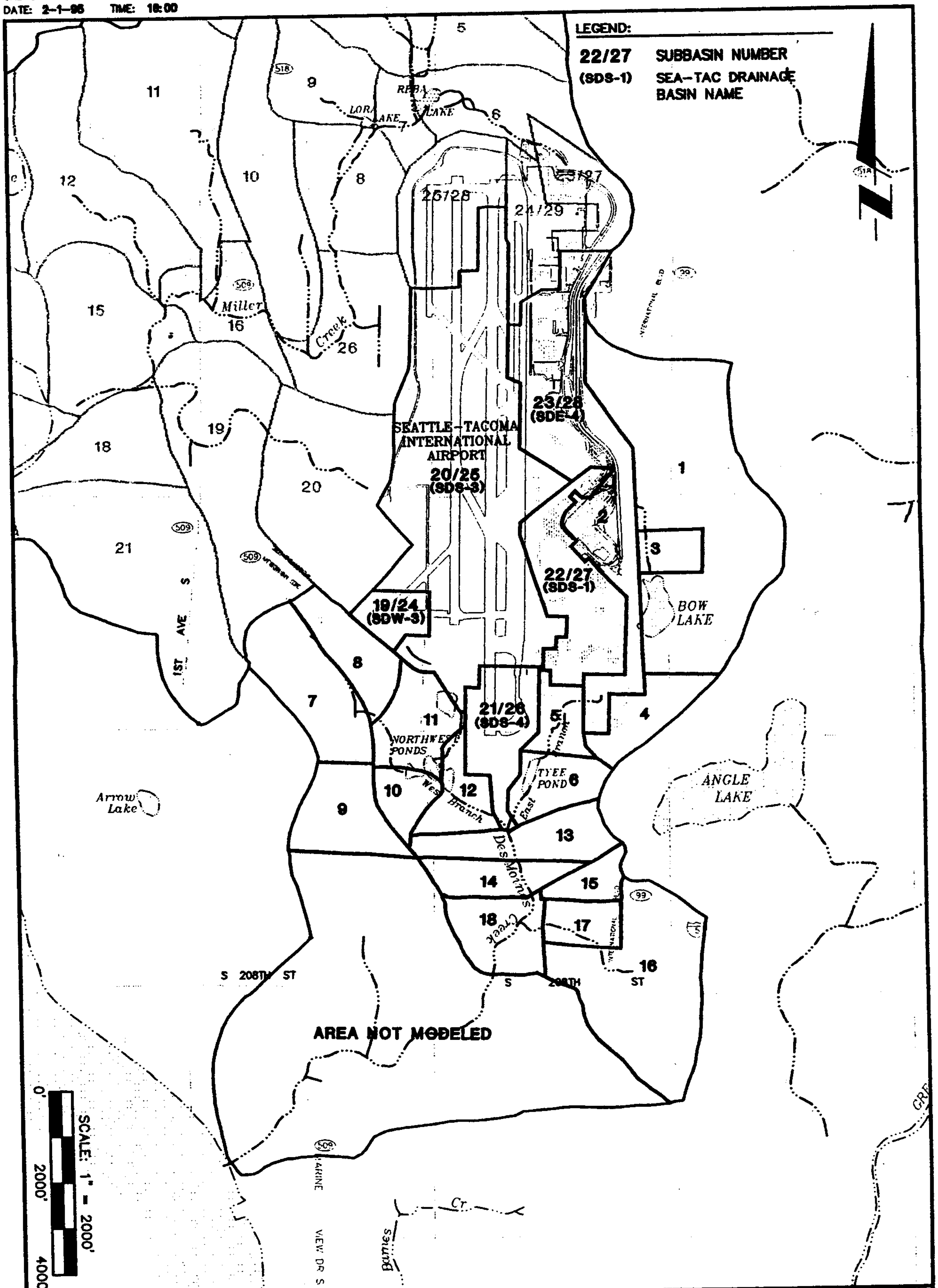

PROJECT NUMBER 94-121	DESIGNED R.M.M. / DRAWN N.B. / CHECKED R.A.M. / DATE 1-29-96 / R.A. MONTGOMERY PROJECT MANAGER	**FIGURE 6** DES MOINES WATERSHED SUBBASINS

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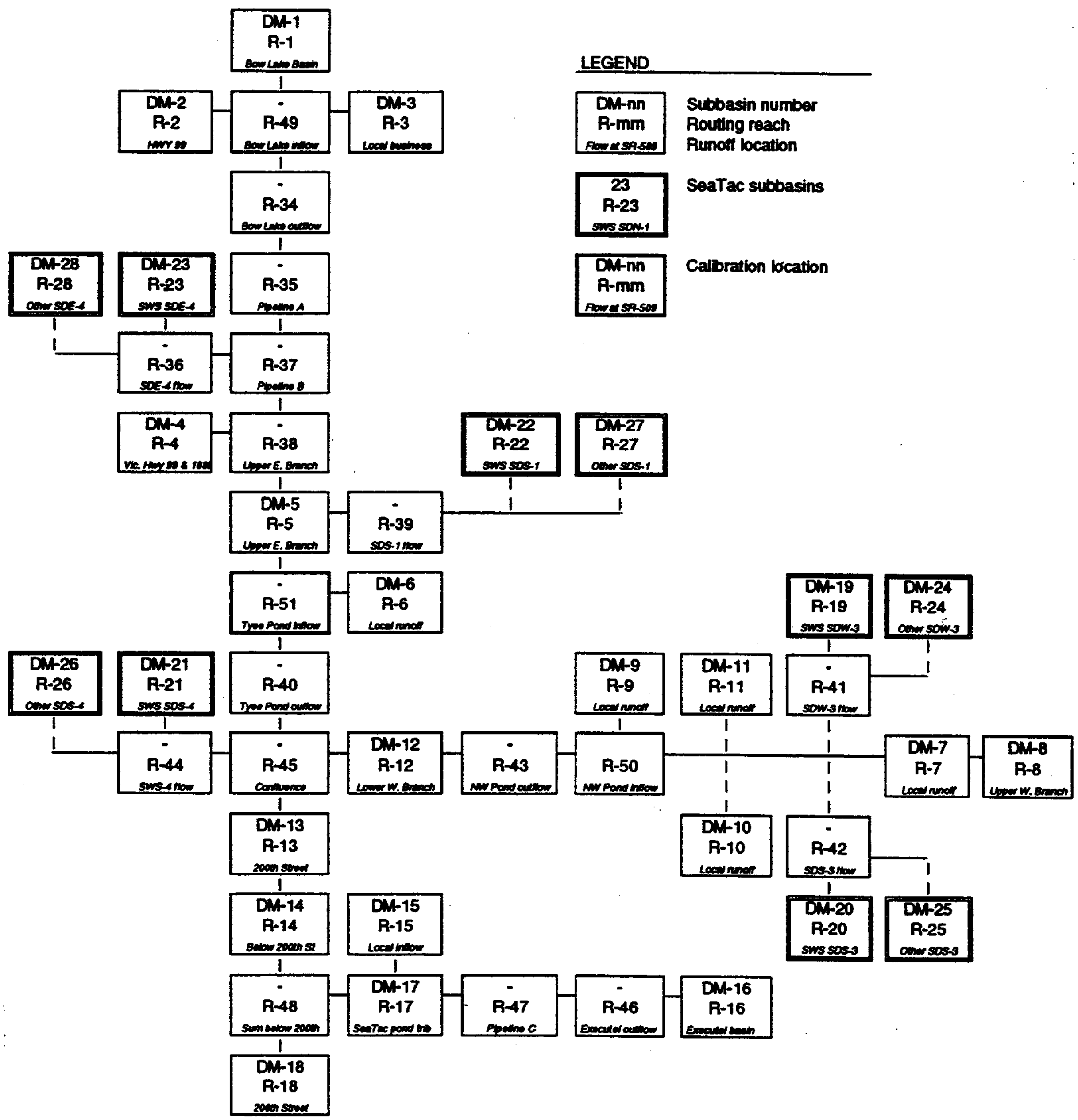

Des Moines Creek HSPF Model Schematic

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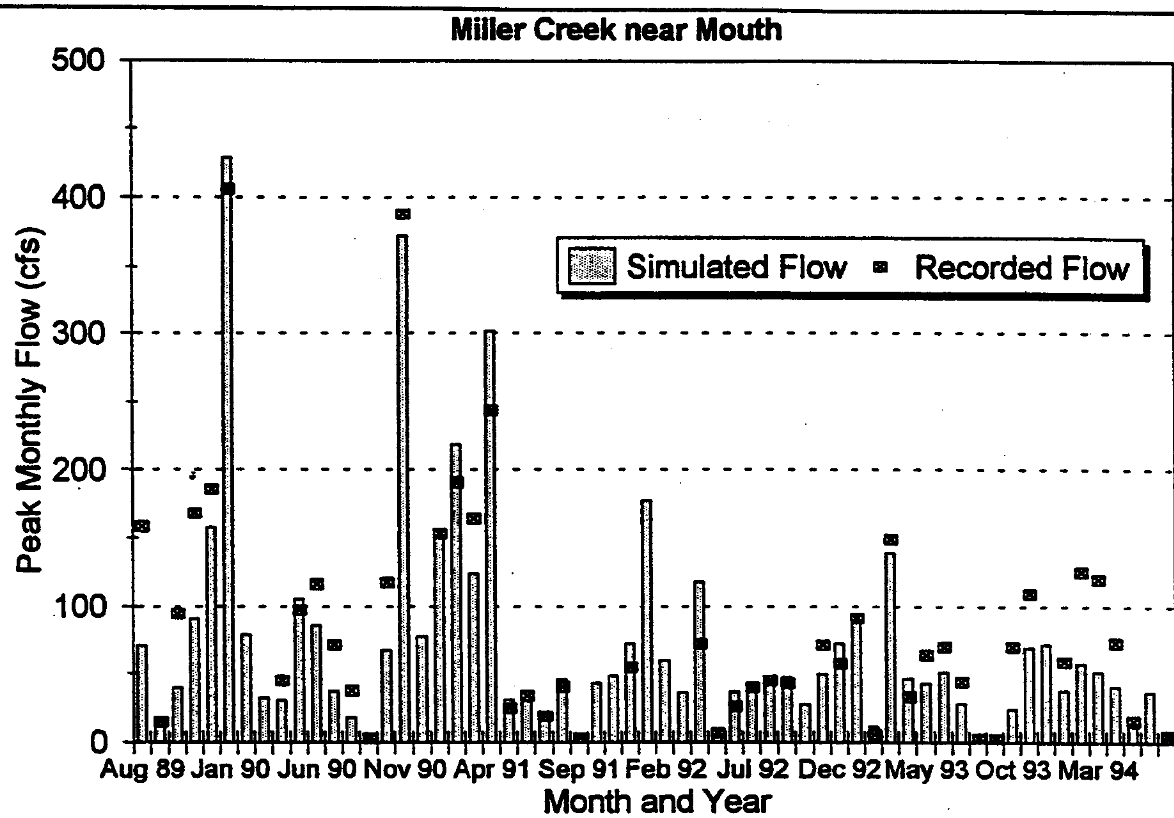
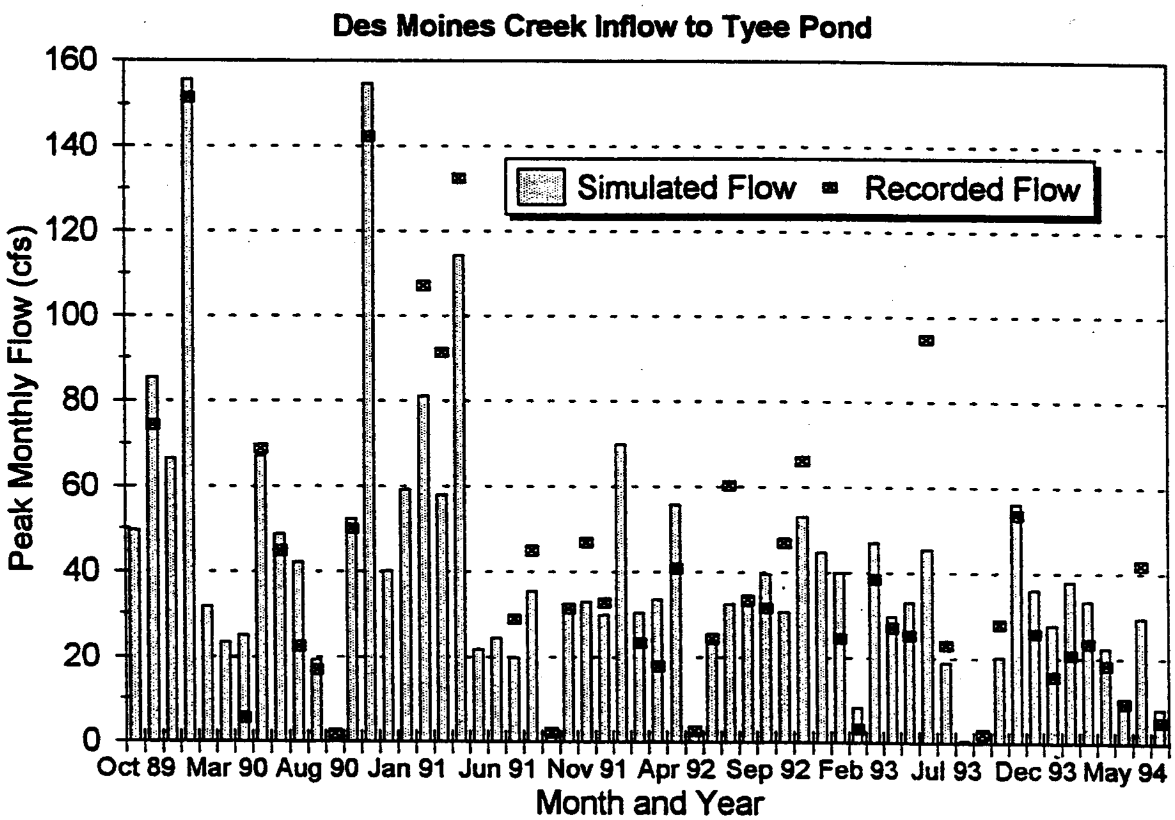
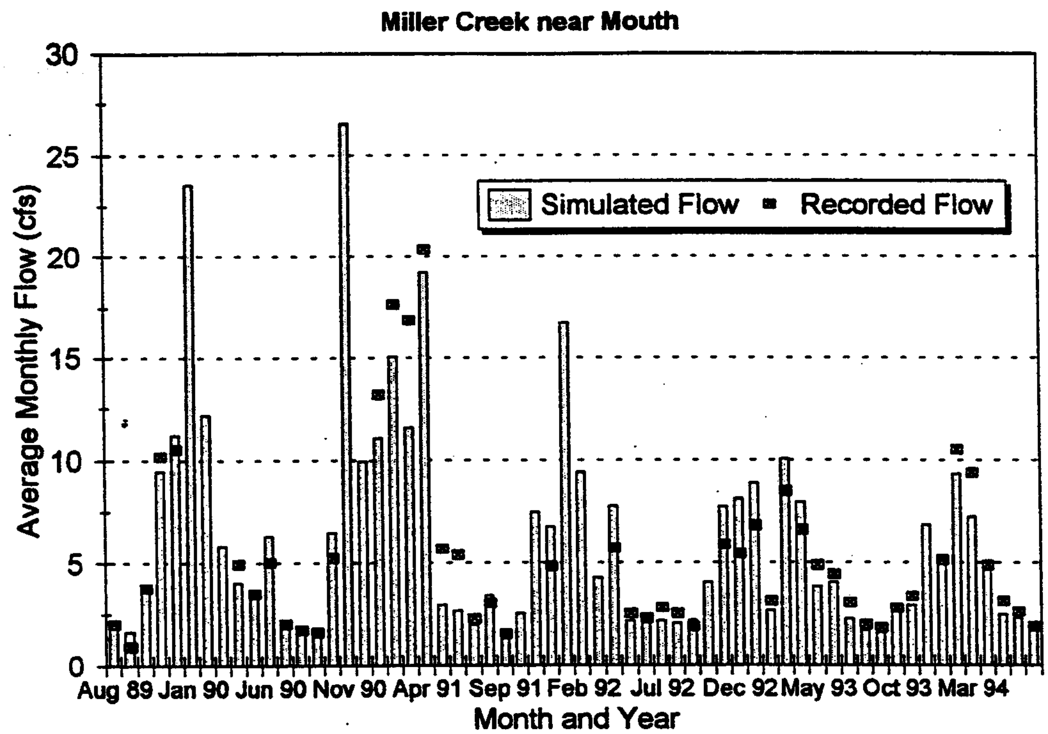
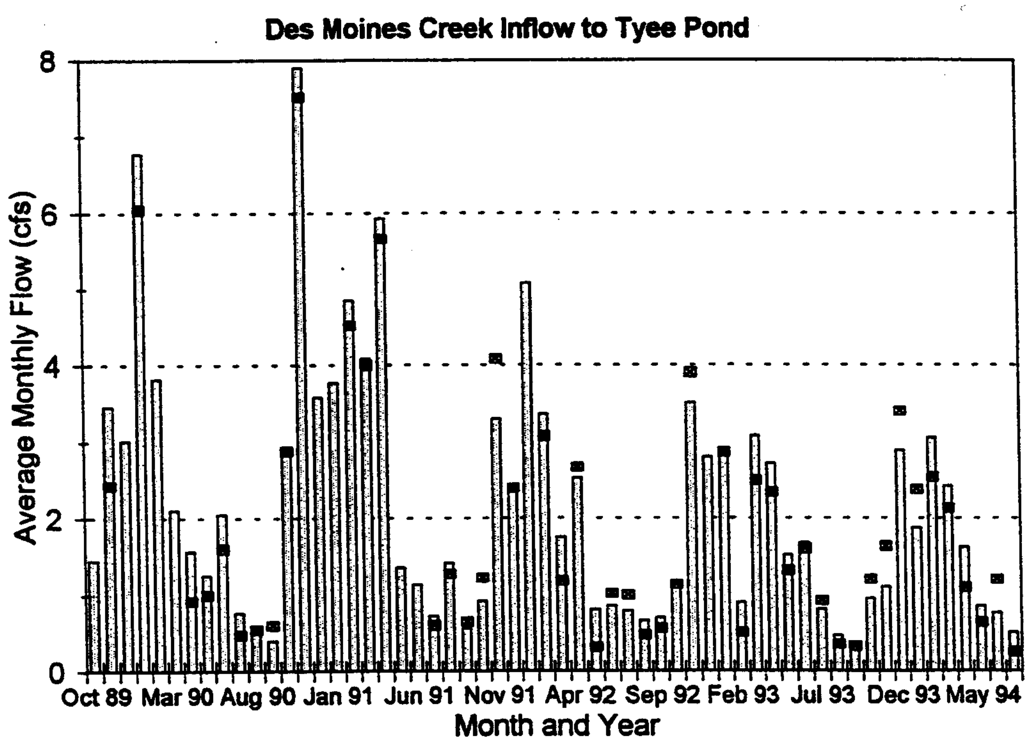




Calibration Results - Monthly Peak Flows

MONTGOMERY WATER GROUP, INC. Water Resources • Environmental • Civil Engineering	SeaTac Airport Master Plan Update EIS Hydrologic Modeling Study	Figure 8





Calibration Results - Monthly Average Flows

MONTGOMERY WATER GROUP, INC. Water Resources • Environmental • Civil Engineering	SeaTac Airport Master Plan Update EIS Hydrologic Modeling Study	Figure 9

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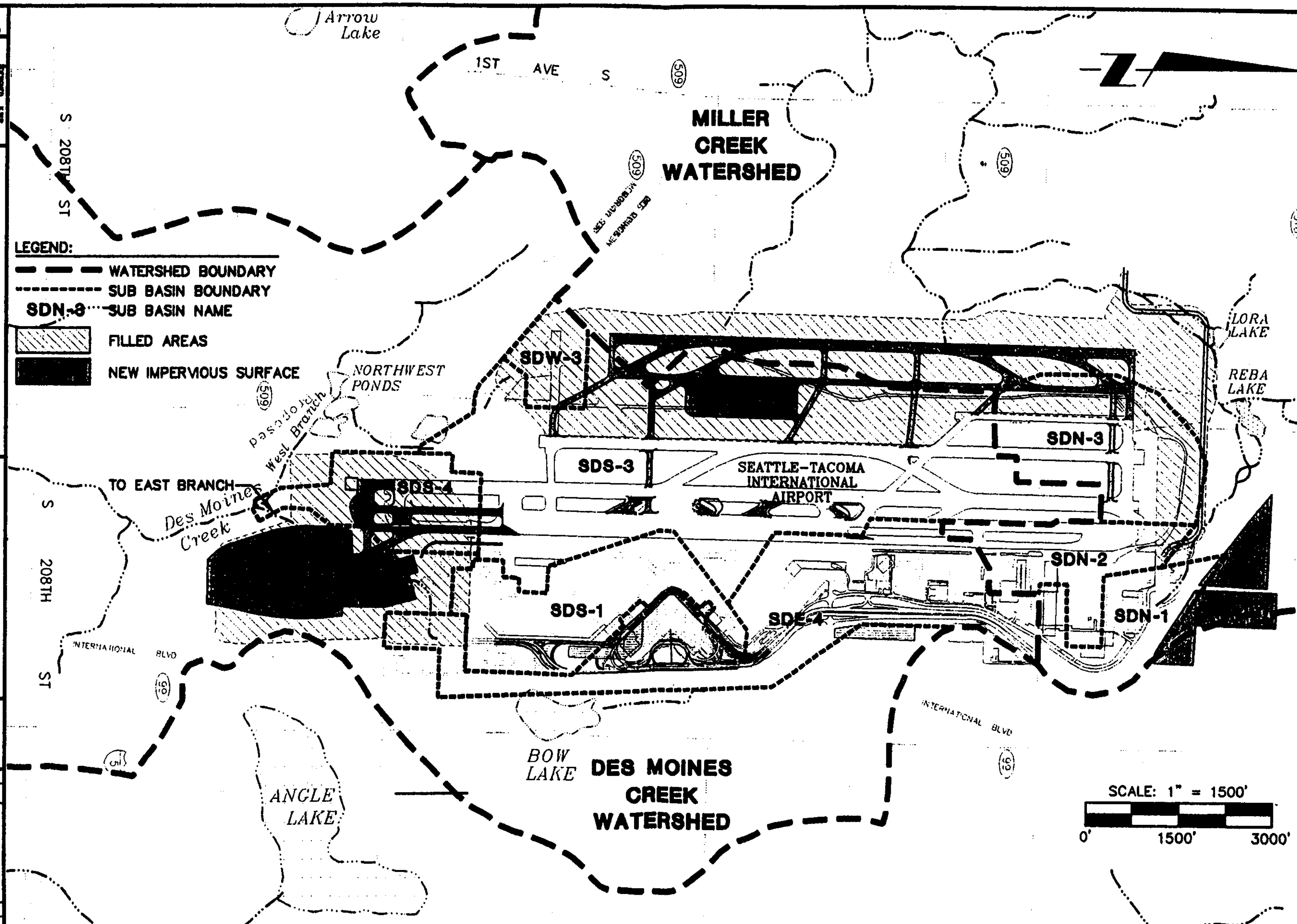

FIGURE 10
PROPOSED LAND USE CHANGES
AT SEA-TAC AIRPORT

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PROJECT NUMBER 94-121

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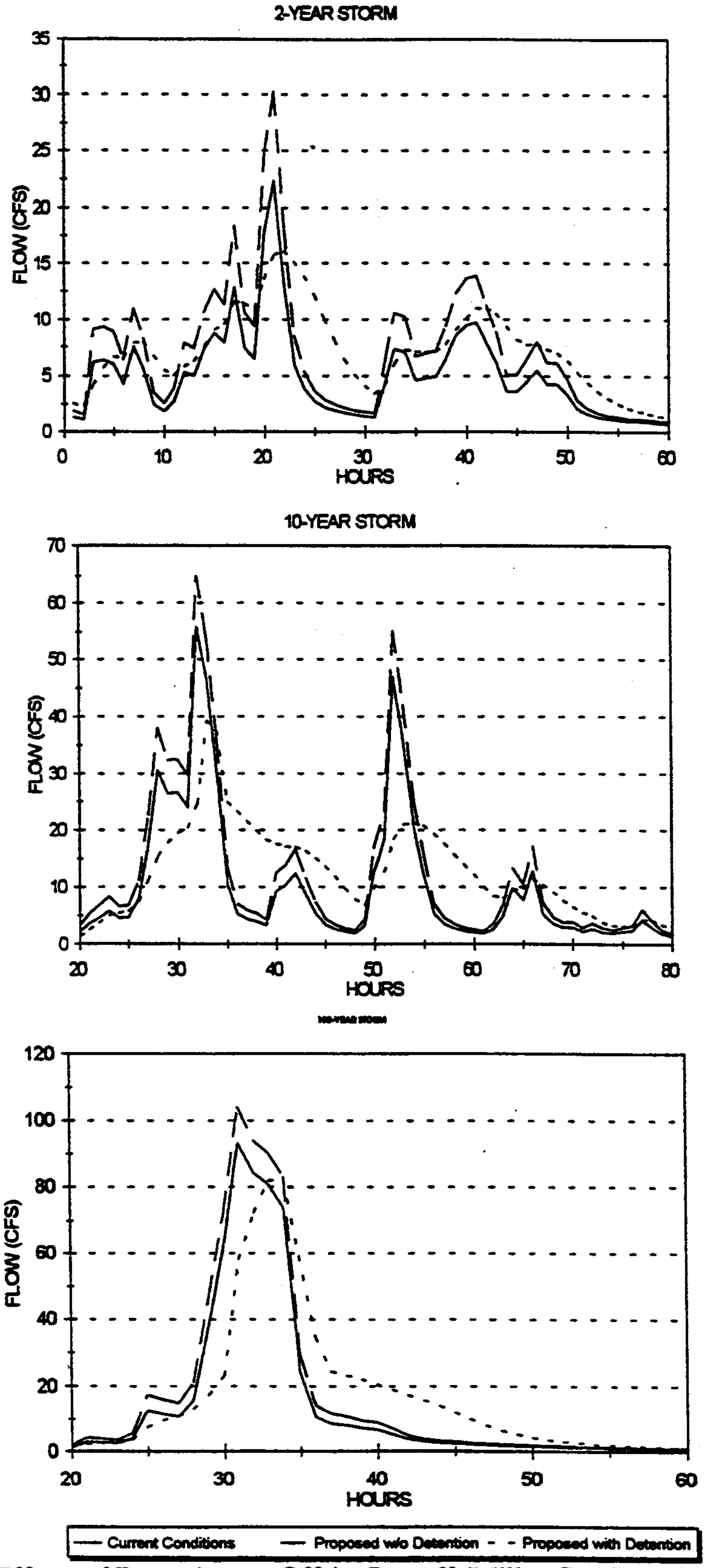

Effects of Detention on Offsite Runoff (Miller Creek Node 24)

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AIRPORT COMMUNITIES COALITION, Appellant, v. STATE OF WASHINGTON DEPARTMENT OF ECOLOGY, and THE PORT OF SEATTLE, Respondents.	No. PCHB 01-160 CERTIFICATE OF SERVICE

Holly Simmelink, Certified PLS, certifies that, on February 25, 2002, I filed/served the following documents on the following persons by the means specified below:

1. Port Of Seattle's Reply Memorandum Supporting Motion For Partial Summary Judgment On SEPA Issue;

2. Declaration of Roger A. Pearce Supporting Port of Seattle's Motion for Partial Summary Judgment on SEPA Issue; and

2. this Certificate of Service

Joan M. Marchioro
Department of Ecology
2425 Bristol Court S.W., 2nd Floor
Olympia, Washington 98502
By Messenger

ORIGINAL

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I declare under penalty of perjury under the laws of the state of Washington that the foregoing is true and correct.

Executed this 25th day of February 2002, at Seattle Washington.

~~Holly~~ Simmelink, Certified PLS

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