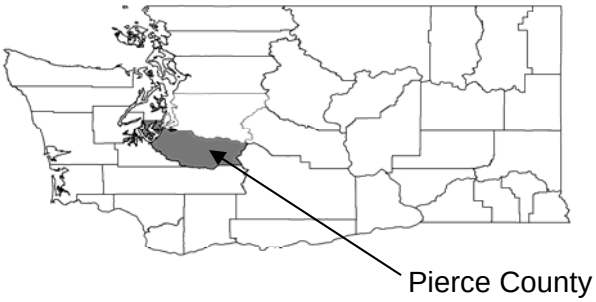
# FLOOD INSURANCE STUDY

# PIERCE COUNTY, WASHINGTON

## AND INCORPORATED AREAS

Volume 1 of 5




| COMMUNITY NAME | COMMUNITY NUMBER |
|---|---|
| BONNEY LAKE, CITY OF | 530274 |
| BUCKLEY, CITY OF | 530139 |
| CARBONADO, TOWN OF* | 530243 |
| DUPONT, CITY OF | 530245 |
| EATONVILLE, TOWN OF | 530283 |
| EDGEWOOD, CITY OF | 530328 |
| FIFE, CITY OF | 530140 |
| FIRCREST, CITY OF | 530141 |
| GIG HARBOR, CITY OF | 530142 |
| LAKEWOOD, CITY OF | 530333 |
| MILTON, CITY OF | 530294 |
| ORTING, CITY OF | 530143 |
| PIERCE COUNTY, (UNINCORPORATED AREAS) | 530138 |
| PUYALLUP, CITY OF | 530144 |
| ROY, CITY OF | 530262 |
| RUSTON, TOWN OF | 530300 |
| SOUTH PRAIRIE, TOWN OF | 530145 |
| STEILACOOM, TOWN OF | 530146 |
| SUMNER, CITY OF | 530147 |
| TACOMA, CITY OF | 530148 |
| UNIVERSITY PLACE, CITY OF | 530332 |
| WILKESON, TOWN OF | 530268 |

*No Special Flood Hazard Areas identified

**REVISED PRELIMINARY**

**December 19, 2014**

**Effective: TBD**

## Federal Emergency Management Agency

**FLOOD INSURANCE STUDY NUMBER**
**53053CV001A**

## NOTICE TO
## FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) may not contain all data available within the repository. It is advisable to contact the community repository for any additional data.

Selected Flood Insurance Rate Map panels for the community contain information that was previously shown separately on the corresponding Flood Boundary and Floodway Map panels (e.g., floodways, cross sections). In addition, former flood hazard zone designations have been changed as follows:

| Old Zone | New Zone |
|---|---|
| A1 through A30 | AE |
| V1 through V30 | VE |
| B | X |
| C | X |

Part or all of this FIS may be revised and republished at any time. In addition, part of this FIS may be revised by a Letter of Map Revision process, which does not involve republication or redistribution of the FIS. It is, therefore, the responsibility of the user to consult with community officials and to check the community repository to obtain the most current FIS report components.

**ATTENTION:** On FIRM panels 53053C0166E, 53053C0167E, 53053C0168E, 53053C0169E, 53053C0188E, 53053C0306E, 53053C0307E, 53053C0326E, 53053C0327E, 53053C0328E, 53053C0329E, 53053C0333E, 53053C0363E, 53053C0601E, 53053C0602E, 53053C0603E, 53053C0604E, 53053C0606E, 53053C0607E, 53053C0608E, 53053C0609E, 53053C0612E, and 53053C0626E, the Carbon River and Puyallup River levees have not been demonstrated by the community or levee owner(s) to meet the requirements of Section 65.10 of the NFIP regulations in 44 CFR as it relates to the levee's capacity to provide 1%-percent-annual-chance flood protection. The subject areas are identified on FIRM panels (with notes and bounding lines) and in the FIS report as potential areas of flood hazard data changes based on further review.

FEMA has updated the levee analysis and mapping procedures for non-accredited levees. Until such time as FEMA is able to initiate a new flood risk project to apply the new procedures, the flood hazard information on the aforementioned FIRM panels that are affected by the Carbon River and Puyallup River levees are being added as a snapshot of the prior previously effective information presented on the FIRMs and FIS reports dated 8/19/1987, 5/1980, 9/27/1985, and 6/1/1983. As indicated above, it is expected that affected flood hazard data within the subject area could be significantly revised. This may result in floodplain boundary changes, 1%-percent- annual -chance flood elevation changes, and/or changes to flood hazard zone designations.

The effective FIRM panels (and the FIS report) will again be revised at a later date to update the flood hazard information associated with the Carbon River and Puyallup River levees when FEMA is able to initiate and complete a new flood risk project to apply the new levee analysis and mapping procedures.

Initial Countywide FIS Effective Date: To Be Determined

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**PUBLISHED SEPARATELY**


Flood Insurance Rate Map Index
Flood Insurance Rate Map

# FLOOD INSURANCE STUDY

# PIERCE COUNTY, WASHINGTON
# AND INCORPORATED AREAS

## 1.0 INTRODUCTION

### 1.1 Purpose of Study

This Flood Insurance Study (FIS) revises and updates information on the existence and severity of flood hazards in the geographic area of Pierce County, including the Cities of Bonney Lake, Buckley, Dupont, Edgewood, Fife, Fircrest, Gig Harbor, Lakewood, Milton, Orting, Puyallup, Roy, Sumner, Tacoma, and University Place; the Towns of Carbonado, Eatonville, Ruston, South Prairie, Steilacoom, and Wilkeson; and the unincorporated areas of Pierce County (referred to collectively herein as Pierce County), and aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This study has developed flood-risk data for various areas of the community that will be used to establish actuarial flood insurance rates and to assist the community in its efforts to promote sound floodplain management. Minimum floodplain management requirements for participation in the National Flood Insurance Program (NFIP) are set forth in the Code of Federal Regulations at 44 CFR, 60.3.

In some states or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirements. In such cases, the more restrictive criteria take precedence and the State (or other jurisdictional agency) will be able to explain them.

Please note that the Cities of Auburn, Milton and Pacific are geographically located in King and Pierce Counties. The City of Milton is included in its entirety in this FIS report. The flood-hazard information for the Cities of Auburn and Pacific are included in this FIS for informational purposes only. See the separately published FIS report and Flood Insurance Rate Map (FIRM) for King County.

Please note that the Town of Carbonado has no special flood hazard areas identified. This does not preclude future determinations of SFHAs that could be necessitated by changed conditions affecting the community (i.e., annexation of new lands) or the availability of new scientific or technical data about flood hazards.

Please also note that FEMA has identified one or more levees in this jurisdiction that have not been demonstrated by the community or levee owner(s) to meet the requirements of 44 CFR Part Section 65.10 of the NFIP regulations (44CFR65.10) as it relates to the levee's capacity to provide 1-percent-annual-chance flood protection. As such, there are temporary actions are being taken until such time as FEMA is able to

initiate a new flood risk project to apply the new levee analysis and mapping procedures. Please refer to the Notice to Flood Insurance Study Users page at the front of this FIS report for more information.

## 1.2 Authority and Acknowledgments

The sources of authority for this FIS report are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

**Pre-Countywide**

Information on the authority and acknowledgements for each jurisdiction included in this FIS, as compiled from previously printed pre-countywide FIS reports is shown below.

The hydrologic and hydraulic analyses for Pierce County (Unincorporated Areas) were performed by the U.S. Geological Survey (USGS), for the Federal Emergency Management Agency (FEMA), under Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 4. This study was completed in October 1980.

In addition to the work done by the USGS, Dames & Moore revised the hydraulic analyses for the Puyallup and Carbon Rivers using more up-to-date information provided by the U.S. Army Corps of Engineers (USACE). This work was performed under Contract No. C-0542. This study was completed in December 1983.

Hydrologic and hydraulic analyses for the communities of Pierce County were performed by study contractors and are summarized below.

| Community | Contractor | Contract Number | Completion Date |
|---|---|---|---|
| City of Bonney Lake | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76 | August 1978 |
| City of Buckley | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | September 1978 |
| City of Dupont | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | July 1980 |
| Town of Eatonville | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76 Project Order No. 4 | October 1980 |
| City of Fife | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | June 1979 |
| City of Fircrest | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | December 1979 |
| City of Gig Harbor | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | May 1979 |

| Community | Contractor | Contract Number | Completion Date |
|---|---|---|---|
| City of Milton | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | October 1979 |
| City of Orting | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | December 1980 |
| City of Puyallup | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | January 1979 |
| City of Roy | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | August 1980 |
| Town of Ruston | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76 | January 1981 |
| Town of South Prairie | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | April 1980 |
| Town of Steilacoom | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | August 1980 |
| City of Sumner | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | May 1979 |
| City of Tacoma | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76 | January 1981 |
| Town of Wilkeson | USGS for FEMA | Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 12 | January 1980 |

FIS reports were not published for the Town of Carbonado, nor the Cities of Edgewood, Lakewood, and University Place; therefore, the authorities and acknowledgements for those communities are not available.

**Initial Countywide (TBD)**

For the initial countywide FIS, the revised hydrologic and hydraulic analyses for the following streams in Pierce County were prepared by Northwest Hydraulic Consultants, Inc. (nhc) under Contract Number EMS-2001-CO-0067. This work was completed between October 2002 and July 2006.

| Stream | Task Order Number | Completion Date |
|---|---|---|
| Artondale Creek | TO003 | January 2005 |
| Carbon River | TO002 | June 2006 |
| Clarks Creek | TO004 | June 2005 |
| Clear Creek | TO002 | December 2002 |
| Clover Creek | TO002 | June 2003 |
| Crescent Creek | TO003 | July 2006 |
| Fennel Creek | TO002 | December 2004 |

| Stream | Task Order Number | Completion Date |
|---|---|---|
| Lacamas Creek | TO002 | December 2002 |
| Mashel River | TO002 | October 2002 |
| Morey Creek | TO004 | April 2005 |
| North Fork of Clover Creek | TO002 | June 2003 |
| Puyallup River | TO002 | March 2004 |
| South Prairie Creek | TO002 | July 2005 |
| Spanaway Creek | TO002 | October 2002 |
| Swan Creek | TO004 | March 2005 |
| Wapato Creeks I & II | TO002 | August 2003 |
| White River | TO002 | June 2005 |

The hydrologic and hydraulic analyses for Canyon Creek, Diru Creek, East Fork of Clear Creek, North Fork of Clover Creek Tributaries, Rody Creek and Woodland Creek were performed by Tetra Tech / KCM (Tt/KCM). The contract number is not available. This work was completed between mid-2004 and mid-2007.

The coastal analyses for this initial countywide FIS were performed by the Strategic Alliance for Risk Reduction (STARR) for FEMA, under Contract No. HSFEHQ-09-D-0370, Task Order No. HSFE10-11-J-0085. The work was completed in November 2014.

The projection used in the preparation of this map is Universal Transverse Mercator (UTM) Zone 10 North, and the horizontal datum used is North American Datum of 1983 (NAD83).

## 1.3 Coordination

### Pre-Countywide

Streams requiring detailed and approximate study for the original Pierce County Unincorporated Areas Flood Insurance Study were identified at meetings attended by representatives of the study contractor, the Washington State Department of Ecology, FEMA, and Pierce County. These meeting were held in Tacoma, Washington, on January 14 and May 19, 1975. The final community coordination meeting, attended by the above-mentioned parties, was held on June 18, 1985. All problems raised at that meeting have been resolved. Coordination with Pierce County was maintained throughout the study.

The initial Consultation Coordination Officer (CCO) meeting for the City of Bonney Lake study was held on November 6, 1975 for the purpose of identifying flood problems that should be included in the study. Because the City of Bonney Lake did not have a representative in attendance, the study contractor was directed by a FEMA representative in attendance to study the areas of the community that were delineated on a preliminary

Flood Hazard Boundary Map (FHBM). No additional flood problems were identified in the community.

A final CCO meeting was held on December 12, 1978, attended by representatives of the study contractor, FEMA, and the community. No corrections to the study resulted from this meeting.

The initial CCO meeting for the City of Buckley study was held on November 6, 1975, for the purpose of identifying flood problems that should be included in the study. Because the City of Buckley did not have a representative in attendance, the study contractor was directed by a FEMA representative in attendance to study the areas of the community that were delineated on a preliminary FHBM (Reference 1). No additional flood problems were identified in the community.

A final CCO meeting was held on January 9, 1979, attended by representatives of the study contractor, FEMA, and the community. All problems raised at that meeting have been addressed in this study.

The initial CCO meeting for the City of Dupont study was held on November 7, 1975, for the purpose of identifying flood problems that should be included in the study. Because the City of Dupont did not have a representative in attendance, the flood sources to be studied by approximate and detailed methods were selected by the study contractor and agreed upon by FEMA. Subsequent hydrologic analysis of Sequalitchew Creek and discussions with community officials and Fort Lewis Military Reservation personnel indicated the need to revise the scope and methods of study. The necessary revisions were proposed to and authorized by FEMA in June 1979.

A final CCO meeting was held on September 9, 1981, attended by representatives of the study contractor, FEMA, and the community. All problems raised at that meeting have been addressed in this study.

On February 14, 1985, Dames & Moore was instructed by FEMA to proceed with an existing data study for the Town of Eatonville, Washington, using the detailed-study data for the Mashel River from the Pierce County, Washington, FIS (Reference 2).

The final CCO meeting was held on June 26, 1985, and was attended by representatives of FEMA, the study contractor, and the town. No problems were raised at the meeting.

An initial CCO meeting for the City of Fife was held on November 7, 1975 to identify areas requiring detailed or approximate study.

Results of the hydrologic and hydraulic analyses were coordinated with Consoer, Townsend and Associates.

The results of the study were reviewed at the final CCO meeting held on November 13, 1979, and attended by representatives of FEMA, the study contractor, and the city. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the City of Fircrest was held on November 7, 1975 to identify areas requiring detailed or approximate study.

During the course of the study, the extent of past flooding along Leach Creek was documented by discussions with the Fircrest Public Works Director and local residents.

The results of the study were reviewed at the final CCO meeting held on May 27, 1980, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the City of Gig Harbor was held on November 7, 1975, for the purpose of identifying the flooding sources to be studied in detail. This meeting was attended by representatives of FEMA and the study contractor.

The results of the study were reviewed at the final CCO meeting held on June 2, 1980, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the City of Milton was held on November 7, 1975, for the purpose of identifying the flooding sources to be studied in detail. This meeting was attended by representatives of FEMA and the study contractor. A representative from the community did not attend.

Results of the hydrologic and hydraulic analyses for Hylebos Creek and Hylebos Creek Tributary were coordinated with Consoer, Townsend and Associates, a consulting engineering firm that investigated flooding along these creeks in 1974. The extent of past flooding along Hylebos Creek was documented by photographs supplied by the town and discussions with local residents.

The results of the study were reviewed at the final CCO meeting held on March 12, 1981, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the City of Orting was held on November 17, 1975, for the purpose of identifying the flooding sources to be studied in detail. This meeting was attended by representatives of FEMA, the community, and the study contractor.

An intermediate coordination meeting was held on May 7, 1980, to discuss the preliminary results of the study and its effect upon the community. Representatives of the USGS, FEMA, and the city attended the meeting.

During the course of the study the extent of previous flooding along the Puyallup and Carbon Rivers was documented by discussions with local residents and city officials.

On September 3, 1981, Dames & Moore was instructed by FEMA to revise the hydraulic analysis on the Puyallup and Carbon Rivers based on new cross section data available from the USACE.

An initial CCO meeting for the City of Puyallup was held on November 1975. Riverine and shallow flooding areas requiring detailed study were identified at this meeting. At a subsequent meeting between the study contractor and the City of Puyallup in October 1978, the boundaries and severity of shallow flooding areas were delineated and verified. Results of the hydrologic analyses were coordinated with the USACE.

The results of the study were reviewed at the final CCO meeting held on August 6, 1979, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the City of Roy was held on November 7, 1975, for the purpose of identifying the flooding sources to be studied in detail. This meeting was attended by representatives of FEMA and the study contractor.

During the course of the study, the extent of past flooding along Muck Creek was documented by discussions with and photographs from local residents.

The results of the study were reviewed at the final CCO meeting held on April 13, 1981, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

On February 14, 1985, Dames & Moore was instructed by FEMA to proceed with an existing data study for the Town of Ruston, Washington, using the detailed-study data for Puget Sound (Commencement Bay) from the FIS for the City of Tacoma, Washington (Reference 3).

An initial CCO meeting for the Town of Ruston was held on October 3, 1975, along with representatives of FEMA and the City of Tacoma to identify flooding sources requiring detailed or approximate study.

The results of the study were reviewed at the final CCO meeting held on June 26, 1985, and attended by representatives of FEMA, the study

contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the Town of South Prairie was held on January 14, 1975, and was attended by representatives of the USGS, FEMA, the Washington State Department of Ecology, and Pierce County.

During the course of the study, the extent of past flooding along South Prairie Creek was documented by discussion with local residents.

The results of the study were reviewed at the final CCO meeting held on February 3, 1981, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the Town of Steilacoom was held on November 7, 1975, to identify flood sources requiring detailed or approximate study.

Information provided by the National Oceanic and Atmospheric Administration was used in the analysis of tidal flooding elevations.

The results of the study were reviewed at the final CCO meeting held on September 2, 1981, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the City of Sumner was held in November 1975. Streams requiring detailed study were identified at the meeting.

An intermediate meeting was held on January 9, 1979, at the Sumner City Hall to discuss the preliminary results of the study and its effects upon the city. Representatives of the study contractor, FEMA, and the City of Sumner attended the meeting. A few minor changes were made as a result of the meeting.

During the course of work by the study contractor, Pierce County officials provided information useful to the study.

The results of the study were reviewed at the final CCO meeting held on December 17, 1979, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting for the City of Tacoma was held on October 3, 1975, and was attended by representatives of FEMA, the study contractor, and the City of Tacoma.

The hydrologic analyses for Puyallup River were coordinated with the USACE.

An intermediate coordination meeting, held on November 25, 1981, was attended by representatives of the city, the study contractor, and FEMA. The purpose of the meeting was to discuss possible revisions to the scope of study for Flett Creek due to the construction of a storm drainage system. The 6-foot diameter interceptor pipe contains 1- and 0.2-percent-annual-chance flooding, flood boundaries are shown around the holding basins, and flood boundaries were determined for Flett Creek upstream of the interceptor pipe inlet. These revisions were acceptable to the community.

The results of the study were reviewed at the final CCO meeting held on December 12, 1982, and attended by representatives of FEMA, the study contractor, and the community. All problems raised at that meeting have been addressed in this study.

An initial CCO meeting was held on November 6, 1975, and was attended by representatives of the USGS and FEMA. The Town of Wilkeson was invited to participate in the meeting, but did not have any representatives in attendance. During the course of the study, the extent of past flooding along Wilkeson Creek was documented by discussions with local residents.

**Initial Countywide (TBD)**

For this initial countywide study, the following coordination occurred on a stream-wide basis.

For Carbon River, Clear Creek, Clover Creek, Fennel Creek, Lacamas Creek, Mashel River, North Fork of Clover Creek, Puyallup River, South Prairie Creek, Spanaway Creek, Swan Creek, Wapato Creek I & II, and White River, the initial Study Time and Cost Meeting was held on September 19, 2001, and attended by representatives of FEMA, Pierce County, and nhc.

For Artondale Creek, Clarks Creek, Crescent Creek, Morey Creek, the initial Study Time and Cost Meeting was held in July 2003 and was attended by representatives of FEMA, Pierce County, and nhc.

For Canyon Creek, Diru Creek, East Fork of Clear Creek, North Fork of Clover Creek Tributaries, Rody Creek, and Woodland Creek, a startup meeting for this study, held on April 6, 2004, was attended by representatives of FEMA, Michael Baker Jr. Inc., Black and Veatch, Northwest Hydraulic Consultants, Pierce County, and Tt/KCM.

For Puget Sound, an initial meeting was held on February 20, 2013, and attended by representatives of FEMA, STARR and Pierce County.

To assure that user is aware of all revisions to this FIS, it is advisable to contact the community repository of flood-hazard data located at the

Pierce County Public Works and Utilities Department, 2401 South 35th Street, Tacoma, Washington, 98409.

## 2.0 AREA STUDIED

### 2.1 Scope of Study

This FIS report covers the geographic area of Pierce County, Washington, including the incorporated communities in Section 1.1.

**Pre-Countywide**

Streams studied by detailed methods and published in the August 19, 1987 effective FIS report for the Unincorporated Areas of Pierce County are as follows:

**Bonney Lake Outflow** – 0.8 mile within Pierce County

**Carbon River** – from the confluence of the Puyallup River upstream for approximately 8.4 miles

**Chambers Creek** – 4 miles upstream from the outlet of Steilacoom Lake

**Clear Creek** – from the confluence with the Puyallup River upstream for 3.8 miles

**Clover Creek** – from the mouth at lake Steilacoom upstream to the community of Frederickson excluding the portion of the stream that flows through McChord Air Force Base for 12.8 miles

**Debra Jane Creek** – 0.9 miles from the mouth to the Bonney Lake corporate limits

**Fennel Creek** –from the Puyallup River confluence upstream to the Kelly Lake Road for 5.3 miles

**Greenwater River** – 3.8 miles from the mouth of the river

**Horn Creek** – 3.7 miles from the mouth

**Hylebos Creek** – 2.9 miles from the corporate limits of the City of Tacoma and 2.9 miles upstream from mouth for Hylebos Creek Tributary

**Lacamas Creek** – 8.3 miles from the corporate limits of the City of Roy

**Leach Creek** – 2.5 miles from the mouth

**Little Mashel River** – 2.2 miles from the confluence with the Mashel River

**Mashel River** – This study covers the river reach from the confluence with Little Mashel River upstream for 2.2 miles

**Muck Creek** – from U.S. Highway 7 upstream for 4.4 miles, and from 0.4 mile downstream to 0.1 mile upstream of the Town of Roy

**Nisqually River** – from the mouth upstream to the confluence with Ohop Creek and from the Snoqualmie National Forest boundary to the Mt. Rainier National Park boundary for 38.3 miles

**North Creek** – from the confluence at Gig Harbor upstream to the City of Gig Harbor corporate limits, approximately 0.3 miles.

**North Fork Clover Creek** – from the mouth upstream for 1.0 mile and 0.4 mile upstream along North Fork Clover Creek Tributary No. 1

**Ohop Creek** – 10.3 miles from the mouth upstream to Kapowsin Lake

**Puyallup River** – from the corporate limits of the City of Tacoma upstream for 30.0 miles to the community of Electron

**Salmon Creek** – 1.5 miles from the mouth

**South Creek** – from 1 mile downstream of U.S. Highway 7 upstream to the Kapowsin Highway crossing (6.4 miles) and 6.6 miles of streams tributary to South Creek

**South Prairie Creek** – from the Carbon River confluence upstream to the eastern corporate limits of the Town of South Prairie for 6.2 miles

**Spanaway Creek** – from the Spanaway Loop road upstream to Spanaway Lake for 2.1 miles

**Squally Creek** – 1.1 miles from the mouth

**Swan Creek** – 6.0 miles from the mouth

**Tanwax Creek** – 3.8 miles from 0.5 mile upstream of the Eastonville Cutoff Road to Tanwax Lake

**Unnamed Creek** – between Kreger and Silver Lakes for 1.5 miles

**Wapato Creek I** – from the mouth at Blair Waterway upstream to the Union Pacific Railroad near Cedarhurst Road East for 8.1 miles

**Wapato Creek II** – from the diversion structure at Valley and Meridian intersection to 1.5 miles upstream

**White River** – from the mouth upstream for 5.4 miles, 0.4 mile near the City of Buckley and from the confluence with the Greenwater River upstream for 4.3 miles

**Wilkeson Creek** – 1.4 miles from State Route 165

The lakes studied in detail were: American, Clear, Gravelly, Harts, Kapowsin, Kreger, Little, Louise, Ohop, Rapjohn, Silver, Snake, Spanaway, Steilacoom, Tanwax, and Wapato.

Also studied in detail were the tidal areas of Puget Sound at Gig Harbor and Nisqually Reach, Day Island, and Commencement Bay.

The areas studied by detailed methods were selected with priority given to all known flood hazard areas and areas of projected development. Flett Creek was originally studied by detailed methods from the mouth upstream to the corporate limits of the City of Tacoma. After this analysis was completed, the drainage basin was altered at Tacoma. The effect of this change on the downstream portion of Flett Creek was not determined; therefore, the area in Pierce County was changed to an approximate designation.

The approximate method of analysis was used to study parts of Clarks Creek, Flett Creek, Horse Haven Creek, Mashel River, Morey Creek, Muck Creek, Nisqually River, South Creek, Tanwax Creek, Unnamed Tributary to North Creek, Wilkeson Creek, and Voight Creek.

Lake Tapps and areas of shallow flooding caused by ponding were studied by approximate methods. Floods were not studied in detail on Lake Tapps because the maximum water level of that lake is controlled.

Approximate analyses were used to study those areas having a low development potential or minimal flood hazards. The scope and methods of study were proposed to, and agreed upon by, FEMA and Pierce County.

**Initial Countywide (TBD)**

New detailed study streams and streams that were re-studied by detailed methods for this initial countywide FIS are as follows:

***Artondale Creek*** – This study covers Artondale Creek from Wollochet Bay to 0.8 miles upstream at the confluence with West Branch Tributary. The West tributary from the confluence is 0.58 miles and East Tributary from the confluence is 0.91 miles.

***Canyon Creek*** – The detail study reach begins at about 72nd Street East and extends 3.9 miles to 128th Street East. It also includes the West Fork. The main stem of Canyon Creek parallels Canyon Road and joins

with the West Fork near the 90th Street East crossing. The West Fork originates south of 118th Street East and parallels 52nd Avenue East.

***Carbon River*** – The study reach extends from the stream's confluence with the Puyallup River upstream 8.4 miles.

***Clarks Creek*** – The detail study reach begins from the confluence with the Puyallup River upstream for 3.8 miles to 15th Avenue SW. The study reach also includes approximately 1 mile of a small tributary, Meeker Ditch, which enters Clarks Creek at RM 3.4.

***Clear Creek*** – The study reach extends from below Pioneer Way to the confluence of the Puyallup River.

***Clover Creek*** – The study reach extends 12.7 miles from Steilacoom Lake upstream to approximately 71st Avenue East.

***Crescent Creek*** – The detail study reach begins from the confluence with Gig Harbor, in Puget Sound, and extends upstream approximately 3.3 miles to Crescent Lake.

***Diru Creek*** – The study reach begins at 84th Street East and extends south to the creek's headwaters at 128th Street East. This reach is approximately 3.0 miles.

***East Fork of Clear Creek*** – The detail study covers from just upstream 72nd Avenue East and extends south approximately 3.5 miles to State Route (SR) 512.

***Fennel Creek*** – The study reach extends from the stream's confluence with the Puyallup River upstream 5.4 miles to the Kelly Lake Road Bridge,

***Lacamas Creek*** – The study reach begins at the creek's confluence with Muck Lake, and extends upstream approximately 8.2 miles to a location southeast of the intersection of 8th Avenue South and 304th Street South.

***Mashel River*** –The study reach begins just below the stream's confluence with the Little Mashel River, and extends upstream approximately 2.24 miles to just below a railroad bridge.

***Meeker Ditch*** – See Clarks Creek description.

***Morey Creek*** – The study reach extends from the stream's confluence with Clover Creek upstream approximately one mile to a divergence from Spanaway Creek.

***North Fork of Clover Creek*** – The study reach extends 1.3 miles from its confluence with the main stem of Clover Creek upstream to Brookdale Road.

***North Fork of Clover Creek Tributaries*** – The downstream boundary of the study reach is at Brookdale Road. The downstream area includes North Fork Tributaries 1, 2, 4, and 5. The study area extends north to 96th Street East and east of Bingham Road.

***Puyallup River*** – The study reach extends from Commencement Bay in Tacoma upstream approximately 30 miles to Electron, Washington.

***Rody Creek*** – The study reach is in two segments. The first begins at the confluence with Clarks Creek and extends south approximately 0.5 miles to the upstream side of Pioneer Way. The second segment starts at the downstream side of the culvert under 84th Street East and continues south for about 3.0 miles to 128th Street East.

***South Prairie Creek*** – The study reach begins at the creek's confluence with the Carbon River east of Orting, and extends upstream approximately 6.2 miles.

***Spanaway Creek*** – The study reach extends from the stream's crossing of Spanaway Loop Road upstream approximately two miles to the outlet from Spanaway Lake.

***Swan Creek*** – The detailed study reach begins at the crossing with 64th Street East at river mile (RM) 2.6 and extends to the headwaters, which originate south of 112th Street East and west of Waller Road.

***Wapato Creek I*** – The detailed study begins at the Blair Waterway, in Puget Sound and extends upstream approximately 8.1 miles to the Union Pacific Railroad crossing near Cedarhusrt Road East (7th Ave NW).

***Wapato Creek II*** – The detailed study begins at a diversion structure entrance, located northeast of the intersection of Valley and Meridian Avenues, and extends upstream for approximately 1.5 miles.

***White River*** – The study reach begins at the confluence with the Puyallup River, and extends upstream approximately 5.5 miles to the Pierce-King county line.

***Woodland Creek*** – The study reach begins at the creek's confluence with Clarks Creek west of Puyallup. The downstream portion of the detailed study reach extends to just south of 80th Street East. At this point the creek enters a relatively deep canyon that is not part of this floodplain study. The creek emerges from the canyon approximately 1,100 feet upstream near 90th Street East. The detailed study resumes at this location and extends south until 104th Street East.

For this initial countywide study, it has been determined that detailed floodplain boundaries reflecting the potential extent of the 1-percent-annual-chance flood for Muck Creek, the Nisqually River and South Creek in Pierce County, have become less accurate considering the migratory nature of the streams. Thus, the detailed areas of these streams have been converted to approximate zones. Please note that profiles of tributaries of these streams have not been updated at the front end to remove backwater effects.

The coastal areas surrounding Pierce County were also revised by STARR for this initial countywide FIS.

This initial countywide study incorporated the determinations of Letter of Map Revisions (LOMRs) issued by FEMA, for the projects listed by community as follows.

**Table 1. Letters of Map Revisions Incorporated for the Initial Countywide Study**

| Community Name | Case Number | Streams | Date |
|---|---|---|---|
| City of Bonney Lake | 94-10-056P | Bonney Lake Outflow, Debra Lane Creek | November 17, 1994 |
| Pierce County, Unincorporated Areas | 05-10-0265P | Clover Creek | June 1, 2005 |
| Town of Steilacoom | 08-10-0544P | Shannon Street Waterway | March 31, 2009 |

## 2.2 Community Description

Pierce County is located in western Washington, near the southern end of the Puget Sound. It is bordered by the counties of Yakima on the east, King and Kitsap on the north, Mason and Thurston on the west, and Lewis on the south. The area of Pierce County is 1,676 square miles. According to the U.S. Census Bureau, the 2013 population estimate of Pierce County is 819,743 (Reference 4). Tacoma, in northwest Pierce County, is the largest of 18 incorporated cities in the county and is the county seat.

The four main rivers in Pierce County are the Nisqually, Puyallup, White, and Carbon. The White and Carbon Rivers are major tributaries to the Puyallup River.

The Nisqually River, which originates from glaciers on Mount Rainier, forms the majority of the western and southern boundaries of Pierce County from Puget Sound to Mt. Rainier. The Puyallup River emerges from glaciers on the western slopes of Mt. Rainier and discharges into Commencement Bay at the City of Tacoma.

The Puyallup River valley south of the City of Orting has a slope of approximately 45 feet per mile, whereas the valley slope at Orting decreases to approximately 25 feet per mile. Decreases in channel slope tend to reduce the ability of a stream or river to transport sediment. Therefore, it is likely that some of the sediment carried by the Puyallup River in the steep valley upstream of Orting is deposited near Orting. The riverbed has been shown to be as much as 5 feet higher than parts of the flood plain outside the channel levees indicating there has been a substantial amount of material deposited over the years.

The White River is a glacier-fed stream that rises on the north-eastern slopes of Mt. Rainier and discharges into the Puyallup River downstream of the City of Sumner at River Mile 10.3.

The topography of Pierce County is diverse, and the range of changes in elevation of land surface is extreme. The terrain changes from flat or moderately hilly around the inlets of Puget Sound in the northwestern part of the county, to rolling upland toward the central part and foothills and mountainous areas in the southeast. The elevation of land around the Puget Sound inlets ranges from near sea level to only a few hundred feet above sea level. The land elevation increases in the southeasterly direction, reaching a maximum of 14,410 feet at the top of Mt. Rainier. The soils of Pierce County were derived from a wide variety of sources, including glacial till and outwash, lake sediment, stream alluvium, and weathered bedrock (Reference 5).

Vegetation consists mainly of evergreen conifers such as Douglas fir, hemlock, and cedar trees. There are lesser stands of deciduous trees and a considerable amount of understory plants (Reference 5).

The climate of Pierce County is predominately marine type, with moist air masses reaching Puget Sound from the Pacific Ocean. The maritime air has a moderating influence in both winter and summer. There is a well-defined dry season in summer and a rainy season in winter (Reference 6). Mean annual precipitation increases from 39 inches at the City of Tacoma in the Puget Sound area to 71 inches at the community of Electron and 204 inches at Paradise Ranger Station on Mt. Rainier. Winter snowfall ranges from 11 inches at Tacoma to 587 inches at Paradise Ranger Station. Seventy-five percent of the precipitation falls from October through March. Mean annual temperatures range from 51°F at Tacoma to 46°F at Electron and 38°F at Paradise Ranger Station. The highest monthly mean temperatures are in July and the lowest are in January, with ranges for Tacoma, Electron, and Paradise of 64°F-39°F, 60°F-34°F, and 53°F-26°F, respectively (Reference 7).

City of Bonney Lake

The City of Bonney Lake is located in north-central Pierce County, in west-central Washington. The city was incorporated in 1949 and had a population of 275 in 1950. The population increased approximately tenfold during the next 20 years, with a population of 2,700 in 1970 (Reference 8). The city is continuing to grow rapidly, as indicated by the 1977 population of 4,380 (Reference 9). According to the U.S. Census Bureau, the 2013 population estimate of Bonney Lake was 18,289 (Reference 4). The city is situated approximately 4 miles east of Sumner, 14 miles east of Tacoma, and 31 miles south of Seattle. The city is located in the western foothills of the Cascade Mountain Range and has a great diversity of topographic relief.

Development consists almost entirely of single-family residences plus a small number of service-oriented businesses. The single-family residences are distributed fairly evenly throughout the city, with the highest density occurring on the lake shores. Most of the private businesses are located along State Route 410 and the portion of Old Buckley Highway adjacent to it.

City of Buckley

The City of Buckley is located at the north-central boundary of Pierce County, in west-central Washington, in the foothills of the Cascade Mountain Range, and is situated adjacent to the White River. Buckley is located on State Route 410. Nearby communities also located on State Route 410 include the Cities of Sumner and Tacoma, which are 11 and 21 miles west, respectively, and the City of Enumclaw, which is 4 miles north.

Buckley was incorporated in 1890 and had a population of 878 at that time. The population remained fairly stable during the next 50 years as evidenced by the 1940 population of 1,170. The city grew substantially during the following 20 years, having a population of 2,705 in 1950 and 3,538 in 1960 (Reference 9). The 1977 population of 3,030 represents a 14 percent decline since 1960 (Reference 8). A significant portion of the population (870 in 1978) resides at Rainer State School, a residential institution for the developmentally disabled. According to the U.S. Census Bureau, the 2013 population estimate of Buckley was 4,453 (Reference 4).

The soils within the high ground portion of town are primarily cemented, gravelly, sandy, clay glacial till, which has a high water-holding capacity and poor drainage characteristics. Vegetation consists of brush and grasses over most of this area, except for the hilly areas in the southern part of town, which are covered with coniferous trees.

Soils on the White River flood plain are composed mainly of coarse-textured alluvium which has a low water-holding capacity and very good drainage characteristics. Vegetation on the flood plains is mainly coniferous trees (Reference 10).

Most of the city is located on high ground with a westward slope of approximately 40 feet per mile. However, the northern portion of the city lies in a deep canyon containing the White River and its adjacent flood plains. The depth of the canyon ranges from approximately 30 feet at the northeast corporate limits to over 100 feet at the northwest corporate limits.

Development in the city has been limited entirely to the area of high ground and consists mainly of single-family residences plus several local businesses. The majority of the residential development and almost all of the business development is concentrated at the west end of the city in the area surrounding State Route 410. Rainier State School is located at the east end of the city.

Town of Carbonado

The Town of Carbonado is located some 30 miles south east of Tacoma in the Carbon River valley. The town of Carbonado was incorporated in 1948. The land area is approximately 0.4 square mile. According to the U.S. Census Bureau, the 2013 population estimate of Carbonado was 613 (Reference 4).

City of Dupont

The City of Dupont is near the western boundary of Pierce County. Dupont, which has an incorporated land area of approximately 5.4 square miles, is located 15 miles southwest of the City of Tacoma. Dupont is surrounded by Puget Sound and unincorporated areas of Pierce County.

Dupont was incorporated in 1951, and the population was 354 at the time of the 1960 census (Reference 8). According to the U.S. Census Bureau, the 2013 population estimate of Dupont was 9,266 (Reference 4).

The soil in the Dupont area is derived mainly from stratified sand and gravel, and locally contains muck, silt, and clay. Throughout most of the community, the soil is characterized by rapid drainage rates and low water-retention capacities (References 10 and 11).

Dupont is primarily a residential community with limited commercial development. Development along the Puget Sound shoreline at Dupont consists of a 0.3-mile-long jetty, located approximately 1 mile south of Sequalitchew Creek, and a dock structure immediately north of the creek.

A potential future development proposed for the northern Dupont shoreline includes a dock and marshaling yard for the shipment of logs and other forest products.

Town of Eatonville

Eatonville is located in southern Pierce County, in western Washington. The town is comprised mostly of residential and commercial development and experiences a relatively stable population. In 1960, 1970, and 1980, the town population was estimated at 896, 852, and 998, respectively (Reference 12). According to the U.S. Census Bureau, the 2013 population estimate of Eatonville was 2,810 (Reference 4).

Eatonville is bordered by unincorporated Pierce County and is situated 30 miles southeast of Tacoma, Washington, 20 miles due west of Mt. Rainer, and slightly more than 1 mile northeast of the Charles Lathrop Pack Demonstration Forest.

The land in Eatonville slopes to the southeast and drains into the Mashel River, which flows westerly through its extreme southern portion. Gravelly clay is the typical soil type within the corporate limits. Deciduous vegetation includes maples and alders. Douglas fir, hemlock, and western red cedar are typical of local evergreen vegetation.

City of Edgewood

City of Edgewood is located about 2 miles east of the City of Tacoma in the northeast part of Pierce County. The city is bordered by the City of Milton and the City of Fife to the west, the City of Puyallup and Pierce County Unincorporated Areas to the south, the City of Sumner and the City of Pacific to the east and King County to the north. The total land area of the city is 8.9 sq. miles. The City of Edgewood was incorporated in 1995. According to the U.S. Census Bureau, the 2013 population estimate of Edgewood was 9,591 (Reference 4).

City of Fife

Fife is near the northern boundary of Pierce County. Fife is bordered by the City of Tacoma to the west and the City of Milton to the east, and the remainder of the city is bordered by unincorporated Pierce County land. Fife is approximately 28 miles south of Seattle.

Fife was incorporated in 1957, and the population was 1,463 when the first census was taken in 1960 (Reference 8). According to the U.S. Census Bureau, the 2013 population estimate of Fife was 9,421 (Reference 4).

Commercial development occurs primarily along State Route 99 and 20th Street East, two east-west arterial highways in northern Fife. Most residential development is located in the northeastern part of the city and

near Wapato Creek in the vicinity of 54th Avenue East and Valley Avenue. Most of the remaining areas of the city have not been developed and are used for agricultural purposes.

Residential development is minimal in areas within the Wapato Creek flood plain. Wapato Creek flows northwesterly through central Fife.

Fife is situated on a generally level flood plain that was formed by the historic periodic flooding of Puyallup River. The resulting predominant type of soil in Fife is known geologically as Recent alluvium. This alluvium, which consists of stratified silt, clay, sand, and gravel deposits, is characterized by its medium water-retention capacity and slow to moderate drainage rate (References 10 and 11).

Native vegetation consists primarily of grasses, shrubs, and deciduous trees.

City of Fircrest

The City of Fircrest is in northwestern Pierce County. It is bordered by the City of Tacoma to the northeast and the unincorporated areas of Pierce County elsewhere.

Fircrest was incorporated in 1925, and had a population of 441 when its first census was taken in 1930. The population at the end of subsequent decades was 486 in 1940, 1,459 in 1950, 3,565 in 1960, and 5,651 in 1970 (Reference 8). According to the U.S. Census Bureau, the 2013 population estimate of Fircrest was 6,613 (Reference 4).

Leach Creek, a tributary to Chambers Creek, is the only stream that flows through Fircrest. The drainage area of Leach Creek is 4.73 square miles, approximately 0.1 mile downstream of the town at 40th Street West. The flood plain along Leach Creek has been heavily developed in Fircrest since 1969, with single-family residences lining both banks.

Only one short stretch of Leach Creek, approximately 0.1 mile long, is within the corporate limits of Fircrest. Channel improvements, which consist mainly of channel clearing and bank protection, have been made along the creek during the course of development.

Leach Creek flows southerly through Fircrest. Ground elevations range from approximately 190 feet along the creekbed to approximately 420 feet west of the creek and 320 northwest, northeast, and east of the creek.

Soil in the Fircrest area consists principally of peat, clay, silt, fine to coarse sand, and gravel deposits derived from gravelly glacial drift and cemented glacial till. In most areas of the community the soil is characterized by its medium water-retention capacity and moderate drainage rates. However, in low areas, such as along Leach Creek, the

soil has a high water-holding capacity and correspondingly slow drainage rates (Reference 10 and 11).

Native vegetation in Fircrest consists of an abundance of coniferous and deciduous trees, shrubs, and grasses.

City of Gig Harbor

The City of Gig Harbor is near the northwestern boundary of Pierce County. Gig Harbor, which has an incorporated land area of approximately 1.0 square mile, is located 4 miles northwest of Tacoma and 28 miles southeast of Bremerton.

Gig Harbor was incorporated in 1946, and had a population of 803 when the first local census was taken in 1950 (Reference 8). According to the U.S. Census Bureau, the 2013 population estimate of Gig Harbor was 7,798 (Reference 4).

Residential development is distributed throughout most of the town. Commercial development is located primarily along the Gig Harbor shoreline. The business offices and warehouses of a public utilities company are located northeast of Harborview Drive at the confluence of North Creek and an unnamed tributary to it. That development included replacement of the natural channels for North Creek, downstream of North Harborview Drive, and the unnamed tributary, downstream of Harborview Drive, with long, underground culverts.

North Creek originates approximately 0.5 mile northwest of Gig Harbor and flows southeasterly to the vicinity of Harborview Drive. The unnamed tributary to North Creek originates less than 0.25 mile south of the corporate limits of Gig Harbor and flows north-easterly to this same general vicinity. Both streams then flow easterly to separate underground culverts at North Harborview Drive. The culverts converge before discharging their flow into Gig Harbor. Gig Harbor is an inlet of Puget Sound.

Land elevations range from approximately 300 feet in the westernmost parts of town to sea level along the eastern corporate limits at the Gig Harbor shoreline of Puget Sound.

Land elevations range from approximately 300 feet in the westernmost parts of town to sea level along the eastern corporate limits at the Gig Harbor shoreline of Puget Sound.

The soil in the Gig Harbor area is composed mainly of cemented glacial till. This till, which consists of silt, clay, sand, and gravel deposits, has medium water retention and drainage characteristics (References 10 and 11).

Native vegetation consists primarily of coniferous and deciduous trees, shrubs, and grasses.

City of Lakewood

The City of Lakewood is located about 9 miles south of the City of Tacoma in southwestern Pierce County. The city is bordered by the Town of Steilacoom to the west and the City of University Place and the City of Tacoma to north, and Pierce County Unincorporated areas to the south and southeast. The total land area of the city is 49.1 sq. miles. The City of Edgewood was incorporated in 1996. According to the U.S. Census Bureau, the 2013 population estimate of Lakewood was 59,097 (Reference 4).

City of Milton

Milton is located along the Pierce and King County boundary, in the southeastern Puget Sound region of western Washington. It has an incorporated area of approximately 1.4 square mile that is 5 miles east of Tacoma and 26 miles south of Seattle.

Milton was incorporated in 1907, and had a population of 448 when the first census was taken in 1910. The population grew slowly for the next three decades, and totaled 671 by 1940. The rate of population growth increased substantially as indicated by population totals of 1,374 in 1950 and 2,218 in 1960 (Reference 8). According to the U.S. Census Bureau, the 2013 population estimate of Milton was 7,239 (Reference 4).

Milton is primarily a residential community with very little commercial development. Most of the development is on the gentle slopes or tops of hills. However, a few homes and commercial buildings are located within the Hylebos Creek floodplain.

Hylebos Creek flows southwesterly through Milton. Hylebos Creek Tributary flows south-southeasterly to its confluence with Hylebos Creek.

The soil in the Milton area is the product of weathering and erosion of material deposited during glacial periods. It consists predominantly of clay, silt, sand, and gravel derived from cemented glacial till. Moderately high water-retention capacity and slow to medium drainage rates characterizes this soil (References 10 and 11).

City of Orting

The City of Orting is located in western Washington in north-central Pierce County. Orting, which has an incorporated land area of approximately 1.6 square miles, is located 19 miles southeast of the City of Tacoma and 8 miles south of the City of Sumner.

Orting was incorporated in 1889, and the population was 623 when the first census was taken in 1890 (Reference 8). The population has grown at a fairly constant rate, with increases during each decade generally ranging between 100 and 200. According to the U.S. Census Bureau, the 2013 population estimate of Orting was 7,023 (Reference 4).

Orting lies near the southern terminus of a broad glacial valley that at one time was an arm of Puget Sound. The valley floor, which is flat and approximately 2 miles wide in the vicinity of Orting, is bounded by steep hillsides that rise several hundred feet above it. Within the Orting corporate limits, the Puyallup River is situated near the center of the valley and the Carbon River flows along the northeast valley edge. Southeast of Orting, the valley separates into two smaller valleys, one containing the Puyallup River and the other the Carbon River.

The Puyallup River emerges from glaciers on the western slopes of Mt. Rainier and discharges into Commencement Bay at Tacoma. The Puyallup River valley south of Orting has a slope of approximately 45 feet per mile while the valley slope at Orting levels off to approximately 25 feet per mile. Decreases in channel slope tend to reduce the ability of a stream or river to transport sediment. Therefore, it is likely that some of the sediment carried by the Puyallup River in the steep valley upstream of Orting is deposited near Orting. The riverbed has been shown to be as much as 5 feet higher than parts of the flood plain outside the channel levees indicating there has been a substantial amount of material deposited over the years.

The Carbon River emerges from Carbon Glacier on the northern slopes of Mt. Rainier, and discharges into the Puyallup River downstream of Orting at River Mile 17.9.

The soil in the Orting area consists principally of clay, silt, and sand, derived from glacial outwash. In most portion of the community the soil is characterized by a moderate water-retention capacity and slow drainage rates (References 10 and 11).

The vegetation along the flood plains in Orting is primarily cottonwoods and evergreen species such as Douglas fir.

Commercial and residential development is located primarily between the Puyallup and Carbon Rivers along State Highway 162. Several homes built alongside the Carbon River are located within what was once a meander of the main channel that was formed prior to the construction of the levee which now runs along the south channel bank. The State Soldiers Home, a home for retired military personnel, is located southwest of the Puyallup River at the western corporate limits of Orting. Many farms are also located along the Puyallup River.

City of Puyallup

The City of Puyallup is situated in northwestern Pierce County, in west-central Washington. It is in the southeastern part of the Puget Sound region, and is surrounded by unincorporated areas of Pierce County. Puyallup is located 8 miles southeast of Tacoma and 29 miles south of Seattle, at the junction of State Highways 410 and 512.

Puyallup was incorporated in 1890. In 1900, the population was 1,884. The population grew at a uniform rate between 1900 and 1960, with increases during each decade ranging from 1,000 to 2,000. The population increased from 12,063 to 14,742 in 1970 (Reference 8). According to the U.S. Census Bureau, the 2013 population estimate of Puyallup was 38,609 (Reference 4).

Residential development is distributed uniformly throughout the city. Commercial development is concentrated near the center of the city and in the north along State Highway 410.

There has been very little development along the flood plain of the Puyallup River. The flood plain along Clarks Creek is used mostly for recreational purposes.

The Burlington Northern Railroad, which passes through the City of Puyallup, is constructed on fill material, creating a drainage divide for storm runoff. Runoff north of the railroad results from local precipitation. However, runoff south of the railroad includes drainage from over 4 square miles of hills in addition to that originating on approximately 2 square miles of flat land adjacent to the railroad. This runoff drains into Clarks Creek near the south-west corporate limits. Clarks Creek, which flows northwesterly, empties into the Puyallup River 2.5 miles downstream of the corporate limits.

Two-thirds of the city is on flat land bounded by the Puyallup River to the north and a hillside to the south. This area has an average westward slope of 10 to 15 feet per mile with ground elevations ranging from 70 feet near the eastern corporate limits to 30 feet at the western corporate limits. The remainder of the city is situated on the hills to the south. This area ranges in elevation from 100 to 400 feet. Most of the land adjacent to the hills is used for agricultural purposes. The principal crops are vegetables and berries.

City of Roy

Roy is situated in the southwestern part of Pierce County. The city, which has an incorporated land area of approximately 0.4 square mile, is located 19 miles south of the City of Tacoma and 24 miles east of the City of Olympia. Roy is bordered by Fort Lewis Military Reservation to the north and west and by unincorporated Pierce County land to the east and south.

Roy was incorporated in 1908, and the population was 315 when the first census was taken in 1910 (Reference 8). According to the U.S. Census Bureau, the 2013 population estimate of Roy was 804 (Reference 4).

Development in Roy is predominantly residential and agricultural. Development along Muck Creek consists of only a few scattered single-family dwellings.

Muck Creek drainage has its headwater on a plateau west of the Puyallup River. Streamflow along Muck Creek varies according to the nature of the channel and overbank areas.

Roy lies in a broad glacial valley which is oriented northeast-southwest. Ground elevations range from 300 feet at Brandenburg and Halverson Marshes in the northwest to 370 feet on Hyland Hill in the southeast.

The soil in the Roy area consists principally of clay, sand, and gravel deposits, derived from glacial outwash. In most portion of the community, the soil is characterized by its low water-retention capacity and rapid drainage rates (References 10 and 11). However, much of the low-lying land in the northwestern part of the city is generally swampy because the water table is usually above the ground elevations in that area.

Vegetation in the vicinity of Roy consists mostly of evergreen forest. However, most of the area within the corporate limits of Roy has been developed, either residentially or agriculturally, and there is little substantial natural ground cover in the city.

Town of Ruston

The Town of Ruston is located at the northern border of Pierce County, in the southeastern part of the Puget Sound region. The City of Tacoma completely borders Ruston, except for the shoreline of Commencement Bay. According to the U.S. Census Bureau, the 2013 population estimate of Ruston was 767 (Reference 4).

The Ruston shoreline and the eastern half of the town contain extensive commercial development. This area is serviced by the Burlington Northern Railroad, whose tracks almost divide the town into northern and southern halves.

Native vegetation has been drastically modified because of urbanization. The soil in the area is derived mainly from cemented glacial till, and locally contains peat, muck, clay, silt, fin to coarse sand, gravel, cobbles, and boulders. Land elevations range from approximately 200 feet at the southern boundary to near sea level at Commencement Bay.

Town of South Prairie

The Town of South Prairie is in the north-central part of Pierce County. South Prairie, which has an incorporated land area of approximately 0.4 square mile, is located 4 miles southwest of the City of Buckley, 8 miles southeast of the City of Sumner, and 20 miles southeast of the City of Tacoma.

South Prairie was incorporated in 1909, and the population was 264 when the first census was taken in 1910 (Reference 8). The population gradually decreased during subsequent decades. In 1979 the population was 197 (Reference 9). The population has started in rise in recent time. According to the U.S. Census Bureau, the 2013 population estimate of South Prairie was 435 (Reference 4).

The soil in the South Prairie area consists principally of clay, silt, sand, and gravel deposits, derived from cemented glacial till. This soil is generally characterized by its moderately high water-retention capacity and corresponding medium to slow drainage rates (Reference 10 and 11).

The South Prairie Creek flood plain is relatively free of development except for a few single-family dwellings.

Town of Steilacoom

The Town of Steilacoom is near the western boundary of Pierce County. It is on Puget Sound, in the southern part of the Puget Sound region. Steilacoom, with an incorporated land area of approximately 2.1 square miles, is located 9 miles southwest of the City of Tacoma.

Steilacoom was incorporated in 1854, and the population was 270 when the first census was taken in 1890. The population increased slowly for several decades; by 1940, the population was only 832. The rate of population growth increased substantially during the following three decades, with the population totaling 1,233 in 1950, 1,569 in 1960, and 2,850 in 1970 (References 8 and 9). According to the U.S. Census Bureau, the 2013 population estimate of Steilacoom was 6,122 (Reference 4).

The soil in the Steilacoom area is composed mainly of glacial till. This till, which consists of compact unstratified clay, sand, and gravel deposits with a few cobbles and boulders locally, has medium water retention and drainage characteristics (References 10 and 11).

Native vegetation consists primarily of grasses, shrubs, and coniferous trees.

Residential development is distributed uniformly throughout most of the town. Commercial development is located primarily in the northwestern

part of town along Commercial, LaFayette, and Rainier Streets. The only development along the Puget Sound shoreline at Steilacoom has been at Gordon Point and Sunnyside Beach, two beach areas along a shoreline that is otherwise bordered by high steep banks. There is a marina at Gordon Point, and the Steilacoom Sewage Treatment Plant is located at Sunnyside Beach. There are also a few part-related facilities and structures at each of these areas. Development along the Chambers Bay shoreline consists solely of a large lumber company.

City of Sumner

Sumner is near the north-central boundary of Pierce County. The city is located 11 miles southeast of Tacoma and 3 miles northeast of Puyallup. It is entirely surrounded by unincorporated areas of Pierce County.

Sumner, incorporated in 1891, had a population of 531 when its first census was taken in 1900. Population increases ranged from 173 to 676 during each decade between 1900 and 1960. The 1960 population of 3,156 increased to 4,325 by 1970 (Reference 8). According to the U.S. Census Bureau, the 2013 population estimate of Sumner was 9,589 (Reference 4).

Residential development is distributed throughout most of the city. Commercial development is located primarily near the center of the city. Development on lower parts of the Puyallup River floodplain consists of a mobile home park and an apartment complex located 0.8 and 0.7 mile, respectively, downstream of the State Highway 162 Bridge and the Sumner Waste Water Treatment Facility, located at the confluence of the Puyallup and White Rivers. There is very little development on lower parts of the White River floodplain, which, at the north end of the city, is wide and primarily used for agricultural purposes.

The Puyallup River flows northwesterly along the southwestern corporate limits. The White River, a tributary of the Puyallup River, flows southwesterly along the northwestern corporate limits. The confluence point of the two rivers is just west of the city.

The elevation of Sumner generally ranges from approximately 600 feet in the easternmost part of the city to approximately 45 feet at the confluence of the Puyallup and White Rivers.

The soil in the Sumner area is composed mainly of recent alluvium. This alluvium, which consists of stratified silt, clay, muck, sand, and gravel deposits, has medium water retention and drainage characteristics (References 10 and 11). Most of this alluvium was deposited by the Puyallup and White Rivers.

Native vegetation consists primarily of grasses, except along the banks of the Puyallup and White Rivers, where there is an abundance of deciduous trees and shrubs.

City of Tacoma

The City of Tacoma is located at the northern Pierce County boundary, in the southeastern part of the Puget Sound region. It is 31 miles south of the City of Seattle and 28 miles northeast of the City of Olympia. The City of Tacoma is bounded by King County and the City of Fife to the northeast, the City of University Place and Puget Sound to the west, and the City of Lakewood to the south. The remainder of Tacoma is bounded by unincorporated areas of Pierce County.

Tacoma was incorporated in 1875, and by 1890 the population was 36,006. The rate of population growth during the following years has varied considerably. Between 1940 and 1950, the population increased by over 34,000 to 143,673 (Reference 8). The 1979 population was 157,800 (Reference 9). According to the U.S. Census Bureau, the 2013 population estimate of Tacoma was 203,446 (Reference 4).

The old flood plains along Puyallup River have a substantial amount of commercial development outside the levees that confine the river channel. Very little development has occurred along the Swan Creek flood plain. The Flett Creek flood plain contains light to moderate amounts of residential and commercial development. Single-family dwellings are located along small portions of the shorelines of both Snake and Wapato Lakes.

The Port of Tacoma, with its numerous associated waterways, contains extensive commercial development. Although only a small amount of commercial development has occurred along Ruston Way on the southwest side of Commencement Bay, this shoreline area is considered to have a high-development potential for both commercial and multi-family residential growth. Single-family dwellings occupy most of the available shoreline areas along the northeast side of Commencement Bay, along Salmon Beach, and on Day Island.

Native vegetation has been drastically modified and replaced by urbanization.

The soil in the Tacoma area is derived mainly from cemented glacial till, and locally contains peat, muck, clay, silt, fine to coarse sand, gravel, cobbles, and boulders. Throughout most of Tacoma, the soil is gravelly sandy loam having average soil drainage and water retention rates. The soil in South Tacoma contains more gravel, pebbles, and cobbles than other areas of the city and has more rapid drainage characteristics. Depressions of low areas, such as portions of Flett Creek, Snake Lake, and the delta flats of northeast Tacoma contain clay, sand, silt, peat, and muck. These areas drain very slowly, and consequently, have a high water holding capacity. Shoreline drainage and water retention rates are variable (Reference 10 and 11).

City of University Place

The City of University Place is located 6 miles east of City of Tacoma in western Pierce County. The city is bordered by the City of Tacoma to north and east and the City of Lakewood to the south and Puget Sound to the west. The total land area of the city is 8.9 sq. miles. According to the U.S. Census Bureau, the 2013 population estimate of University Place was 32,040 (Reference 4). The City of University Place was incorporated in 1995.

Town of Wilkeson

The Town of Wilkeson is in north-central Pierce County. Wilkeson, which has an incorporated land of approximately 0.5 square mile, is 4 miles south of the City of Buckley and 25 miles southeast of the City of Tacoma.

Wilkeson was incorporated in 1909, and the population was 899 when the first census was taken in 1910. According to the U.S. Census Bureau, the 2013 population estimate of Wilkeson was 484 (Reference 4).

The soil in the Wilkeson area consists principally of clay, silt, sand, and gravel deposits, derived from gravelly glacial drift and bedrock. The rate of soil drainage varies considerably throughout town, ranging from rapid in some areas to very slow in others (References 10 and 11).

Wilkeson lies in a narrow valley that is oriented in a northwest-southeast direction. The ground elevations of the valley floor range from approximately 760 feet at the northwest corporate limits to 820 feet at the southeast corporate limits. The hills bordering the valley generally rise between 200 and 300 feet above the valley floor.

The Wilkeson Creek flood plain is relatively free of development except for a few single-family dwellings. Several older homes along the creek banks are situated above the flood plain.

## 2.3 Principal Flood Problems

Major floods occur from October through March as a result of rainstorms, which are often augmented by melting snows.

Major floods on the Puyallup River were recorded 18 times at the City of Puyallup between 1914 and 1943. The largest flood, 57,000 cfs, occurred on December 10, 1933 and caused significant damage. Since beginning operation of Mud Mountain Dam on the White River in 1948, the U.S. Army Corps of Engineers has been able to regulate discharges in the Puyallup River downstream from the City of Sumner. To reduce flooding, they attempt to prevent peak discharges from exceeding 45,000 cfs at USGS Gage 12101500 on the Puyallup River. Since 1943, this has only been exceeded once, on February 8, 1996, when the peak reached

46,700 cfs. (Reference 108) The natural flow of this event would have been 76,000 cfs.

Pierce County has been declared a federal disaster area nine times between 1990 and 2009 due to flooding. In each case, significant flooding occurred along the Puyallup River. The February 1996 storm, which was the largest, was approximately a 60-year event in the reach downstream of the White River confluence and greater than a 100-year event upstream. Significant flooding problems that occurred during the 1996 event are described in the following paragraphs. (Reference 108)

Damage from the 1996 event was extensive throughout the Puyallup River basin, but was particularly destructive upstream of the White River confluence. Beginning at the upstream end of the study reach, significant flood damage first occurred at RM 28.1, where water breached a left bank levee and flowed through a small community of homes. These floodwaters flowed parallel to the river along the landward side of the levee, eventually rejoining the main channel approximately 1.3 miles downstream near the Kapowsin Creek confluence. A second major levee breach occurred at RM 25.1, where water overtopped the right bank levee. This water flowed parallel to the river along the landward side of the levee for approximately five miles before it re-entered the main channel. The Town of Orting was spared significant damage because levees coupled with flood-fighting by residents and municipal officials kept most of the town dry. However, the levees that provided Orting significant protection in 1996 do not meet FEMA requirements for flood protection and therefore, as will be described later, were not considered to provide flood protection in this study. As a result, most of the town will be shown to be within the 100-year floodplain. Upstream of the Carbon River confluence, levee breaches also occurred at RM 20 (left bank), RM 19.3 (right bank), and RM 18.2 (right bank). (Reference 108)

Severe flooding occurred along the reach of the Puyallup River between the Carbon River confluence and the City of Puyallup. Within this reach there are numerous low earthen levees / berms; however, they were too low to prevent flooding. As a result, many homes and communities were flooded, including the following mobile home parks: Del de Rio (RM 14.25, Bowman Hilton (RM 13.25), River Park Mobile Manor (RM 12.5), and Rainier Manor (RM 11). (Reference 108).

Downstream from the city of Puyallup, the river is confined between certified earthen flood control levees which contained most of the flow, although according to county officials, water levels came very close to overtopping the levees in several locations. Near RM 5.7, there is an opening in the left levee where Clarks Creek flows into the Puyallup River. During this flood, water backed up Clarks Creek, flooded homes, and flowed north into the Canyon Creek drainage. (Reference 108)

Flooding reports in the Canyon Creek study area are mostly associated with undersized culverts at roads and driveways. Some of the flooding problems may be maintenance issues. (Reference 95)

According to County officials and local residents, the February 1996 event overtopped both the left and right levees on the Carbon River upstream from the Highway 162 and railroad bridges near South Prairie Creek. The flooding in the left floodplain was bounded by 177th Street East and the abandoned railroad grade; the right bank flooding reportedly flowed north from the river, overtopped State Route 162, and likely flowed into South Prairie Creek. The February 1996 storm was approximately a 50-year event. (Reference 96)

Overbank flooding along the Clarks Creek study reach has been observed. Locations of observed flooding include the intersection of 66th Avenue E. and 56th Street East (RM 0.7); the reach between West Pioneer Avenue and 10th Avenue Southwest (RM 2.6 - 3.4); and in the vicinity of the confluence of Meeker Ditch and Clarks Creek (RM 3.4 - 3.5). Minor flooding of low- laying areas along the Meeker Ditch study reach has also been observed. (Reference 97)

Severe flooding along Clear Creek occurred along the study reach during the February 1996 flood of record. It is believed that this event had a recurrence interval of almost 100 years. (Reference 98)

There are several areas along the main stem of Clover Creek that experience flooding problems. The first area is just downstream from Interstate 5 (RM 1.2). Here, water overtops the right (north) bank and floods low lying residential properties near Cochise Lane Southwest. Most of this water does not re-enter Clover Creek, but rather, it flows north along 58th Avenue toward 112th Street Southwest. Although we could not find anyone that can recall this occurrence, historical records suggest that during very large floods (in excess of the 100-year flood event), water can cross 112th Street SW and flow north to the Lakewood Town Center. Once at the town center, it appears the water will either infiltrate or enter Ponce de Leon Creek, which empties into Lake Steilacoom. A second problem area is just upstream of Interstate 5 (RM 1.7). Here, water will overtop the right bank and flood land occupied by several apartment buildings, parking lots and gravel pits. The third problem area is in the vicinity of Pacific Avenue and A and B Streets (RM 5.8) in Parkland. The February 1996 and January 1997 floods caused significant damage to homes and businesses and closed several roads including Pacific Avenue. The fourth and last major flooding problem is within the Brooksdale Golf Club (RM 7.7). Here, water will overtop the right bank, flow across the golf course, and enter the North Fork. (Reference 99)

Substantial flooding has been observed the central portion of the Crescent Creek study reach. In this region, primarily between cross-sections AL and AS, Crescent Creek has been channelized and flows directly adjacent to Crescent Valley Drive. Overtopping of the main roadway, as well as several driveway crossings has been observed during past events. (Reference 100)

Several reports of flooding have been documented in the Diru Creek study area. According to the draft *Clear/Clarks Creek Basin Plan* (Reference 93), road flooding has been reported along 66th Avenue East in the 9700 and 9800 blocks. In addition, the culverts at 90th Street East, 96th Street East, and 104th Street East may have been overtopped in the past. (Reference 101)

Along Fennel Creek, the only flooding of note in the study reach occurs at the McCutcheon Road bridge. County road crews and local residents report the road having overtopped several times in recent years, most notably in 1996. Road crews estimate McCutcheon Road to have been inundated during the February 1996 storm by approximately 6 inches at the road crown; however, a resident who lives adjacent to the McCutcheon Road bridge estimates the 1996 flooding to have been greater than 1 foot at the road crown. The February 1996 storm was approximately a 1-percent-annual-chance event. (Reference 103)

On Lacamas Creek, the storms that occurred in February 1996 are believed to have delivered flows close in magnitude to the 1-percent-annual-chance event. Several locations of roadway overtopping were identified through anecdotal evidence including: State Route 507, 280th Street South, 56th Avenue South, 20th Avenue South, and 8th Avenue South (including adjacent driveways). (Reference 104)

Morey Creek experienced a significant flood in February 1996. It is reported that this event caused flooding of Morey Creek in the vicinity of 138th Street South. During recent discussions with residents they commented that the field between house 2004 138th Street and 19th Avenue South had flooded during the storm a few years back, presumably they were referring to the 1996 event. The Clover Creek Basin Plan suggests that this flooding may be have been caused by vegetation or debris clogging the channel. Surveys followed a moderate event that occurred on October 20, 2003, during which the low chord of the driveway bridge to 2004 138th Street South was submerged by 0.2 foot. However, even with this there was only minor flooding of the area. Residents also commented that there was significant flooding at 14010 18th Avenue Court in recent years. Significant flooding has also been documented in fields on both sides of Spanaway Loop Road, but the roadway has not recently fully overtopped. In this same area, a new storm water detention facility east of Spanaway Loop Road at 138th Street has been reported to have increased groundwater upwelling and subsequent flooding. This, however, is unrelated to flooding produced by Spanaway Creek and flow does not connect to or from Morey Creek. Also noted during the survey was a minor blockage of the culvert under Perimeter Rd. that was causing roughly a 1-foot drop into the culvert under the roadway. This did not appear to be directly causing any flooding problems upstream. (Reference 106)

Flood damage along the Nisqually River is generally limited to an area near the community of McKenna at River Mile 21.8 (Reference 14) and to the Nisqually Delta, which is a wide 3-mile-long flood plain at the mouth of the river. The land from McKenna to LaGrande Dam has a narrow flood plain with limited access. Approximately 18,000 cubic feet per second (cfs) in the Nisqually River at McKenna is considered to represent the upper limit of zero flood damage (Reference 15). This flow has been exceeded six times during the period of record (1947-78) at the USGS gaging station on the Nisqually River below Powell Creek near McKenna (gage no. 12088400) at River Mile 31.6. The USGS website has no flooding information beyond 1979 for this gage. At this station, the three most severe floods occurred in December 1975 (30,700 cfs), January 1965 (25,700 cfs), and January 1974 (23,200 cfs) (References 16-20). An estimated flood of 42,000 cfs at the same site occurred in December 1933, inundating most of the delta (Reference 21). An estimated peak discharge of 50,000 cfs inundated much of the town of McKenna in 1996.

Flooding is rare in many of the North Fork Tributaries, though residents described two flooding events that occurred on a reach of Tributary 1 upstream of the E-1 regional detention pond, which is south of 128th Street East. Flooding occurred on properties in 1990 and 2000, but little damage was sustained. This is currently an area of low-density residential development. Many homeowners along the creek have hobby farms and graze livestock on pastures by the creek. One homeowner noted that floodwater that used to pond on his property appears to recede much faster following the construction of the E-1 regional detention pond (completed in 1998). (Reference 107)

In February 1996, South Prairie Creek experienced a major flood with an estimated peak discharge of approximately 9,000 cfs at the stream's mouth. This was nearly a 100-year event (A 100-year flood is estimated to have a peak discharge of 9,700 cfs). The 1996 event caused widespread flooding across the valley floor. The highway was damaged in many places as was the highway bridge near RM 3.5. Just upstream from this bridge an earthen berm failed along the right bank and flow left the channel and crossed Spring Site Road. The road was severely damaged. The bridge just downstream was damaged because the river eroded the right bank, exposing the piles of one intermediate pier and the west abutment. (Reference 110)

Spanaway Creek experienced a significant flood in February 1996. This event caused significant flooding in the vicinity of Spanaway Loop Road and 138th Street S. Fields on both sides of 138th Street and Spanaway Loop Road flooded, but neither roadway fully overtopped. (Reference 111)

Extensive flood plains on the White River are located in the Sumner area at the mouth. Flows in the White River are regulated by Mud Mountain Dam, upstream of the study reach at approximately RM 28. The Corps of

Engineers operates this facility to limit flow in the Puyallup River below the White River to 50,000 cubic feet per second (cfs) and, secondarily, to limit dam discharge to 12,000 cfs (COE, 1999; COE, 2004). At these flows, some overbank flooding occurs in the study reach between the county line and 142nd Avenue in Sumner. The affected areas are primarily agricultural fields. Road overtopping is not a concern at typical flood levels." (Reference 114)

A split flow condition occurs on the White River for high flows near the upstream end of the study reach. Flow breaks out over the left (east) bank approximately 1,500 feet upstream of 8th Street East and is conveyed through the floodplain, overtopping the road, before rejoining the main channel roughly 4,000 (river) feet downstream of 8th Street. In larger flood events (i.e. the 1-percent and 0.2-percent-annual-chance floods), overbank flooding results in some additional road overtopping in the floodplain between 8th Street East and the Tacoma Avenue crossing. (Reference 114)

Little information exists as to flooding problems in the Woodland Creek study area. Long-time residents have indicated that very little flooding has taken place over the last 40 years. The lower portion of the basin, where flooding may be more likely, is sparsely populated and has no history of property damage. The only known flooding problems in the area are as follows: from 9300 to 9600 Block of 72nd Avenue East the creek has overtopped numerous driveways and culverts; at 80th Street East the road has been overtopped; and at 102nd Street East the road has been overtopped. (Reference 115)

<u>City of Bonney Lake</u>

The City of Bonney Lake has not experienced any major flooding to date. Debra Jane Creek and Bonney Lake Outflow have drainage areas of 1.26 and 0.46 square miles, respectively, at their mouths. Although the open-channel portions of these streams do not have substantial flooding potential, the presence of road fills with small underlying culverts, which cut across the natural drainage of the streams, creates localized backwater during peak flow events. This situation has not resulted in any flood damage because there has been no development in these areas.

Debra Jane Creek and Bonney Lake Outflow are almost fully contained within the corporate limits. These streams, which originate at lake outlets, flow through gradually sloping wooded areas and then through steep canyons into a valley east of the corporate limits. Fennel Creek is located in this valley. The only portion of Fennel Creek which lies within the corporate limits is the reach from State Route 410 to approximately 300 feet downstream.

Fennel Creek flows through a wide grassy valley and only scattered development exists in its flood plain. Peak flows are generally contained within a well-defined main channel, and concrete-box culverts located at

Old Buckley Highway and State Route 410 do not cause any significant backwater. There has been no development along the portion of the stream that is located within the corporate limits.

Bonney Lake and Debra Jane Lake are completely encompassed by the corporate limits. The elevations of these lakes are maintained by means of outlet structures. Both lake outlets have paved roads above them which act as spillways should the outlet structures become plugged or not be able to dissipate peak inflows quickly enough. On Debra Jane Lake, flow over the road begins when the lake elevation reaches the top of the outlet structure, while on Bonney Lake, road overflow begins when the lake elevation is 0.7 foot above the top of the outlet structure.

Lake Tapps is an artificial lake, created as part of the development of the White River Project in 1912. The lake level is controlled by Puget Sound Power and Light Company for the purpose of electrical generation. The normal full pool elevation is 543 feet (Reference 23).

There are a few small depression- areas scattered throughout the city which occasionally become flooded. Rain falling on these areas normally percolates quickly into the ground, which is composed mainly of glacial till. However, some shallow flooding caused by ponding in these areas has been experienced in the past as a result of heavy rainfall following extreme cold spells which freeze the ground and thus prohibit percolation.

City of Buckley

The White River is confined to a deep canyon as it flows through Buckley. Because only the high-ground area south of the canyon has been developed, the city has never experienced any flood damages caused by riverine flooding. However, the flood plain areas adjacent to the river are inundated by floodwater frequently and, therefore, are not considered suitable for development without the addition of flood protection measures.

The highest known discharge in the White River at Buckley was 28,000 cfs in December 1933 (Reference 24). Local residents, who lived in Buckley during 1933, were contracted concerning the degree of flooding caused by the December 1933 flood. They reported that flooding was confined to the flood plains adjacent to the river. The most significant recent flood was that of November 24, 1986, which had a magnitude of 14,900 cfs (Reference 22).

The City Engineer and several residents were contacted to determine if any of the developed areas of the city had never experienced flooding caused by poor drainage; no such flood areas were identified.

City of Dupont

The principal flood problem affecting Dupont is flooding that occurs along the Puget Sound shoreline as a result of extreme high tides. However, most shoreline areas at Dupont are protected from flooding by high, steep banks, which rise to elevations substantially higher than potential flood elevations. Although a high tide with a 1-percent-annual-chance flood occurred on December 15, 1977, no flood damage is known to have resulted.

Sequalitchew Creek flooding is generally not considered to be a problem for the City of Dupont because floodflows are substantially reduced by diversions upstream of the Dupont corporate limits. Furthermore, the terrain along the creek as it flows through Dupont diminishes the probability of flooding. Between the eastern corporate limits of Dupont and a road crossing near the center of the city, the creek flows at a low gradient through or alongside marshes, lakes, and ponds that are confined within banks and are acknowledged as wetlands having minimal development potential. Downstream of the road crossing, the creek is contained in a steep-sided canyon.

Town of Eatonville

The Mashel River flows through the southern part of Eatonville and constitutes the primary flood hazard.

Extreme storm events cause ponding to occur in western Eatonville near Eatonville Highway and Center Street West, and at low flat areas near the corporate limits and railroad tracks in the northeastern portion of the town.

City of Fife

Although there is some potential for flooding of Wapato Creek, no significant flooding from the creek has been reported by residents of the community. Overflow of Wapato Creek would be caused by intense rainfall.

In 1997, Hylebos Creek flooding closed several lanes of Interstate 5.

The most recent flooding in the vicinity of Fife occurred on Puyallup River in 1933. That flood was estimated to be greater than a 1-percent- annual-chance flood. Levees that were shown providing protection from the 1-percent-annual-chance flood for the City of Fife and surrounding areas have been de-accredited in the revised studies of the Puyallup River allowing flood water to breach levees. See more details in the countywide engineering section of this report.

City of Fircrest

The only major flooding in Fircrest occurs along Leach Creek, usually during November through March, as a result of either winter storms or spring rain.

The highest annual peak discharge recorded for Leach Creek at Fircrest was 417 cfs on October 20, 2003 (Reference 22). That flood is the most recent major flood that Fircrest has experienced.

City of Gig Harbor

Flooding caused by the overflow of North Creek has never been known to occur, however, hydrologic and hydraulic computations made for this study indicate that North Creek between Harborview Drive and North Harborview Drive does have potential for flooding.

Flooding along the Puget Sound shoreline at Gig Harbor occurs occasionally as a result of extremely high tides.

The only reported case of flooding associated with the unnamed tributary to North Creek occurred during a heavy rainstorm on March 5, 1972. The public utilities company located northeast of Harborview Drive experienced flooding when the culvert that runs underneath its property became plugged with debris, forcing considerable flow out of the storm drains and onto its grounds.

The most recent occurrence of shoreline flooding was during the December 15, 1977, high tide, which was considered to be a 1-percent-annual-chance flood. Although several homeowners located along North Harborview Drive experienced minor flooding, only a few of the homes were actually damaged.

City of Milton

The main channel of the downstream reach of Hylebos Creek in Milton is narrow and shallow, and its banks are frequently overtopped by floodwater during heavy rainstorms. The flood plain is generally flat and wide, which allows floodwater to spread over a broad area.

Historically, because development on the Hylebos Creek floodplain has been minimal, only minor damage has resulted from flooding. The most severe flooding in recent years along Hylebos Creek, as reported by several residents of the community, occurred on March 6, 1972. The peak discharge of the 1972 flood on Hylebos Creek is unknown; however, based on peak discharges recorded at gaging stations on nearby streams, that flood had was an estimated 2-percent-annual-chance flood. The floors of a factory and of an adjacent building-supply company near Porter Way were inundated by the 1972 flood. Several homes also experienced minor flooding during that flood.

Hylebos Creek Tributary has a potential flood problem where the culvert and embankment at the Birch Street crossing restrict flood-flow passage. This constriction raises flood elevations, although no flood damage has resulted thus far.

City of Orting

Major floods on the Puyallup and Carbon Rivers usually occur from November through February and results from combinations of warm rain and snowmelt.

Most of the flood damage from the Carbon River occurs in the lower 4-mile reach in the vicinity of Orting. The steep gradient of the river upstream of Orting causes high velocities that erode the streambanks and result in channel changes during high flows. The channel capacity in the Orting area is estimated at 6,000 cfs (Reference 15). In December 1977, 10,000 cfs was recorded at the USGS gaging station at Fairfax (gage no. 12093900) at River Mile 17.7. The Fairfax gage also reported 12,000 cfs in 1996 and 13,000 cfs in 2006.

City of Puyallup

Flooding on the Puyallup River is caused by snowmelt and rainfall. On Clarks Creek, flooding is caused by localized thunderstorms and/or flooding on the Puyallup River. There are flood plains located along some reaches of the river which are subject to flooding. Flooding of these areas generally occurs only from peaks of a 10-percent-annual-chance flood or greater. The maximum flood of record occurred on December 10, 1933, and had a discharge of 57,000 cfs; however, this occurred before the building of Mud Mountain Dam, which now regulates peak floodflows of the lower Puyallup River. The December 2, 1977 flood had a peak discharge of 40,600 cfs in the vicinity of Puyallup, which represents a 6.6-percent-annual-chance flood.

Portions of old Puyallup River channels are located south of State Highway 410, between 7th Street Northwest and 9th Street Northwest, and south of East Main Avenue, between 15th Street Southeast and 23rd Street Southeast. The river no longer flows in these channels due to a combination of physical alterations, including channel realignment and levee construction. However, during periods of high flow in the Puyallup River, water backs into these channels to about the same elevation as that reached by the river.

Historically, minor flooding has frequently occurred along portions of Clarks Creek. Flood heights along Clarks Creek are primarily caused by backwater from the Puyallup River due to the extremely flat gradient of the creek combined with the high stages reached by the Puyallup River.

Shallow flooding of portions of the flat land south of the railroad is a common occurrence. The principal causes of this flooding are:

1) The flatness of the land combined with the high runoff into this area from the hills to the south.

2) The inability of Meeker Ditch to drain the area west of State Highway 512 when Clarks Creek, (the creek it flows into), is at a high stage.

The shallow flooding south of River Road and east of 7th Street is caused by interior drainage.

City of Roy

The only major flooding in Roy occurs on Muck Creek, usually during November through March, as a result of either winter storms or spring rain.

Annual peak-discharge data for Muck Creek at Roy were recorded at a streamflow gaging station (No. 12090200) operated by the U.S. Geological Survey from 1957 to 1976. The gage was located on the north bank of Muck Creek between Warren Street and the Burlington Northern Railroad. The highest annual peak discharge recorded at the gage was 692 cfs on March 7, 1972. The recurrence interval of that peak is estimated to be approximately a 5.9-percent-annual-chance discharge. No flooding resulted from the 1972 peak, according to reports from local residents.

The maximum known flood in Roy occurred in December 1933. Peak discharges and recurrence intervals for this flood are unknown.

The area upstream from the Burlington Northern Railroad Bridge and the Warren Street Bridge is especially prone to flooding due to the constrictions in the flood plain caused by these bridges.

Town of Ruston

Flooding in the Town of Ruston is caused by extreme high tides along the Puget Sound shoreline. This situation is much worse if strong winds occur. The most recent occurrence of shoreline flooding was during the December 15, 1977, high tide, which was considered to be a 1-percent-annual-chance flood. However, the water surface was calm at the time, and consequently only scattered minor flooding was experienced.

Town of South Prairie

Floods on South Prairie Creek usually occur between November and February and result from combinations of warm rain and snowmelt. Local residents reported that the town experienced some major flooding from South Prairie Creek during the 1920s and 1930s. The State Highway 162 Bridge crossing, which was then located approximately 0.3 mile

downstream of its present location, was considered by many to be one of the primary causes of that historic flooding.

Annual peak discharge data for South Prairie Creek at South Prairie are collected at a streamflow gaging station (No. 12095000) operated by the U.S. Geological Survey. The two highest annual peak discharges recorded at the gage since its operation began in 1950 are 8,170 cfs on February 8, 1996, and 6,850 cfs on December 11, 1955.

Town of Steilacoom

The principal flood problem effecting Steilacoom is flooding that occurs along the Puget Sound and Chambers Bay shorelines as a result of extreme high tides. However, except for the beach areas, the shoreline is not subject to flooding because its steep banks are substantially higher than potential flood elevations. Even on the beach areas, the occurrence of an extreme high tide on December 15, 1977, did not cause flood damage. This high tide was a 1-percent-annual-chance flood. The lack of damage from this tide was probably due to the calmness of the water surface at the time.

City of Sumner

Major floods on the Puyallup and White Rivers usually occur between November and February and result from combinations of warm rain and snowmelt.

The highest recorded flow on the Puyallup and White Rivers occurred in December 1933, prior to the construction of Mud Mountain Dam on the White River. A peak discharge of 28,000 cfs was recorded on the White River at Buckley Gaging Station No. 12098500, 28 miles upstream of its mouth (References 18-19 and 24-27). On the Puyallup River, a peak discharge of 57,000 cfs was recorded at Puyallup Gaging Station No. 12101500, 4 miles downstream of its confluence with the White River (References 18-19 and 24-27). There is no record of the peak discharge on the Puyallup River upstream of the confluence during the December 1933 flood. Recurrence intervals were not determined for these peaks because they would not reflect current conditions afforded by the flood control operation of the dam.

The February 9, 1996, flood caused flooding along some of the lower portions of the study area. During that flood, the peak discharge in the Puyallup River above the White River was estimated to be 46,700 cfs.

Flood heights that occur on the White River, which has a relatively flat slope in the vicinity of Sumner, are often influenced by the flooding on the Puyallup River. During the 1977 flood, the White River at its mouth had an estimated peak discharge of 14,600 cfs, which has an estimated average return period of 3 years. Overflow of the riverbank occurred near

the mouth of the river and was caused primarily by backwater from Puyallup River.

City of Tacoma

Flooding from extreme high tides along the Puget Sound shoreline at Tacoma is one of the flood problems affecting the city. The most recent occurrence of shoreline flooding was during the December 15, 1977, high tide, which was considered to be a 1-percent-annual-chance flood. However, the water surface was calm at the time of that high tide and consequently only scattered minor flooding was experienced. Had there been strong winds and waves during the 1977 high tide, substantial flood damage might have occurred.

Flooding along Flett and Swan Creeks is caused by intense rainfall, usually after several days of light but persistent rain, which has saturated the soil. The flat gradient of Flett Creek and the correspondingly slow drainage of its floodwaters have aggravated flooding problems there in the past. The channel of Swan Creek, except near its mouth, is relatively steep and narrow, and flooding is correspondingly most extensive near its mouth. Swan Creek merges with Clear Creek outside the corporate limits. The combined flow then enters Puyallup River at River Mile 2.9. High water on Puyallup River prevents normal drainage and results in backwater covering the low-lying areas in the flood plain behind the dikes of Puyallup River.

Major floods on Puyallup River usually occur between November through February and result from combinations of warm rain and snowmelt. The 1933 flood caused by the overflow of Puyallup River in the vicinity of Tacoma was considered to be a 1-percent-annual-chance flood.

Town of Wilkeson

Floods on Wilkeson Creek usually occur between November and February and results from combinations of warm rain and snowmelt. Local residents reported that no major floods have occurred since the 1920s or 1930s. The discharges of these historic floods could not be determined because the only streamflow records available on Wilkeson Creek are those which were collected during July-October 1949.

## 2.4 Flood Protection Measures

Within this jurisdiction, there are one or more levees that have not been demonstrated by the communities or levee owner(s) to meet the requirements of 44 CFR Part 65.10 of the NFIP regulations as they relates to the levee's capacity to provide 1%-percent-annual-chance flood protection. Please refer to the Notice to Flood Insurance Study Users page at the front of this FIS report for more information.

Limited regulation of floodplain development is provided by the Shoreline Management Program of Pierce County and the Washington State Department of Ecology.

Other than a regional detention pond on Canyon Creek, no significant flood protection measures exist along the creek in the study area. (Reference 95)

There are levees on the Carbon River that are not considering protecting from the 1% annual-chance-flood based on the criteria outlined in Section 65.10 of the NFIP regulations and in Appendix H of FEMA's Guidelines and Specifications for Flood Hazard Mapping Partners (2003). (Reference 96)

Two culverts convey Clear Creek through the Puyallup River levee at the mouth of the study reach. Both culverts have a gate over the outlet to prevent backwater flooding from the Puyallup River. (Reference 98)

At several locations along the main stem of Clover Creek, low earthen berms have been constructed to prevent flooding. However, none provide protection from the 100-year flood. Several storm water detention facilities have been constructed within the basin. The effects of these were considered in the hydrologic evaluation. No major structural flood protection measures that could prevent flooding during a 1-percent-annual-chance event are planned for Clover Creek or the North Fork of Clover Creek. (Reference 99)

Significant capital improvements related to the North Fork of Clover Creek include two regional detention facilities, the 90 acre-foot W-1 regional detention pond on the main stem of North Fork Clover Creek upstream of the confluence of Tributary 4, and the 100 acre-foot E-1 regional detention pond south of 128th Street East. (Reference 99)

City of Bonney Lake

Flood hazards within the City of Bonney Lake are generally of a much localized nature. Therefore, rather than building any flood protection structures, the city has chosen instead to restrict development to only those areas that do not have any significant flood-hazard potential.

City of Buckley

The magnitude of flood discharges in the White River at Buckley is controlled by operation of Mud Mountain Dam, located 4.5 miles upstream of the eastern corporate limits. The capability of the dam to regulate releases independent of flood inflow to the dam can be demonstrated by the following statistics. The available useable storage capacity created by the dam is 106,000 acre-feet. The highest known rate of inflow to the dam since its completion in 1942 occurred during a storm in December 1977. The flood was estimated to be a 1-percent-annual-

chance flood. The peak discharge released from the dam during the storm was equal to the design flood release for a 50-year flood. The maximum contents resulting from the storm were 42,280 acre-feet. A storm of similar intensity and duration, which occurred in January 1965, resulted in a maximum content of 44,130 acre-feet, the highest recorded storage attributable to storm runoff (Reference 16).

The Puget Power Diversion Dam has no effect on the 1- and 0.2-percent-annual-chance flood because it has no storage capacity. The control gates on the White River Canal limit flooding along the canal during periods of flooding in the White River.

Floodplain management measures have not been necessary for the White River flood plains because there has been no interest in developing these areas.

City of Dupont

The flow in Sequalitchew Creek and through its associated upland marshes is controlled by a diversion structure located at the outlet of Sequalitchew Lake, which is upstream of the Dupont corporate limits. Flood flows are diverted into a large drainage canal that flows through the northeastern corner of the city and then empties into Puget Sound.

Town of Eatonville

The Eatonville storm-sewer system services the residential area in the center of town and the ponding areas in western and northeastern Eatonville. Storm flows are carried into the unincorporated area north of Eatonville.

No other forms of flood protection are known to exist within Eatonville.

City of Fife

Substantial channel improvements were made to lower Puyallup River in the vicinity of Fife between 1914 and 1919 by the Inter-County Improvement District. These improvements, which included channel straightening and levee construction, no longer provide 1-percent-annual-chance flood protection for Fife since these levees have been de-accredited by FEMA. Some flood protection is provided by Mud Mountain Dam, which was completed by the USACE in 1948. Mud Mountain Dam is 36 miles upstream of Fife on White River, a tributary to Puyallup River, and it was constructed for the purpose of controlling the magnitude of peak flood discharges of the lower Puyallup River.

City of Fircrest

A low earth-filled dam, located on Leach Creek approximately 100 feet upstream from the Fircrest corporate limits, forms a small flood-control holding pond. A manually operated gate in the dam and a glory-hole-type spillway near the upstream face prevent overtopping of the dam during small floods. The dam has minimal to negligible effects on the 1- and 0.2-percent-annual-chance floods.

The City of Tacoma has constructed and operates a pump station on Leach Creek within the city limits of University Place.

City of Gig Harbor

There are no known flood protection measures in operation or planned on North Creek, the unnamed tributary to North Creek, or along the Puget Sound shoreline at Gig Harbor.

City of Milton

The potential for flood damage along Hylebos Creek and Hylebos Creek Tributary has been minimized by a town ordinance that prevents indiscriminate landfill along flood plains and by town refusal to issue building permits for proposed construction within known flood-prone areas.

A drainage study is being conducted for Milton by Sitts and Hill Engineers, Incorporated. That study will recommend that some natural depressions, swamps, and lakes within the town be used for stormwater detention to reduce flood discharges by Hylebos Creek and Hylebos Creek Tributary.

City of Orting

Levees have been constructed on both sides of the Puyallup and Carbon Rivers. The levees along the Puyallup River provide less than 3 feet of freeboard with respect to the 1-percent-annual-chance flood. Furthermore, the degree of flood protection now provided against lesser frequency flood is expected to decrease in the future as additional sediment is deposited in the channel. The levee that runs along the west bank of the Carbon River provides more than 3 feet of freeboard between the top of the levee and the 1-percent-annual-chance flood; however, due to insufficient information regarding the levee's maintenance, it has not been credited with protecting Orting from the 1-percent-annual-chance flood.

Although several methods of increasing flood protection from the Puyallup River have been proposed, none of them have been undertaken. A 1974 study by the USACE included proposals such as raising existing levees, constructing new levees outside the existing ones, constructing a ring

levee around Orting, and removing gravel from the channel (Reference 28).

City of Puyallup

The confluence of the Puyallup and White Rivers is 0.1 mile upstream of the City of Puyallup. There is no regulation of flood discharges on the Puyallup River upstream of the confluence with the White River. However, White River flood discharges are regulated by the operation of Mud Mountain Dam, located 30 miles upstream of Puyallup. This dam, which was built for the purpose of controlling peak flood flows of the lower Puyallup River, serves as a flood protection device for the City of Puyallup.

An extensive network of storm drains and drainage ditches has been constructed, which has eliminated or reduced shallow flooding in some parts of the city south of the railroad. The primary drainage structure west of State Highway 512 is Meeker Ditch. This ditch normally drains the runoff quickly enough to prohibit shallow flooding in this area. However, during corresponding high flow in the Puyallup River and in Clarks Creek, where Meeker Ditch discharges, the drainage capabilities of the ditch are greatly reduced, and shallow flooding results. A substantial portion of the runoff from the east part of the city is collected in a 6-foot diameter storm drain which empties into the Puyallup River at the State Highway 512 Bridge.

Flood damage protection along portions of Clarks Creek is accomplished by means of non-structural measures. These measures consist of using flood-prone areas for parks or recreational purposes.

Town of Steilacoom

Flood damage protection along the Steilacoom shoreline is accomplished by means of nonstructural measures. These measures consist of using flood-prone areas for parks or recreational purposes.

City of Sumner

There are no dams or reservoirs to regulate flood discharges on the Puyallup River upstream of the confluence with the White River. However, Mud Mountain Dam, which is located on the White River 30 miles upstream of Sumner, had a usable storage capacity of 106,000 acre-feet. This storage helps to decrease the discharge downstream of the confluence by controlling the discharge on the White River, thus serving as a flood protection device for the city. The dam regulates flood flows to within bank flows, allowing a maximum of 15,500 cfs in the White River in the Sumner area, and 43,500 cfs in the Puyallup River downstream of Sumner.

Levees located on the Puyallup River upstream of the White River provide the city with some degree of protection against flooding; however, it has been ascertained that these levees may not protect the city from rare events such as the 1-percent-annual-chance flood. The criteria used to evaluate protection against the 1-percent-annual-chance flood are: adequate design, including freeboard; structural stability; and proper operation and maintenance. Levees that do not protect against the 1-percent-annual- chance flood are not considered in the hydraulic analysis of the 1-percent- annual-chance floodplain.

City of Tacoma

The City of Tacoma has constructed a flood control facility for Flett Creek in the vicinity of the southern corporate limits of Tacoma.

Swan Creek joins Clear Creek in a flat swampy area just beyond the corporate limits of Tacoma. The combined flow then goes through culverts under the Puyallup River levee and discharges through flap gates into the river. The flap gates prevent high flow in Puyallup River from entering into Clear Creek and Swan Creek. However, if Puyallup River remains at flood stage for a long time, or if there is high flow in Swan and/or Clear Creek at the same time as the high flow in the river, flooding near the mouths of Clear and Swan Creek can result because the flow in the creeks is prevented from discharging into Puyallup River by the closed flap gates.

The flood levels of Snake and Wapato Lakes are limited by their respective outlet structures, which allow for free outflow above specified lake elevations.

Substantial channel improvements, including channel straightening and levee construction, were made to the lower Puyallup River in the vicinity of Tacoma between 1936 and 1950 by the USACE. However, these levees have been de-accredited by FEMA in the hydrologic and hydraulic analyses for this countywide revision. Mud Mountain Dam was completed by the USACE in 1948. It is located 36 miles upstream of Tacoma on White River, a tributary to Puyallup River, and was constructed for the purpose of controlling the magnitude of peak flood discharges of the lower Puyallup River.

## 3.0 ENGINEERING METHODS

For flooding sources studied by detailed methods in the community, standard hydrologic and hydraulic study methods were used to determine the flood-hazard data required for this study. Flood events of a magnitude that is expected to be equaled or exceeded once on the average during any 10, 50, 100, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year flood, have a 10-, 2-, 1-, and 0.2-percent

chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long-term average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood that equals or exceeds the 1-percent-annual-chance flood in any 50-year period is approximately 40 percent (4 in 10); for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the community at the time of completion of this study. Maps and flood elevations will be amended periodically to reflect future changes.

**Note:** Within this jurisdiction there are one or more levees that have not been demonstrated by the community or levee owner to meet the requirements of 44CFR 65.10 as it relates to the levee's capacity to provide 1%-percent-annual-chance flood protection. Please refer to the Notice to Flood Insurance Study Users page at the front of this FIS report for more information.

## 3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for each flooding source studied by detailed methods affecting the communities within Pierce County.

### Pre-Countywide Analyses

The peak discharge-frequency for streams studied by detailed methods was computed from regression equations that relate peak discharge-frequency data to drainage area and mean annual precipitation. Drainage area and annual peak-discharge data for 53 continuous-record and 14 partial-record gaging stations (References 16-20, 24, 25, and 29) and mean annual precipitation data associated with each drainage area (Reference 30) were used in the development of the regression equations. The gaging stations, operated by the USGS from as early as 1914, are located in Pierce and Thurston Counties. For Muck Creek (at Roy), data from 25 gaging stations were used. Among the 25 gaging stations, one was located on Muck Creek at Roy. The number of years of peak-discharge record for the individual gaging stations ranged from 6 to 34 years with 6 stations having less than 10 years, 11 having 10 to 20 years, and 8 having more than 20 years. Peak discharge-frequency relationships were determined for each peak-discharge record by using the standard log-Pearson Type III method of analysis as outlined by the Water Resources Council (Reference 31).

Discharges for Leach Creek are recorded at the U.S. Geological Survey Gaging Station No. 12091200 at 40th Street West approximately 0.1 mile downstream from the Fircrest corporate limits. The peak discharge-frequency relationship for Leach Creek was based on a statistical analysis of the 21 years of annual peak-discharge records for this gaging station (References 16-20, 25, and 29). The standard log-Pearson Type

III method of analysis was used to develop this relationship (Reference 31).

Regional peak discharge-frequency relationships were supplanted with the gaging-station relationship for Leach Creek for two reasons. First, the holding pond upstream of Fircrest exercises some flood-flow retention that is not expressed by the regional relationships. And second, portions of the flood flows in Fircrest originate from storm drains that relieve runoff from areas outside of the Leach Creek drainage basin (that is, approximately 0.7 square mile of drainage from the nearby Fleet Creek basin to the east). The net effect of these factors is that the actual peak discharges of Leach Creek at Fircrest are greater than those computed using regional peak discharge-frequency relationships.

The peak discharge-frequency relationship for Swan Creek was computed using the log-Pearson Type III method of analysis for 21 annual peaks recorded at Gaging Station No. 12102200 (References 16-18 and 25). This gage, which was operated between 1951 and 1971, was located on Swan Creek at South 72nd Street. The regression equations discussed previously were not used to compute the Swan Creek relationship because peak runoff from Swan Creek is uncharacteristic of most other streams in the region, perhaps due to differences in topography, soils, or both. The discharges from the gaging station relationship are approximately six times as great as those computed from the regression equations.

Regional peak discharge-frequency relationships that were developed previously (References 32 and 33) were investigated before developing the regression equations used in this study. However, these relationships were not used because additional peak-discharge data have since become available, the log-Pearson Type III method of analysis has since been improved and standardized, and relationships for a smaller region were needed to more accurately reflect localized flood flow conditions.

Analyses were carried out to establish the peak elevation-frequency relationships for the remaining flooding sources studied in detail.

Regional relationships were developed for estimating the differences between mean lake elevation and the 10-, 2-, 1-, and 0.2-percent-annual-chance peak elevations, based on log-Pearson Type III analysis (Reference 31). Elevation records collected on Ohop Lake from 1960 to 1975, on Tanwax Lake from 1962 to 1975, and on seven other lakes in western Washington with similar hydrologic settings (References 16-20) were used as a source of data for this analysis. These relationships were applied to determine the flood-peak elevations for 12 lakes in Pierce County by adding difference values to lake elevations at time of photography, which were considered to be at the mean levels.

Peak discharge-frequency data for the Puyallup River were computed using a combination of methods. The methodologies used are related to

the degree of expected flood discharge reduction due to the operation of Mud Mountain Dam. The 10-percent-annual-chance peak discharge was obtained from a peak discharge-frequency relationship developed for the Puyallup River at Puyallup gaging station No. 12101500, operated by the U.S. Geological Survey. The gaging station is located 0.25 mile downstream of the City of Puyallup and its period of record extends from 1915 to the present. However, because construction of Mud Mountain Dam was not completed until 1948, the peak discharge-frequency relationship was developed using only the annual peaks for the period from 1948 to 1978 (References 16-19 and 24-27).

The primary consideration by the USACE in the operation of Mud Mountain Dam during large floods is to maintain the flow in the lower Puyallup River at a maximum discharge no greater than 45,000 cfs. Discharges obtained from the peak discharge-frequency relationship for the 2-, 1-, and 0.2-percent-annual-chance peaks are all greater than 45,000 cfs because it does not accurately reflect the peak reducing capabilities of Mud Mountain Dam during severe floods. The regulated flow of 45,000 cfs was, therefore, used for both the 2- and 1-percent-annual-chance flood events. However, the 0.2-percent-annual-chance peak discharge would surpass 45,000 cfs because the unregulated flow in the Puyallup River would exceed that amount. Therefore, the 0.2-percent-annual-chance peak discharge was computed as the total of the unregulated flows in the Puyallup River upstream of the confluence with the White River and the White River downstream of Mud Mountain Dam.

The magnitudes of flood discharges in the White River at Buckley are a result of releases from Mud Mountain Dam. The USACE, who operate the dam, provided their design flood release figures for floods of the selected recurrence intervals (Reference 34). Because Mud Mountain Dam is operated for flood-control purposes, it is anticipated that runoff from tributaries entering the White River downstream from the dam would have receded substantially prior to the release of flood discharges. Therefore, the design flood release figures were used for the study reach at Buckley without any adjustment for intervening inflow.

Elevations for Steilacoom Lake are controlled by culverts at the outlet of the lake, and the elevations for the selected recurrence intervals were developed from a culvert rating using discharges from the peak discharge- frequency relationships from Chambers Creek.

Peak elevation-frequency relationships for Snake Lake and Wapato Lake were computed using regression equations that relate peak elevation-frequency data to drainage area, lake surface area, and mean annual precipitation. Drainage area, lake surface area, and peak elevation data for nine hydrologically similar lakes in western Washington (References 16- 20, 24, and 25) and mean annual precipitation data associated with each drainage area (Reference 30) were used in the development of the regression equations. The standard log-Pearson Type III method of analysis was used to determine the peak elevation-frequency relationships for each peak elevations record.

The peak tidal elevation-frequency relationship for floods along the Puget Sound shoreline was developed by analyzing 71 years of annual peak tidal elevations at the Seattle Tidal Station, obtained from the National Oceanic and Atmospheric Administration in 1975, with the log-Pearson Type III method, using 0.2 skew. Values of the 10-, 2-, 1-, and 0.2-percent-annual- chance peak tidal elevations were then transferred from Seattle to Commencement Bay, Day Island, and the Nisqually Reach by applying adjustments determined from tide prediction tables (Reference 35).

Wave run-up was determined by comparing the high-water-mark elevations of the December 1977 storm against the recorded high-tide levels as transferred from Seattle.

Peak elevations of Debra Jane Lake and Bonney Lake were computed hydraulically based on their respective outlet structures, road overflow sections, and computed inflow. No adjustments were made for storage because of the small size of the lakes.

The 1-percent-annual-chance tide elevation at Gig Harbor was verified by a survey of high-water marks left by the December 15, 1977, high tide, which had a recurrence interval of approximately 100 years at the Seattle gage.

The peak tidal elevation-frequency relationship reflects the still-water elevations due to tidal and storm-surge effects, but it does not include wave cresting or run-up effects. Future developments should make allowance for potential additional hazards caused by wave action.

The City of Dupont is located at the downstream end of the Sequalitchew Creek drainage basin. However, hydrologic analysis of the drainage basin indicates that only a small portion of the potential surface runoff of the basin ever reaches the Dupont. Large portions of the drainage basin consist of noncontributing lakes and swamps, which either have no outlets or drain very slowly, and much of the runoff from the remaining areas is diverted into a drainage canal upstream of Dupont.

**Initial Countywide Analyses (TBD)**

Analysis of the hydraulic characteristics of flooding along the all study reaches were performed using detailed methods. Detailed methods involved using a HEC-RAS (Reference 89 and 90) water surface profile computer model. Conventional HEC-RAS modeling techniques were not used for the Clear Creek Study.

In accordance with FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (2002), the starting water surface elevations for the profiles and backwater models were estimated using normal depth procedures. Starting water surface elevations for the Carbon River were

assumed to have coincident peaks with the Puyallup River. The starting water surface elevation for Clarks Creek used “with levee” conditions on the Puyallup. The downstream boundary conditions for Crescent and Wapato Creeks consist of culvert structures, thus the normal depths starting water surface elevations recommended by FEMA were not implemented.

The following streams were newly studied or re-studied for this initial countywide FIS.

*Artondale Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 94).*

For this study, hydrologic analyses were carried out to establish peak discharge-frequency relationships for the Puyallup River. Flood frequency discharges were estimated for the 10-, 50-, 100-, and 500-year floods for existing land use conditions.

*Canyon Creek Study*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 95).*

Since no gauges exist on Canyon Creek that could provide a significant period of record, peak annual flows were computed from the HSPF modeling software. The HSPF model was originally created and calibrated by the USGS in 1995 and has subsequently been updated for various studies. Most recently the model was updated and used to estimate existing and future conditions flows for the *Clear/Clarks Creek Basin Plan,* which is set to be finalized in 2005. Future flow conditions are based on the land use conditions developed in the *Comprehensive Plan for Pierce County Washington* (1993) and are discussed in greater detail in *Clear/Clarks Creek Basin Plan*. No modifications were made to either the existing or future flow conditions modeling that was done for the Basin Plan. Both existing and future land use conditions are documented in detail in the *Clear/Clarks Creek Basin Plan.*

At the direction of Pierce County, the rainfall record used in the HSPF model for this study was the 156-year synthetic rainfall record developed by MGS Engineering Consultants, Inc. (2001). The log-Pearson Type III distribution was applied to annual peak flows generated from the HSPF model.

Slight modifications were made to the original HSPF model in order to facilitate the extraction of peak flows for the necessary recurrence intervals. To ensure that these changes did not have an impact on the model results, the 100-year existing conditions peak flows computed for this study were compared with the peak flows listed in the Draft *Clear/Clarks Creek Basin Plan*. There were no significant differences.

Because only three flow locations were defined in the HSPF model, additional flow input locations were linearly interpolated, based on tributary drainage area. Flow input locations were typically assumed to occur near major road crossings. Flows were apportioned conservatively; the area tributary to a street crossing was generally assigned to the next upstream street crossing.

*Carbon River Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 96).*

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for the Carbon River. Flood frequency discharges were estimated for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods using Bulletin #17B procedures (USGS, 1981). USGS-operated stream gauging station, Carbon River near Fairfax, Washington (12094000), has recorded instantaneous peak discharges over water years 1930 to 2000. The gage is located approximately 6.5 miles upstream of the upstream study reach boundary and approximately 15 miles upstream of the Carbon River mouth.

For the purposes of the hydrologic analysis, the Carbon River was divided into three reaches. Flow quantiles for the upstream reach (South Prairie Creek confluence to upstream study reach boundary) were estimated by fitting a flood frequency curve to the instantaneous annual maximum flows at USGS station 12094000 and then adjusting for differences in drainage areas and precipitation. For the middle reach (Voight Creek confluence to South Prairie Creek confluence), the flow quantiles for the upstream reach of the Carbon River were summed with the flow quantiles for South Prairie Creek, which were estimated by fitting a flood frequency curve to the instantaneous annual maximum flows at USGS station 12095000. Subsequently, a further adjustment for drainage area and precipitation was made. The flow quantiles for the middle reach were adjusted based on drainage area and precipitation to estimate the flow quantiles for the downstream study reach boundary (mouth to Voight Creek confluence).

The resultant flows are summarized in Table 2 - Summary of Discharges.

*Clarks Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 97).*

Flood frequency quantiles for current and future conditions on Clarks Creek were estimated using a Hydrologic Simulation Program – Fortran (HSPF) model of the Clear/Clarks Creek basin, originally developed by

the USGS (Mastin, 1996), and later modified by nhc (nhc, 2003) and CH2M Hill (CH2M Hill, 2003).

*Clear Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 98).*

Conventional HEC-RAS modeling techniques were not used for this FIS because the flooding is the result of interior drainage ponding rather than stream capacity/conveyance limitations. See the above nhc report for more information.

*Clover Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 99).*

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for Clover Creek and its North Fork. The historical stream flow data available for streams within the Clover Creek basin consists of three U.S. Geological Survey (USGS) gaging stations located on Clover Creek (12090355, 12090360, and 12090500), one on the North Fork (12090400), and three on unnamed tributaries (12090340, 12090365, 12090380). Due to the limited period of record available for most of these stations, the discharges used in this study were generated using an existing continuous simulation model of the drainage basin. The USGS created the model in the early 1990s using the Hydrologic Simulation Program Fortran (HSPF) package (Mastin, 1992). A complete summary of the hydrologic analysis was completed separately in July 2002. In short, the HSPF model was used to estimate historical annual instantaneous peak flows along the stream for a period covering 51 years (1949 – 1999). Bulletin #17B (WRCHC, 1981) procedures were then used to fit a Log-Pearson III distribution to the annual peak flows to generate 10%, 2%, 1%, and 0.2%-chance-exceedance flood quantiles at key locations along the stream.. Since approval of the hydrologic analysis two minor modifications have been made to the flows to be used in the hydraulic analysis. Firstly, the HSPF reach 15 which extends from the confluence of North Fork and Clover Creeks downstream to Spanaway Creek was divided and an additional frequency analysis was performed at the upstream end of the reach. Secondly, the flow quantiles in the lower end of the model, downstream of Spanaway Creek, were revised slightly for consistency (less than 1% change for 1%-chance-exceedance quantile).

*Crescent Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 100).*

The HEC-RAS model for Crescent Creek includes 83 cross-sections. These cross-sections serve to represent the geometry of the channel and floodplain along the study reach. Since approximate methods were employed for the effective FIS study of Crescent Creek no previous cross-sectional data exists. As a result, 35 new channel cross-sections were surveyed along the study reach. At select locations, such as culvert or bridge crossings, surveyed channel cross-sections were copied upstream or downstream to represent similar channel geometry. This occurred at 21 locations. In reaches where access to the channel was unavailable, channel cross-sections were interpolated to augment the surveyed data and occurred at 27 locations. Topographic data provided by Pierce County, in the vicinity of Crescent Creek was used to extend these cross-sections to include the floodplain. The topographic data was developed from aerial photographs taken in 1998, with a contour interval of two feet.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised studies was the National Geodetic Vertical Datum of 1929 (NGVD 29). With the finalization of the North American Vertical Datum of 1988 (NAVD 88), most studies are being prepared using NAVD 88 as the referenced vertical datum. The initial hydraulic analysis for the Crescent Creek FIS was conducted using the NGVD 29 vertical datum; however, the vertical datum was adjusted to NAVD 88 for the final submission. Precise conversion factors between the two vertical datums vary by location and can be obtained from the Army Corps of Engineers' CORPSCON utility (USACE, 1997). Specifically, NGVD 29 elevations along the Crescent Creek study reach can be converted to NAVD 88 elevations by adding 3.48 ft.

The hydraulic analysis of the Crescent Creek study reach includes 12 culvert crossings, 11 bridges, and 1 in-line weir structure. Within the HEC-RAS model, all culverts were modeled using the Highest Upstream Energy method, while bridges were modeled either with the Energy, or Pressure and/or Weir methods, depending on level of inundation.

A split-flow reach and in-line weir structure were implemented between cross-sections 1 and 4 to model the main stem of Crescent Creek and expected overtopping of the roadway at Crescent Valley Drive, respectively.

The downstream boundary conditions for Crescent Creek consists of culvert structure, thus the normal depth starting water surface elevations recommended by FEMA (FEMA, 2003) were not implemented. Alternatively, a culvert structure was included in the HEC-RAS model and the culvert routine was used to determine headwater depths. To facilitate the computation, cross-section A was placed at the downstream end of the culvert structure within the Gig Harbor estuary. A fixed tailwater depth was then selected as the boundary conditions for the study reach. The

mean higher high water tide level of 9.34 feet, NAVD 88 (5.86 feet, NGVD 29) was selected for all flood profiles modeled (USDC, 2003).

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the creek and floodplain areas. The channel "n" values for the main channel of Crescent Creek typically range from 0.04 to 0.055. Overbank "n" values range from 0.04 to 0.15. No reliable high water marks were available to calibrate the Crescent Creek model.

*Diru Creek Study*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 101).*

Since no gauges exist on Diru Creek that could provide a significant period of record, peak annual flows were computed from the HSPF modeling software. The HSPF model was originally created and calibrated by the USGS in 1995 and has subsequently been updated for various studies. Most recently the model was updated and used to estimate existing and future conditions flows for the *Clear/Clarks Creek Basin Plan,* which is set to be finalized in 2005. Future flow conditions are based on the land use conditions developed in the *Comprehensive Plan for Pierce County Washington* (1993) and are discussed in greater detail in *Clear/Clarks Creek Basin Plan*. No modifications were made to either the existing or future flow conditions modeling that was done for the Basin Plan. Both existing and future land use conditions are documented in detail in the *Clear/Clarks Creek Basin Plan.*

At the direction of Pierce County, the rainfall record used in the HSPF model for this study was the 156-year synthetic rainfall record developed by MGS Engineering Consultants, Inc. (2001). The log-Pearson Type III distribution was applied to annual peak flows generated from the HSPF model.

The HSPF model was modified to provide a higher resolution of flow data in the upper portion of the Diru Creek Basin. The upper basin was subdivided into three subbasins in order to determine the flows upstream and downstream of the large pond south of 112th Street East. A detailed description of the modifications to the HSPF model for the upper portion of the basin is provided in the appendix. Flows from the lower portion of the basin were extracted from the same model used for the *Clear/Clarks Creek Basin Plan* because that model was previously calibrated and represented the most verified data available.

Because only two flow locations were defined in the lower portion of the HSPF model, additional flow input locations were linearly interpolated, based on tributary drainage area. Flow input locations were typically assumed to occur near major road crossings. Flows were apportioned

conservatively; the area tributary to a street crossing was generally assigned to the next upstream street crossing. Flows were apportioned at a higher resolution in the upper portion of the basin where development adjacent to the creek is more extensive and a greater accuracy of flow impacts was desired.

*East Fork Clear Creek Study*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 102).*

Since no gauges exist on the East Fork or the West Stem that could provide a significant period of record, peak annual flows were computed from the HSPF modeling software. The HSPF model was originally created and calibrated by the USGS in 1995 and has subsequently been updated for various studies. Most recently the model was updated and used to estimate existing and future conditions flows for the *Clear/Clarks Creek Basin Plan,* which is set to be finalized in 2005. Future flow conditions are based on the land use conditions developed in the *Comprehensive Plan for Pierce County Washington* (1993) and are discussed in greater detail in *Clear/Clarks Creek Basin Plan*. No modifications were made to either the existing or future flow conditions modeling that was done for the Basin Plan. Both existing and future land use conditions are documented in detail in the *Clear/Clarks Creek Basin Plan.*

At the direction of Pierce County, the rainfall record used in the HSPF model for this study was the 156-year synthetic rainfall record developed by MGS Engineering Consultants, Inc. (2001). In order to estimate flow quantiles for both existing and future conditions, the log-Pearson Type III distribution was applied to annual peak flows generated from the HSPF model.

Slight modifications were made to the original HSPF model in order to facilitate the extraction of peak flows for the necessary recurrence intervals. To ensure that these changes did not have an impact on the model results, the 100-year existing-conditions peak flows computed for this flood study were compared with the peak flows listed in the Draft *Clear/Clarks Creek Basin Plan*. No significant differences were noted.

Because only three flow locations were defined in the HSPF model, additional flow input locations were linearly interpolated, based on tributary drainage area. Flow input locations were typically assumed to occur near major road crossings. Flows were apportioned conservatively; the area tributary to a street crossing was generally assigned to the next upstream street crossing.

*Fennel Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 103).*

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for Fennel Creek. Flood frequency discharges were estimated for the 10-, 50-, 100-, and 500-year floods for both existing and future landuse conditions using Bulletin #17B procedures (Water Resources Council, 1981). No historical stream flow gaging records are available for Fennel Creek, therefore annual instantaneous peak discharges were estimated using an HSPF model of the watershed created for this study by nhc.

*Lacamas Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 104).*

The revised hydrologic information for Lacamas Creek was obtained from a previous study developed for Pierce County by James M. Montgomery Consulting Engineers. The hydrologic analyses performed in the study consisted of the development of a HEC-1 model of the Lacamas Creek drainage basin. From this model the 50-, 10-, 4-, and 1-percent-annual-chance quantiles were determined. The 2- and 0.2-percent-annual-chance quantiles were determined from curves fit to the HEC-1 quantiles.

*Mashel River Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 105).*

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for the Mashel River study reach. Flood frequency discharges were estimated for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods for existing land use conditions using Bulletin #17B procedures (Water Resources Council, 1981). The computed discharges were estimated by applying a log-Pearson III analysis to 26 years of annual instantaneous peak flows recorded at USGS gage 12087000 -- Mashel River near La Grande, Washington. The gage is located approximately 1.2 miles downstream from the study reach. Flows upstream of the Little Mashel River confluence are also based upon the log-Pearson III analysis of the gage data, but have been adjusted to account for the reduced drainage area.

*Morey Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 106).*

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for the Morey Creek study reach. Morey Creek stream flows were generated using an existing HSPF (Hydrologic Simulation Program Fortran) model that was developed by the USGS (Mastin, 1992). Some modifications were made to the model during the Spanaway Creek FIS (nhc, 2002). For this study the same HSPF model was used to estimate historical annual instantaneous peak flows along the stream for a period covering 51 years (1949 – 1999). Bulletin 17B (WRCHC, 1981) procedures were then used to fit a Log-Pearson III distribution to the annual peak flows to generate 10-, 2-, 1-, and 0.2-percent-annual-chance flood quantiles at the upstream end of the stream. Flood quantiles were also estimated for the 1-percent-annual-chance future build-out land use condition.

The flows used in this study vary slightly from those used to model a small portion of Morey Creek in the Spanaway Creek FIS because of two reasons. First, the Morey Creek flows used in the Spanaway Creek FIS were determined by split flow optimization with HEC-RAS and they did not include any of the local inflow area to Morey Creek. Second, for this FIS a separate frequency analysis for Morey Creek was performed which estimated discharges independently from Spanaway Creek.

*North Fork of Clover Creek*

See Clover Creek Study.

*North Fork of Clover Creek Tributaries*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 107).*

The continuous stream flow gauges on the North Fork of Clover Creek do not provide an adequate period of record for predicting the return frequencies needed, nor are there enough gauges throughout the basin to make accurate estimates for all tributaries modeled. Peak annual flows were computed from the Hydrologic Simulation Program – FORTRAN (HSPF) modeling software. The HSPF model was originally created and calibrated by the USGS in 1995 and has been subsequently updated to include the E-1 and W-1 regional detention ponds for various studies. Most recently the model was updated and used to estimate flows for the North Fork of Clover Creek FIS (Northwest Hydraulic Consultants (nhc), 2004). This latest update was also used for this FIS.

The rainfall record used in the HSPF model for this study was based on a 51-year hourly period of record from 1949 to 2000. The primary source of rainfall data was the nearby McMillan Reservoir NOAA-NWS precipitation data set. The McMillan reservoir period of record began in 1961, with gaps between December 1980 and September 1985. SeaTac precipitation data was used to extend the data and fill data gaps. NHC

also made adjustments to the January 1997 precipitation data to account for the unusual amount of snow accumulation and snowmelt that occurred during that flood event.

The log-Pearson Type III distribution was applied to the annual peak flows generated from the HSPF model. In order to estimate flows in the upper portions of the watershed where they had not been computed in the HSPF model, peak flows were apportioned at key road crossings based on tributary drainage area.

*Puyallup River Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 108).*

For this study, hydrologic analyses were carried out to establish peak discharge-frequency relationships for the Puyallup River. Flood frequency discharges were estimated for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods for existing land use conditions.

*Rody Creek*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 109).*

Since no gauges exist on Rody Creek that could provide a significant period of record, peak annual flows were computed from the HSPF modeling software. The HSPF model was originally created and calibrated by the USGS in 1995 and has subsequently been updated for various studies. Most recently the model was updated and used to estimate existing and future conditions flows for the *Clear/Clarks Creek Basin Plan* (Reference 93), which is set to be finalized in 2005. Future flow conditions are based on the land use conditions developed in the *Comprehensive Plan for Pierce County Washington* (1993) and are discussed in greater detail in *Clear/Clarks Creek Basin Plan* (Reference 93). No modifications were made to either the existing or future flow conditions modeling that was done for the Basin Plan. Both existing and future land use conditions are documented in detail in the *Clear/Clarks Creek Basin Plan* (Reference 93).

At the direction of Pierce County, the rainfall record used in the HSPF model for this study was the 156-year synthetic rainfall record developed by MGS Engineering Consultants, Inc. (2001). In order to estimate flow quantiles for both existing and future conditions, the log-Pearson Type III distribution was applied to annual peak flows generated from the HSPF model.

Slight modifications were made to the original HSPF model in order to facilitate the extraction of peak flows for the necessary recurrence intervals. To ensure that these changes did not have an impact on the

model results, the 100-year existing conditions peak flows computed for this study were compared with the peak flows listed in the Draft *Clear/Clarks Creek Basin Plan* (Reference 93). There were no significant differences.

Slight modifications were made to the original HSPF model in order to facilitate the extraction of peak flows for the necessary recurrence intervals. To ensure that these changes did not have an impact on the model results, the 1-percent-annual-chance existing conditions peak flows computed for this study were compared with the peak flows listed in the Draft *Clear/Clarks Creek Basin Plan* (Reference 93). There were no significant differences.

Because only three flow locations were defined in the HSPF model, additional flow input locations were linearly interpolated, based on tributary drainage area. Flow input locations were typically assumed to occur near major road crossings. Flows were apportioned conservatively; the area tributary to a street crossing was generally assigned to the next upstream street crossing.

*South Prairie Creek Study*

*For Tables, Photos, and Appendices related to this study, please see the nhc report (Reference 110).*

Annual instantaneous peak discharges for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods were estimated following FEMA guidelines. The log-Pearson Type III distribution was applied following the procedures outlined in Bulletin 17B (U.S. Water Resources Council, 1981) to the annual instantaneous peak discharges collected at a USGS-operated stream gauging station located near the town of South Prairie (RM 5.81, USGS Gage Number 12095000). Annual instantaneous peak discharges have been collected at the gage continuously since 1950. The computed flood frequency estimates were adjusted to reflect changes in the contributing drainage area along the study reach. Comparing the new and previous values reveals that the discharges used in the new study are significantly higher than those used in the previous study (approximately 30% higher). This is because the stream has experienced several major floods since the previous study was published. As a result, the upper end of the log-Pearson flood frequency distribution curve yields higher flood discharge estimates.

*Spanaway Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 111).*

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for the Spanaway Creek study reach. Spanaway Creek stream flows were generated using an existing HSPF (Hydrologic Simulation Program Fortran) model that was developed by the USGS

(Mastin, 1992).In short, the HSPF model was used to estimate historical annual instantaneous peak flows along the stream for a period covering 51 years (1949 – 1999). Bulletin 17B (WRCHC, 1981) procedures were then used to fit a Log-Pearson III distribution to the annual peak flows to generate 10-, 50-, 100-, and 500-yr flood quantiles at key locations along the stream. Flood quantiles were also estimated for the 100-yr future build-out landuse condition.

*Swan Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 112).*

Flood frequency quantiles for current and future conditions on Swan Creek were estimated using a Hydrologic Simulation Program – Fortran (HSPF) model originally developed by the USGS (Mastin, 1996), and later modified by nhc (nhc, 2003) and CH2M Hill (CH2M Hill, 2003). Further calibration of the model was accomplished by nhc using USGS gage records available for Swan Creek.

*Wapato Creek I & II Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 113).*

The revised hydrologic information for Wapato Creek was obtained from a previous study developed for Pierce County by James M. Montgomery Consulting Engineers. The hydrologic analyses performed in the study consisted of the development of a HEC-1 model of the Hylebos Creek area, including the Wapato Creek drainage basins. From this model the 50-, 10-, 4-, and 1-percent-annual-chance quantiles were determined. The 2- and 0.2-percent-annual-chance quantiles were determined from curves fit to the HEC-1 quantiles.

*White River Study*

*For Tables, Photos, and Appendices related to this study, please see the nhc report (Reference 114).*

The revised hydrologic information for the White River was obtained from flood frequency analysis performed on gage data collected by the U.S. Geological Survey at several gages on the White River below Mud Mountain Dam. Because the Lake Tapps hydropower facility is no longer operating and was historically constrained from operating under high tailwater conditions, it was assumed that return flow from Lake Tapps does not contribute to the flood events.

*Woodland Creek*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 115).*

Since no gauges exist on Woodland Creek that could provide a significant period of record, peak annual flows were computed from the HSPF modeling software. The HSPF model was originally created and calibrated by the USGS in 1995 and has subsequently been updated for various studies. Most recently the model was updated and used to estimate existing and future conditions flows for the *Clear/Clarks Creek Basin Plan,* which is set to be finalized in 2005. Future flow conditions are based on the land use conditions developed in the *Comprehensive Plan for Pierce County Washington* (1993) and are discussed in greater detail in *Clear/Clarks Creek Basin Plan*. No modifications were made to either the existing or future flow conditions modeling that was done for the Basin Plan. Both existing and future land use conditions are documented in detail in the *Clear/Clarks Creek Basin Plan.*

At the direction of Pierce County, the rainfall record used in the HSPF model for this study was the 156-year synthetic rainfall record developed by MGS Engineering Consultants, Inc. (2001). The log-Pearson Type III distribution was applied to annual peak flows generated from the HSPF model.

Slight modifications were made to the original HSPF model in order to facilitate the extraction of peak flows for the necessary recurrence intervals. To ensure that these changes did not have an impact on the model results, the 1-percent-annual-chance existing-conditions peak flows computed for this flood study were compared with the peak flows listed in the Draft *Clear/Clarks Creek Basin Plan*.

The decrease in predicted peak flow from 84th Street East to Pioneer Way (80 cfs to 65 cfs) was highlighted for further investigation. The cursory review did not reveal that any significant changes were required in the model formulation. The decrease in flows near Pioneer Way is likely a function of flow attenuation in the broad flat wetland/floodplain just south of Pioneer Way.

Because only four flow locations were defined in the HSPF model, additional flow input locations were linearly interpolated, based on tributary drainage area. Flow input locations were typically assumed to occur near major road crossings. Flows were apportioned conservatively; the area tributary to a street crossing was generally assigned to the next upstream street crossing. Flows were apportioned at a higher resolution in the upper portion of the basin where development adjacent to the creek is more extensive and a greater accuracy of flow impacts was desired.

Peak discharge-drainage area relationships for Pierce County are shown in Table 2, Summary of Discharges.

Elevations for flood of the selected recurrence intervals on the 17 lakes and the Puget Sound studied by detailed methods are shown in Table 3, Summary of Elevations.

There were no new coastal hydrologic analyses performed for this initial countywide FIS report.

## Table 2. Summary of Discharges

| Flooding Source and Location | Drainage Area (square miles) | Peak Discharges (cubic feet per second) 10- Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|---|
| ARTONDALE CREEK | | | | | |
| At Mouth | 3.15 | 128 | 199 | 232 | 315 |
| At Artondale Drive crossing | 2.71 | 118 | 202 | 249 | 375 |
| At Confluence with West Branch tributary | 0.81 | 31 | 56 | 72 | 120 |
| ARTONDALE CREEK WEST BRANCH | | | | | |
| At Mouth | 1.9 | 130 | 202 | 232 | 307 |
| At West Wetland | 1.38 | 81 | 122 | 138 | 178 |
| BONNEY LAKE OUTFLOW | | | | | |
| At Mouth | 0.5 | 16 | 22 | 24 | 30 |
| CANYON CREEK | | | | | |
| Canyon Creek at 96th St. E. | N/A | 11 | 18 | 22 | 33 |
| Canyon Creek at 90th St. E. (West Fork) | N/A | 34 | 47 | 52 | 64 |
| Canyon Creek (below confluence) | N/A | 33 | 60 | 76 | 131 |
| CARBON RIVER | | | | | |
| At Mouth | 230 | 18,600 | 26,800 | 30,400 | 39,100 |
| Upstream of Confluence with Voight Creek | 191 | 15,300 | 22,100 | 25,000 | 32,200 |
| Upstream of Confluence with South Prairie Creek | 97 | 8,700 | 12,700 | 14,500 | 19,100 |
| CHAMBERS CREEK | | | | | |
| At the Downstream Limit of Detailed Study | 106 | 756 | 1,070 | 1,200 | 1,530 |
| At Cross Section Z | 90 | 671 | 945 | 1,060 | 1,360 |
| CLARKS CREEK | | | | | |
| At Mouth | 10.62 | 231 | 291 | 315 | 369 |
| At Stewart Avenue | 9.08 | 206 | 263 | 288 | 345 |
| At Confluence with Diru Creek | 8.69 | 203 | 260 | 284 | 342 |

## Table 2. Summary of Discharges

| Flooding Source and Location | Drainage Area (square miles) | Peak Discharges (cubic feet per second) 10- Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|---|
| At 7th Avenue SW | 4.73 | 128 | 163 | 177 | 209 |
| CLEAR CREEK | | | | | |
| At BNSF Railroad | 2 | 158 | 216 | 243 | 310 |
| CLOVER CREEK | | | | | |
| At Mouth | N/A | 297 | 439 | 506 | 680 |
| At Gravelly Lake Drive | N/A | 291 | 434 | 501 | 677 |
| At Confluence with Spanaway Creek | N/A | 178 | 316 | 396 | 644 |
| At C Street | N/A | 327 | 547 | 665 | 1,010 |
| At Confluence with North Fork Clover Creek | N/A | 89 | 192 | 254 | 453 |
| At 25th Avenue East | N/A | 82 | 127 | 149 | 209 |
| At Military Road | N/A | 28 | 38 | 43 | 54 |
| CRESCENT CREEK | | | | | |
| At Mouth | 5.53 | 280 | 480 | 580 | 880 |
| At River Mile (RM) 0.9 | 4.59 | 220 | 370 | 450 | 660 |
| At (RM) 1.5 | 2.88 | 120 | 200 | 240 | 320 |
| At (RM) 3.1 | 1.26 | 40 | 60 | 70 | 90 |
| DEBRA JANE CREEK | | | | | |
| At Mouth | 1.3 | 45 | 62 | 69 | 85 |
| At Confluence with Bonney Lake Outflow | 0.8 | 26 | 34 | 38 | 48 |
| At Upstream End of Debra Jane Lake | 0.1 | 9 | 12 | 14 | 17 |
| DIRU CREEK | | | | | |
| 124th Street East | N/A | 9.0 | 15.0 | 18.0 | 27.0 |
| 112th Street East | N/A | 7.0 | 9.0 | 9.5 | 10.0 |
| 104th Street East | N/A | 21.6 | 36.7 | 45.5 | 71.7 |
| 84th Street East | N/A | 25 | 55 | 73 | 129 |

## Table 2. Summary of Discharges

| Flooding Source and Location | Drainage Area (square miles) | Peak Discharges (cubic feet per second) 10- Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|---|
| EAST FORK of CLEAR CREEK | | | | | |
| East Fork Clear Creek at 96th St E (West Stem) | N/A | 24 | 32 | 35 | 43 |
| East Fork Clear Creek at 96th St E (Main Stem) | N/A | 19 | 27 | 29 | 31 |
| East Fork Clear Creek at 72nd Street E | N/A | 41 | 59 | 66 | 84 |
| FENNEL CREEK | | | | | |
| At Mouth | 13 | 473 | 642 | 720 | 913 |
| At RM 0.68 | 12.8 | 484 | 647 | 719 | 895 |
| At RM 2.02 | 12.2 | 476 | 630 | 698 | 863 |
| At RM 3.78 | 8 | 340 | 459 | 512 | 642 |
| At RM 5.41 | 5.1 | 247 | 322 | 353 | 423 |
| FLETT CREEK | | | | | |
| At Mouth | 7 | 109 | 150 | 169 | 211 |
| Upstream of the Corporate Limits in Tacoma, Washington | 6 | 84 | 115 | 129 | 160 |
| At Station No. 12091060 | 5.9 | 80 | 109 | 122 | 152 |
| At South 56th Street | 2.6 | 43 | 59 | 66 | 81 |
| At Upstream End of Study Reach | 1.3 | 27 | 36 | 40 | 49 |
| GREENWATER RIVER | | | | | |
| At Mouth | 76 | 5,620 | 8,080 | 9,180 | 11,900 |
| HORN CREEK | | | | | |
| At Mouth | 15 | 192 | 266 | 299 | 376 |
| At Hart's Lake Loop Road (8th Avenue South) | 7 | 113 | 155 | 174 | 217 |
| HYLEBOS CREEK | | | | | |
| At Mouth | 19 | 244 | 339 | 381 | 480 |

## Table 2. Summary of Discharges

| Flooding Source and Location | Drainage Area (square miles) | Peak Discharges (cubic feet per second) 10- Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|---|
| LACAMAS CREEK | | | | | |
| At Mouth | 14.99 | 412 | 691 | 815 | 1,055 |
| At RM 2.90 | 11.28 | 337 | 542 | 626 | 816 |
| At RM 4.85 | 8.24 | 282 | 490 | 580 | 760 |
| At RM 6.64 | 3.17 | 135 | 243 | 289 | 381 |
| LEACH CREEK | | | | | |
| At Bridgeport Way | 7 | 125 | 205 | 245 | 380 |
| At South 40th Street | 5 | 90 | 160 | 200 | 325 |
| LITTLE MASHEL RIVER | | | | | |
| At Mouth | 24 | 1,450 | 1,940 | 2,140 | 2,630 |
| Upstream of Confluence with Midway Creek | 15 | 1,010 | 1,370 | 1,520 | 1,890 |
| MASHEL RIVER | | | | | |
| At Mouth | 81 | 4,995 | 7,215 | 8,250 | 10,900 |
| Upstream of Confluence with Little Mashel River | 56 | 3,490 | 5,045 | 5,770 | 7,620 |
| MEEKER DITCH | | | | | |
| At Mouth | 3.16 | 101 | 130 | 142 | 168 |
| MOREY CREEK | | | | | |
| At Mouth | 6.5 | 35 | 50 | 57 | 73 |
| NORTH CREEK | | | | | |
| At Mouth | 1.7 | 82 | 112 | 126 | 157 |
| At Harborview Drive | 1.6 | 76 | 103 | 116 | 144 |

## Table 2. Summary of Discharges

| Flooding Source and Location | Drainage Area (square miles) | Peak Discharges (cubic feet per second) 10- Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|---|
| NORTH FORK CLOVER CREEK | | | | | |
| At Mouth | N/A | 258 | 418 | 503 | 751 |
| At Confluence with West Fork Clover Creek | N/A | 73 | 121 | 148 | 225 |
| OHOP CREEK | | | | | |
| At Mouth | 44 | 953 | 1,345 | 1,518 | 1,945 |
| At Kapowsin Eatonville Road | 35 | 893 | 1,259 | 1,421 | 1,820 |
| Upstream of the Confluence with Lynch Creek | 18 | 440 | 615 | 690 | 880 |
| Upstream of the Confluence with Ohop Lake | 13 | 364 | 507 | 572 | 724 |
| Downstream of the Confluence with Twenty-five Mile Creek | 10 | 334 | 465 | 524 | 663 |
| Upstream of the Confluence with Twenty-five Mile Creek | 2 | 71 | 97 | 109 | 135 |
| PUYALLUP RIVER | | | | | |
| At Mouth | 948 | 41,000 | 46,000 | 48,000 | 63,000 |
| At Confluence with White River | 484 | 27,500 | 38,600 | 43,500 | 55,100 |
| At Confluence with Carbon River | 186 | 12,200 | 16,800 | 18,600 | 22,600 |
| At Confluence with Kapowsin Creek | 138 | 10,600 | 15,200 | 17,300 | 22,400 |
| RODY CREEK | | | | | |
| 104th Street East | N/A | 16 | 23 | 26 | 33 |
| 84th Street East | N/A | 3 | 4 | 5 | 7 |
| Pioneer Way East | N/A | 28 | 39 | 44 | 56 |
| SALMON CREEK | | | | | |
| At State Highway 167 | 1.2 | 37 | 50 | 57 | 70 |
| At Cross Section H (Private Road) | 0.6 | 24 | 33 | 37 | 45 |

**Table 2. Summary of Discharges**

| Flooding Source and Location | Drainage Area (square miles) | Peak Discharges (cubic feet per second) 10- Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|---|
| SOUTH CREEK TRIBUTARY NO. 1 | | | | | |
| At Mouth | 6 | 109 | 150 | 168 | 210 |
| At Cross Section AD | 4 | 75 | 102 | 114 | 142 |
| At Cross Section AP | 1 | 26 | 35 | 38 | 48 |
| SOUTH CREEK TRIBUTARY NO. 2 | | | | | |
| At Mouth | 1 | 18 | 25 | 29 | 35 |
| At Limit of Detailed Study | 0.1 | 8 | 10 | 12 | 15 |
| SOUTH CREEK TRIBUTARY NO. 3 | | | | | |
| At State Highway 7 | 1 | 20 | 26 | 30 | 36 |
| SOUTH CREEK TRIBUTARY NO. 4 | | | | | |
| At Limit of Detailed Study | 2 | 46 | 62 | 70 | 86 |
| SOUTH PRAIRIE CREEK | | | | | |
| At Mouth | 91 | 6,200 | 8,600 | 9,700 | 12,100 |
| At Tributary Confluence | N/A | 5,600 | 7,800 | 8,700 | 10,900 |
| SPANAWAY CREEK | | | | | |
| Upstream of Morey Creek | 17.6 | 82 | 104 | 104 | 112 |
| Downstream of Morey Creek | 12.0[2] | 51 | 63 | 63 | 67 |
| SQUALLY CREEK | | | | | |
| At Mouth | N/A | 92 | 118 | 129 | 154 |

## Table 2. Summary of Discharges

| Flooding Source and Location | Drainage Area (square miles) | Peak Discharges (cubic feet per second) 10- Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|---|
| SWAN CREEK | | | | | |
| At RM 0.4 | 3.62 | 257 | 340 | 377 | 463 |
| At RM 2.1 | 2.93 | 205 | 271 | 299 | 363 |
| At RM 3.2 | 2.47 | 173 | 227 | 249 | 300 |
| At RM 3.8 | 2.33 | 166 | 220 | 243 | 297 |
| At RM 4.8 | 1.48 | 124 | 163 | 180 | 218 |
| TANWAX CREEK | | | | | |
| Downstream of Mud Lake Outlet | 14 | 270 | 375 | 425 | 535 |
| Upstream of Mud Lake Outlet | 9 | 196 | 271 | 306 | 384 |
| At 352nd Street East (Golden Road) | 6 | 143 | 197 | 222 | 278 |
| At State Highway 161 | 4 | 128 | 177 | 198 | 248 |
| UNNAMED CREEK | | | | | |
| At Silver Lake Outlet | 2 | 31 | 39 | 42 | 51 |
| WAPATO CREEK I | | | | | |
| At Culvert Inlet to Blair Waterway | 3.14 | 176 | 323 | 390 | 540 |
| At RM 3.7 | 2.36 | 144 | 273 | 327 | 470 |
| At RM 5.4 | 1.86 | 135 | 210 | 237 | 330 |
| At RM 7.6 | 0.5 | 52 | 81 | 91 | 125 |
| WAPATO CREEK II | | | | | |
| At Diversion Culvert to the Puyallup | 1.56 | 152 | 240 | 272 | 375 |
| At RM 0.6 | 1.38 | 90 | 139 | 157 | 210 |

## Table 2. Summary of Discharges

| Flooding Source and Location | Drainage Area (square miles) | Peak Discharges (cubic feet per second) 10- Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|---|
| WHITE RIVER | | | | | |
| At Mouth | 494 | 14,000 | 15,300 | 15,500 | 19,000 |
| At State Highway 410 in the City of Buckley | 427 | 13,800 | 14,800 | 15,500 | 17,200 |
| At Confluence with Greenwater River | 294 | 18,600 | 25,800 | 28,900 | 36,700 |
| Upstream of Confluence with Greenwater River | 217 | 13,500 | 18,700 | 20,900 | 26,400 |
| Upstream of Confluence with West Fork White River | 145 | 8,940 | 12,200 | 13,600 | 17,000 |
| WILKESON CREEK | | | | | |
| At State Highway 165 Bridge in Center of Wilkeson | 24.4 | 1,400 | 1,830 | 2,010 | 2,430 |
| WOODLAND CREEK | | | | | |
| Woodland Creek at SR 512 | N/A | 14 | 16 | 16 | 17 |
| Woodland Creek Upstream of 96th Street East | N/A | 19 | 21 | 22 | 24 |
| Woodland Creek at 84th Street East | N/A | 54 | 72 | 80 | 101 |
| Woodland Creek at Pioneer Way | N/A | 32 | 54 | 65 | 100 |

[2] Discharges Reflect Split Flow

**Table 3. Summary of Elevations**

| Flooding Source and Location | Elevation (Feet NAVD) 10-Percent-Annual-Chance | 2-Percent-Annual-Chance | 1-Percent-Annual-Chance | 0.2-Percent-Annual-Chance |
|---|---|---|---|---|
| American Lake | 238.5 | 239.0 | 239.1 | 239.6 |
| Bonney Lake | 612.0 | 612.1 | 612.2 | 612.3 |
| Chambers Bay | | | | |
| At Steilacoom | 12.9 | 13.3 | 13.4 | 13.7 |
| Debra Jane Lake | 568.2 | 568.9 | 569.3 | 569.4 |
| Gravelly Lake | 223.1 | 223.3 | 223.3 | 223.5 |
| Harts Lake | 351.6 | 351.9 | 352.0 | 352.2 |
| Kreger Lake | 534.6 | 534.9 | 535.0 | 535.1 |
| Lake Kapowsin | 586.8 | 587.4 | 587.6 | 588.1 |
| Lake Louise | 229.3 | 229.5 | 229.6 | 229.7 |
| Little Lake | 351.6 | 351.9 | 352.0 | 352.2 |
| Ohop Lake | 527.9 | 528.6 | 528.9 | 529.7 |
| Puget Sound | | | | |
| At Commencement Bay and Salmon Beach | 11.9 | 12.3 | 12.4 | 12.7 |
| At Day Island and Titlow Lagoon | 12.2 | 12.6 | 12.7 | 13.0 |
| At Nisqually Reach | 12.8 | 13.2 | 13.3 | 13.6 |
| At Dupont | 12.9 | 13.2 | 13.3 | 13.6 |
| At Steilacoom | 12.7 | 13.1 | 13.2 | 13.5 |
| Rapjohn Lake | 639.3 | 639.5 | 639.6 | 639.8 |
| Silver Lake | 610.4 | 610.6 | 610.7 | 610.9 |
| Snake Lake | 300.7 | 300.9 | 301.0 | 301.2 |
| Spanaway Lake | 326.8 | 327.2 | 327.4 | 327.7 |
| Steilacoom Lake | 212.3 | 213.2 | 213.6 | 214.6 |
| Tanwax Lake | 605.5 | 605.8 | 605.9 | 606.1 |
| Wapato Lake | 314.3 | 314.5 | 314.6 | 314.8 |

## 3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals. Users should be aware that flood elevations shown on the Flood Insurance Rate Map (FIRM) represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. Flood elevations shown on the FIRM are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

### Pre-Countywide Analyses

Water-surface elevations for floods of the selected recurrence intervals were computed through use of the USACE HEC-2 step-backwater computer program (Reference 36) for the Puyallup and Carbon Rivers. Water-surface elevations for all remaining detailed-study streams were computed through use of a combination of the USGS step-backwater computer program (Reference 37), culvert rating analyses (Reference 38), and computations of flow over roads (Reference 39).

Water-surface elevations for the Mashel River were computed using the USGS step-backwater computer program (Reference 37).

Cross sections used in the backwater analyses of the Puyallup and Carbon Rivers were developed by the USACE. The above-water sections were developed using aerial photographs taken in May 1980 at a scale of 1:4,800 (Reference 40).

Cross section data for the White River were obtained from aerial photogrammetry (Reference 41 and 42), supplemented by field measurement of below-water portions. Elevation data and geometry for one bridge and two concrete broad-crested weirs were obtained by field survey. Data used for the downstream half of the study reach were developed specifically for this study (Reference 41), while data used for the upstream half were developed previously by the USACE for use in a 1974 channel capacity study (Reference 42).

For the remaining detailed-study streams, cross section data for the backwater analyses were obtained by photogrammetry from aerial photographs taken in April 1976 at a scale of 1:4,800 and 1:9,600 (References 41, and 43) and in February 1974 at a scale of 1:4,800 (Reference 42), supplemented by field measurements used to develop the below-water sections. Elevation data and structural geometry for bridges, culverts, road overflow sections, and a few additional channel cross sections were obtained by field survey.

The validity of the computed peak elevations for Debra Jane Lake and Bonney Lake were checked using regional hydrologic relationships developed for estimating the differences between mean lake elevations and the 10-, 2-, 1-, and 0.2-percent-annual-chance peak elevations. The regional relationships are based on log-Pearson Type III data for gages on nine hydrologically similar lakes in western Washington (References 44- 46). Lake elevations at the time of aerial photography were used as means. Peak elevations computed using these relationships were in agreement with those determined hydraulically.

The validity of the profiles computed for the downstream reach of White River was checked using a profile based on high-water marks from the December 2, 1977, flood. Profiles developed for the upstream reach were verified using two computed profiles and a surveyed profile representing a known release of 10,300 cfs from Mud Mountain Dam, which were prepared by the USACE for their channel capacity study.

Analyses of the hydraulic characteristics of streams in the community were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along each stream studied in the community.

Hydraulic analyses of the shoreline characteristics of the flooding sources studied in detail were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along the shoreline.

Areas of coastline subjected to wave attack are referred to as coastal high hazard zones. The entire Puget Sound coastline at Dupont and Steilacoom was designated a coastal high hazard zone. Chambers Bay, which is protected from wave attack by a breakwater extending most of the way across its mouth, was not designated as a coastal high hazard zone.

Water-surface elevations of floods for the selected recurrence intervals were computed through use of a combination of the U.S. Geological Survey step-backwater computer program (Reference 37), culvert rating analyses (Reference 38), and computations of flow over roads (Reference 39).

Cross section data for streams in the area were obtained from aerial photogrammetry (Reference 47), supplemented by field measurement of below-water portions. Elevation data and geometry for culverts, road overflow sections, and a few additional channel cross sections were obtained by field survey.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed (Section 4.2), selected cross-section locations are also shown on the FIRM.

Mean higher-high tide elevations at the mouth were used as starting water- surface elevations for the Nisqually and Puyallup Rivers, Wapato Creek I, and Hylebos Creek.

For Spanaway, Muck, and Lacamas Creeks, starting water-surface elevations were determined from culvert rating curves using approach sections. The design elevation at the upstream opening of the diversion culvert was used as the starting water-surface elevation for Wapato Creek II.

Starting water-surface elevations for Clover and Unnamed Creeks, and the Carbon River were calculated using the slope-area method.

Starting water-surface elevations for the Wapato Creek flood profiles presented in this study were determined from flood profiles for the 0.65-mile reach of Wapato Creek between its mouth and the corporate limits of Fife. The mouth of Wapato Creek is at Commencement Bay near the southeastern tip of Port Industrial Waterway. The mean-higher-high-tide elevation of 12.4 feet at Commencement Bay was used as the starting water-surface elevation for computations at the mouth of Wapato Creek.

The mean tide elevation of Puget Sound at Gig Harbor was used as the starting water-surface elevation at the mouth of North Creek for hydraulic computations.

Starting water-surface elevations were determined by profile convergence from downstream cross sections for the Mashel River; Chambers, Tanwax, and South Creek; the upstream reaches of the White River; Wilkeson, Fennel, and Debra Jane Creek; and Bonney Lake Outflow.

Starting water-surface elevations for step-backwater analysis on the Puyallup River were computed using the profile convergence method. Flooding on Clarks Creek was determined to be coincident with flooding on the Puyallup River; therefore, the elevations at the confluence were used as the starting water-surface elevations.

For the remaining streams, the 1-percent-annual-chance flood elevations were assumed to coincide with those on the main stems; therefore, the water-surface elevations in the mainstream channels were used for the starting water-surface elevations for the tributaries.

Flood profiles were drawn showing computed water-surface elevations to an accuracy of 0.5 foot for floods of the selected recurrence intervals.

The water-surface elevation computer for Muck Creek (at Roy) for a 10-percent-annual-chance flood at the location of streamflow gaging station no. 12090200 was verified by the stage-discharge relationship in use when the gage was discontinued in 1976. As that relationship extends to approximately a 5.9-percent-annual-chance flood, the highest recorded

peak at the gage, it could not be used to verify the water-surface elevations computed for the 2-, 1-, and 0.2-percent-annual-chance floods.

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the Flood Profiles (Exhibit 1) are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

The acceptability of all assumed hydraulic factors, cross sections, and hydraulic structure data was verified by computations that duplicated the profiles of the February 1972 flood from the Nisqually River and the December 1977 flood from the Puyallup and White Rivers.

Generally, the distances on the flood profiles correspond to distances measured along the centerline of the designated watercourses. In several areas, however, the meandering nature of the low-flow streambeds necessitated use of distances measured along the centerline of the 1-percent-annual-chance flow paths. On the maps, these flow lines, used to establish the respective profile distances, are delineated and labeled as Profile Base Lines.

Since the Puyallup and Carbon River levees are not considered as providing 3 feet of freeboard with respect to the 1-percent-annual-chance flood, water-surface elevations were computed for two cases. In the first case, flood elevations were computed before levee overtopping begins, assuming the levees remains intact. In the second case, floods were computed after overtopping occurs, assuming the levees had failed.

Profiles labeled “Puyallup River – With Consideration of Levee” and “Carbon River with Consideration of Levee” represent elevations inside the levees under the assumption that the levees do not fail.

Channel roughness factors (Mannings “n”) used in the hydraulic computations were chosen by nhc. The Manning’s “n” values for all detailed studied streams are listed in Table 4, Manning’s “n” Values.

**Table 4. Manning's "n" Values**

| Stream | Channel "n" Range | Overbank "n" Range |
|---|---|---|
| Artondale Creek | 0.030 - 0.040 | 0.025 - 0.100 |
| Bonney Lake Outflow | 0.040 - 0.060 | 0.050 - 0.070 |
| Canyon Creek | 0.035 - 0.065 | 0.035 - 0.075 |
| Carbon River | 0.030 - 0.034 | 0.040 - 0.150 |
| Clarks Creek | 0.038 - 0.040 | 0.040 - 0.150 |
| Clover Creek | 0.030 - 0.150 | 0.030 - 0.150 |
| Crescent Creek | 0.040 - 0.055 | 0.040 - 0.150 |
| Debra Jane Creek | 0.040 - 0.060 | 0.050 - 0.070 |
| Diru Creek | 0.015 - 0.065 | 0.02 - 0.12 |
| East Fork of Clear Creek | 0.055 - 0.075 | 0.065 - 0.12 |
| Fennel Creek | 0.040 - 0.060 | 0.040 - 0.150 |
| Hylebos Creek | 0.040 - 0.050 | 0.040 - 0.060 |
| Hylebos Creek Tributary | 0.040 - 0.050 | 0.040 - 0.060 |
| Lacamas Creek | 0.035 - 0.060 | 0.050 - 0.130 |
| Leach River | 0.040 - 0.045 | 0.045 - 0.050 |
| Mashel River | 0.055 - 0.060 | 0.080 - 0.150 |
| Morey Creek | N/A | N/A |
| North Creek | 0.045 - 0.060 | 0.045 - 0.060 |
| North Fork Clover Creek | 0.030 - 0.150 | 0.030 - 0.150 |
| Puyallup River | 0.027 - 0.050 | 0.030 - 0.120 |
| Rody Creek | 0.035 - 0.050 | 0.035 - 0.075 |
| South Prairie Creek | 0.035 - 0.037 | 0.040 - 0.150 |
| Spanaway Creek | N/A | N/A |
| Swan Creek | 0.040 - 0.045 | 0.050 - 0.100 |
| Wapato Creek | 0.040 - 0.070 | 0.050 - 0.150 |
| White River | 0.037 - 0.050 | 0.050 - 0.085 |
| Wilkeson Creek | 0.040 - 0.045 | 0.045 - 0.055 |
| Woodland Creek | 0.015 - 0.065 | 0.02 - 0.12 |

Profiles labeled “Puyallup River – Without Consideration of Levee” and “Carbon River without Consideration of Levee” represent overbank elevations that would occur if the levees did not exist.

The hydraulic analyses of North Creek indicate that the capacity of the culvert downstream of North Harborview Drive is insufficient for large floods and will cause ponding upstream. Furthermore, for floods having a 2-percent-annual-chance flood or greater, pond elevations will overtop North Harborview Drive near its intersection with Harborview Drive. Overflow of North Creek at North Harborview Drive will cause shallow flooding, less than 1 foot, of the commercially developed area between there and the Gig Harbor shoreline.

Shallow flooding areas were determined by field investigations and interpretation of aerial photographs (Reference 40). They were also determined by engineering judgment and by historical flood information provided by the City of Puyallup.

The 1-percent-annual-chance flood elevations for creeks studied by approximate methods were established according to the professional judgment of engineers, taking into account flood elevations estimated from available data, existing hydrologic and hydraulic analyses, correlations with similar streams, and field observations.

**Initial Countywide Analyses (TBD)**

The following streams were newly studied or re-studied for this initial countywide FIS.

*Artondale Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 94).*

Forty-nine cross-sections are used in the HEC-RAS model to represent conditions along the study reach. Many of the cross-sections were surveyed by nhc in March 2004, however some were digitally synthesized. Each surveyed cross-section typically only included the stream channel from bank to bank. The floodplain was not surveyed; therefore, the floodplain was added to each cross-section using digital topographic data provided to nhc by Pierce County. The topography was developed by Triathlon Ltd. using aerial photographs taken in 1998.

It is important to note that none of the cross-sections on the East Branch of Artondale Creek were surveyed; rather each was developed from the 1998, 2-foot topography. After developing the cross-sections in this reach, the horizontal location of the canyon invert was compared with the 2002, infrared orthophotos of Pierce County. The comparison revealed that the contours in the canyon were significantly incorrect, both horizontally and vertically. Using a handheld GPS unit, it was determined that the

centerline of the East Branch was actually positioned an average of 80 feet east of where the contours locate it; and in some locations as far as 200 feet east. A channel survey in this canyon would have been difficult to obtain due to its remoteness and extremely dense vegetation; and unnecessary because the 100-year discharge of 72 cfs combined with the relatively wide canyon bottom clearly placed none of the homes east of 72nd Ave NW at risk. Therefore the cross-sections in this reach are based on field notes and engineering judgment. Though BFEs are identified in this reach, they are not as accurate as those on the main stem or West Branch tributary.

Three culverts and five bridges influence hydraulic conditions within the study reach. The culvert crossings are at Wollochet Drive NW, Artondale Drive NW, and within the Gig Harbor Golf and Country Club. The Artondale Drive NW culvert was replaced in September 2004 and the new culvert is included in the HEC-RAS model. The five bridges in the model are driveways to private residences or to the Golf and Country Club. All culverts and bridges were surveyed by nhc to obtain elevation data and structural geometry for input into the hydraulic model.

In accordance with FEMA guidelines, the starting water surface elevation for the backwater model was set equal to the Mean Higher High Water elevation (MHHW) of Commencement Bay, Washington. The Mean Higher High Water elevation, as published by the National Oceanic and Atmospheric Administration, for station 9446484 “Tacoma, Commencement Bay” is equivalent to 5.86 feet NGVD29.

Channel and overbank roughness factors (Manning’s “n” values) used in the hydraulic computations were chosen using engineering judgment and were based on field observations, orthophotos, and published data. The “n” values for the main channel of Artondale Creek range from 0.03 to 0.04. Overbank “n” values range from 0.025 on the golf course to 0.1 in the thick brush of the wetlands.

On October 20, 2003, a flood event with a return period of approximately 35-years occurred on Artondale Creek. Ten high water marks (HWMs) were placed by nhc at the peak of the flooding. Eight of the HWMs were located on the main stem of Artondale; two were located on the West Branch tributary; and no HWMs were located on the East Branch. The peak stream discharge was unable to be determined precisely using the Pierce County operated stream gage at 6222 Artondale Drive NW because the water stage exceeded the gage rating table and was influenced by the driveway bridge low chord. However, estimates put the discharge at approximately 170 cfs. Using this flow, the HEC-RAS model of Artondale Creek was calibrated to the HWMs.

In September 2004, Pierce County replaced the 54-inch culvert under Artondale Drive NW with a slightly larger 3-sided culvert that reduces upstream flooding and is less of a barrier to fish. This culvert significantly influenced the HWMs that were placed in the October 2003 event.

Therefore, the calibration geometry and HWMs are not representative of the current condition geometry, which includes the recently replaced culvert.

*Canyon Creek Study*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 95).*

The HEC-RAS model for the Canyon Creek uses 174 cross-sections to represent the channel and \Cross-sections were developed from a combination of data sources. The floodplain information for each section was developed from Pierce County topographic data; a field survey (August 2004) was used for the channel information.

Dimensions of hydraulic structures were determined during the field survey. Pierce County provided digital topography created by Triathlon Inc. using aerial photographs taken in 1998. These contours were at 2-foot intervals and provided sufficient detail to estimate the extent of the floodplain.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or reviewed studies was the National Geodetic Vertical Datum of 1929 (NGVD 29). With the finalization of the North American Vertical Datum of 1988 (NAVD 88), most studies are being prepared using NAVD 88 as the referenced vertical datum. Typically, NAVD 88 elevations are approximately 3.5 feet higher than NGVD 29 elevations; precise conversion factors between the two systems can be obtained from the National Geodetic Survey's VERTCON utility (NGS, 1994). This study was initially prepared using NGVD 29 as the reference datum for the hydraulic modeling and base map preparation. However, the final Flood Insurance Rate Maps (FIRMs) and flood profiles have been converted to NAVD 88.

Numerous culverts are located along the studied reach on Canyon Creek, primarily at driveways and road crossing. All of the culverts identified in the field or from the County's public drainage inventory were coded into the HEC-RAS model. The hydraulic model includes 33 culverts. Since access to private property was limited, some private culverts in the study area may not have been identified. Where access was granted, culvert dimensions and elevations were surveyed as part of the June 2004 survey.

Roughness values ('n') used for modeling the culverts were chosen based on field observations and engineering judgment. Values depended on the culvert construction material and ranged from 0.013 for concrete culverts to 0.024 for corrugated metal pipe (CMP) culverts.

Manning's roughness coefficients for channels, based on field observations and engineering judgment, ranged from 0.035 to 0.065. The highest roughness values are for the heavily overgrown upper watershed where the stream is intermittent and the channel is vegetated. Overbank roughness values ranged from 0.035 to 0.075.

The downstream boundary of the Canyon Creek study area is the point where the creek enters a steep narrow ravine. Since no backwater effects are expected in this area due to the steep drop in elevation, the starting water surface elevation was assumed to be normal depth for all flood profiles.

No calibration data was available for the Canyon Creek model. Interviews with long-time residents and flooding problems documented in the *Clear/Clarks Creek Basin Plan* (Reference 93) provided anecdotal information that was used as a check for the model. Without specific calibration data it is impossible to judge the accuracy of the water surface profiles. However, considerable care was taken to examine the modeling results for reasonableness based on the information that was available.

In general, the HEC-RAS model results came close to reproducing the previously observed flooding problems on the creek. A comparison of actual flood elevations with flood elevations predicted by the model is not possible due to the lack of information. The modeling results confirmed several observed instances of flooding. At locations where the model did not match flooding, engineering judgment was used to determine if flooding would likely occur based on the existing culvert and channel configurations. The actual cause of some of the flooding problems could be blockages or maintenance issues that are not accounted for in this study. In addition, some previously recorded flooding problems may have been resolved.

*Carbon River Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 96).*

A HEC-RAS model was created to define the hydraulic characteristics of the 8.4-mile Carbon River study reach.

To determine the worst case flooding, four separate scenarios were modeled: (1) with consideration of levees; (2) without consideration of left levee; (3) without consideration of right levee; and (4) without consideration of both levees. Flooding in the vicinity of Voight Creek was modeled separately using the combined Carbon overbank split flow and local Voight Creek flow.

Scenario 1 modeled the 10-, 2-, and 1-percent-annual-chance discharges, and 1-percent-annual-chance BFEs were determined for the

river side of the levees. For cases where the 1-percent-annual-chance flood elevation was greater than the top of the levee elevation, the BFE was simply extended to the unprotected side of the levee, in accordance with FEMA, 2003.

Scenario 2 was split into two separate scenarios: failure of the left levee upstream of the bridges and failure of the left levee downstream of the bridges. For the upstream levee failure, floodwaters are expected to flow south between cross-sections V and W, overtop 177th Street E, and continue flowing in a southwestern to western direction. Since this flow is kept separate from the Carbon River by the high ground of the abandoned railroad grade, it was modeled as a split channel with cross-sections extracted from the topographic data. At cross-section SG of the split channel, a portion of the flow will pass through an opening in the abandoned railroad grade and reenter the Carbon River system; the remaining flow will enter Voight Creek, which will then enter the Carbon River at cross-section M. Voight Creek was also modeled as a separate reach with cross-sections obtained from the topographic data. This approach of separately modeling the upstream and downstream left levee failures produces the worst-case water levels in the left floodplain. The 1-percent-annual-chance discharge was modeled for this scenario and BFEs were determined for the left floodplain of the Carbon River, the split channel, and Voight Creek.

The right levee for the entire study reach was failed to model the 100-year discharge for scenario 3. To further define the right floodplain upstream of the bridges, between cross-sections W and X, four additional cross-sections, based on the bounding cross-sections and the 1998 topographic data, were included for this scenario. Upstream of the bridges, water will flow out of the Carbon River, over the abandoned railroad grade and State Route 162, and then into South Prairie Creek, which will then reenter the Carbon River at cross-section R. South Prairie Creek was modeled as a tributary to the Carbon River using the four most downstream cross- sections from the South Prairie Creek model, which was also developed by nhc. This scenario resulted in right floodplain BFEs along the Carbon River and along the lower portion of South Prairie Creek.

For scenario 4, both the left and right levees were failed along the entire study reach to produce the 0.2-percent-annual-chance inundation limits. The hydraulic analyses for this study were based on unobstructed flow.

For Scenario 1 (with consideration of both levees), a total of thirty-one cross-sections were used in the HEC-RAS model to represent conditions along the study reach. Twenty-seven cross-sections were surveyed by W & H Pacific in spring 2001 and four cross-sections were estimated from adjacent surveyed sections and topographic data. In general, the surveyed cross-sections consist of only channel and levee information.

The 1998 digital topographic data was used to extend the cross-sections into the floodplain.

For Scenario 2, without consideration of left levee upstream of the bridges, cross-sections for the split channel and Voight Creek were estimated based on the digital topographic data

Two bridges influence hydraulic conditions within the study reach. The bridges include the State Route 162 bridge located between cross-sections S and T and the abandoned railroad bridge located between cross-sections U and V. Both bridges were surveyed by W & H Pacific to obtain elevation data and measured by nhc to establish the structural geometry for input into the hydraulic model. The structural geometry and elevation data of the State Route 162 bridge were confirmed by bridge plans, which were obtained from the Washington Department of Transportation.

In accordance with FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2003), the starting water surface elevation was estimated by assuming coincident peaks with the Puyallup River. The assumption of coincident peaks on the Carbon and Puyallup Rivers is appropriate, as the following requirements are true: the ratio of the drainage areas lies between 0.6 and 1.4; the times of the peak flow are similar for the two combining watersheds; and the likelihood of both watersheds being covered by the modeled storm is high. Thus, the starting water surface elevation was taken to be the water surface elevation on the Puyallup River, at the Carbon River confluence, for the appropriate return period flood and levee consideration.

For Scenario 1, with consideration of levees, the water level on the Puyallup River with the consideration of both levees was used as the starting water surface elevation.

The left levee of the Carbon River adjoins the right levee of the Puyallup River upstream of the confluence. Thus, for Scenario 2, without consideration of the left levee, the adjoining levee on the Puyallup River should not be considered. However, since flow in the Puyallup is sub-critical, the right bank levee along the Puyallup River upstream from the confluence does not affect water levels in the Carbon River. Therefore, the water level on the Puyallup for the case with the levees in place was used as the starting water surface elevation.

The right levee of the Carbon River adjoins the right levee of the Puyallup River downstream of the confluence. For Scenario 3, without consideration of the right levee, the adjoining right levee on the Puyallup River should also not be considered. The starting water surface elevation corresponds to the case where the right levee on the Puyallup River, downstream of the confluence, is not considered.

For Scenario 4, without consideration of levees, the starting water surface elevation corresponds to the case where the right levee on the Puyallup River, downstream of the confluence, is not considered.

Eight high water marks along the Carbon River were set and surveyed by USGS following the major flood event in February 1996 (50-year return period). Of the eight surveyed high water marks, only the five most downstream were usable for calibration. The three most upstream high water marks were not used in the calibration procedure, as their location along the length of the river (river station) was deemed to be unreliable. Of the five high water marks that were used, the elevation accuracy (as estimated by USGS) ranged from 0.05 foot to 0.25 foot and the river station accuracy was estimated by nhc to be approximately 50 feet.

Initial channel and floodplain roughness factors (Manning's "n" values) were estimated based on field observations and engineering judgment. To calibrate the hydraulic model, these initial roughness factors were adjusted, within engineering reason, until the computed water surface elevations matched the high water mark elevations as close as possible within 0.6 feet.

*Clarks Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 97).*

The HEC-RAS model for Clarks Creek and Meeker Ditch includes 79 cross-sections; 57 on the former and 22 on the latter. These cross-sections serve to represent the geometry of the channel and floodplain along the study reach. Channel cross-sections, as well as bridge and culvert structures, were surveyed by Parametrix in 2004 for the City of Puyallup (Parametrix, 2005). In reaches where access to the channel was unavailable, channel cross-sections were interpolated to augment the surveyed data. Interpolation occurred at 4 locations. Topographic data provided by Pierce County, the City of Puyallup, and the USGS in the vicinity of Clarks Creek was used to extend both the surveyed and interpolated channel cross-sections to include the floodplain. This collection of topographic data was used to build a triangulated-irregular-network (TIN) of the surrounding area, and a GIS tool developed by the Army Corps of Engineers (HEC-GeoRAS) was used to measure cross-section geometry.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. The initial hydraulic analysis for the Clarks Creek FIS was conducted using the NGVD29 vertical datum; however, the vertical datum was adjusted to NAVD88 for the final submission. Specifically, NAVD88 elevations along the Clarks Creek study reach can be converted to NGVD29 elevations by subtracting 3.49 feet.

The hydraulic analysis of the Clarks Creek study reach includes 12 bridge crossings while the Meeker Ditch study reach includes 2 bridge and 6 culvert crossings. Within the HEC-RAS model, all culverts were modeled using the Highest Upstream Energy method, while bridges were modeled either with the Energy, or Pressure and/or Weir methods, depending on level of inundation.

The downstream limit of the Clarks Creek HEC-RAS model is located at the confluence with the Puyallup River. Due to the nature of flow conditions at the confluence it was determined that the normal depth starting water surface elevation typically recommended by FEMA guidelines (FEMA, 2003) would not be appropriate. Starting water surface elevations assuming coincident 1-percent-annual-chance peaks would also not be appropriate primarily because the vast difference in basin sizes between the Puyallup River and Clarks Creek would make it physically unrealistic. As an alternative the National Service Provider (Michael Baker Jr. Inc.) recommended using the 10-percent-annual-chance water surface elevation (25.0 feet, NGVD29) on the Puyallup River as a reasonable estimate of the starting water surface elevation for Clarks Creek. This elevation is based on “with levee” scenario on the Puyallup River. Another “without levee” scenario was used to define the overbank Puyallup BFEs in the vicinity of Clarks Creek because the levees on the Puyallup River failed to meet certification requirements (nhc, 2004). However, because the initial studies for different flooding sources are typically conducted independent of one another the failure of a levee on the Puyallup do not directly influence BFEs determined for the Clarks Creek FIS. In this case, the final inundation limits and BFE locations in the vicinity will be determined by merging the results from both the Puyallup and Clarks studies during the final preparation of the County-wide FIRM. Thus, for the initial study of Clarks Creek the selection of a starting water surface elevation using “with levee” conditions on the Puyallup was deemed suitable. This value was adopted by nhc for all flood profiles evaluated on Clarks Creek.

Channel and overbank roughness factors (Manning’s “n” values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the creek and floodplain areas. The channel “n” values for the main channel of Clarks Creek typically range from 0.038 to 0.040 (values as high as 0.10 were used in the upper reach of Meeker Ditch). Overbank “n” values range from 0.04 to 0.15. No reliable high water marks are available to calibrate the Clarks Creek model.

*Clear Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report, MONTH, YEAR (Reference 98).*

Conventional HEC-RAS modeling techniques were not used for this FIS because the flooding is the result of interior drainage ponding rather than stream capacity/conveyance limitations. See the above nhc report for more information.

*Clover Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report, MONTH, YEAR (Reference 99).*

Four hundred three cross-sections are used in the HEC-RAS model to represent conditions along the main stem, North Fork and five overflow reaches. In-channel data was surveyed for 244 cross-sections, while 80 of the remaining were interpolated or copied, and 79 synthesized from Light Detection and Ranging (LiDAR) data. Of the 244 surveyed cross-sections, 156 were surveyed by nhc in 2002, 55 were obtained from an existing U.S. Army Corps of Engineers (USACE) HEC-2 model (USACE, 2000), and 33 were extracted from the E431 model created in 1980 for the original FIS study (FEMA, 1987). The floodplain portion of each cross section was obtained from 1:12,000 scale bare-earth-LiDAR data provided by Puget Sound LiDAR Consortium (PSLC, 2004). The LiDAR data were developed by TerraPoint LCC from flights performed in August 2004 and are reported to have a nominal vertical accuracy of one foot. Utilities within HEC-GEORAS 3.1 (USACE, 2002) were used to cut the topographic data from the maps and add it to the in-channel data

Numerous hydraulic structures, sixty-three in total, influence hydraulic conditions within the main stem and North Fork study reaches. Of these 61 structures, 51 are bridges, 10 are culverts, and 5 are in-line weirs. Several of these crossings occur at major highways or arterials, such as Interstate 5, Pacific Highway South, and Pacific Avenue. One unique structure worth mentioning consists of a pair of 12-foot diameter, 1830-foot long corrugated metal pipes that carry Clover Creek beneath the runway at McCord Air Force Base. Except for the culverts at McCord, all bridges and culverts were surveyed by nhc to obtain elevation data and structural geometry for input into the hydraulic model. The McCord culverts were surveyed by the Seattle District of the U.S. Army Corps of Engineers.

At five locations along the study reach, water leaves the channel and follows a separate flow path. A description of each split flow reach is provided below, starting with the one that is the furthest upstream.

The first split flow reach, designated “Golf Course”, develops when water overtops the right bank along the Brookdale Golf Club. This water flows across the golf course and eventually joins the North Fork. The flow split is defined in the HEC-RAS model as a separate reach with junctions at approximately river mile 8.5 on the main stem and river mile 0.9 on the North Fork. Split flow from the left bank of North Fork also joins this reach.

This reach does not convey flow during the 1-percent-annual-chance existing conditions event.

The second flow split, designated “Parkland”, begins about 1.3 miles downstream in the vicinity of 4th Avenue East. Here, water overtops the left bank and flows to the northwest through an old historical swale. During both the 1-percent-annual-chance and 0.2-percent-annaul-chance flooding events flow will pass over Pacific Avenue and rejoin the main stem near 10th Avenue South. The flow split is defined in the HEC-RAS model as a separate reach that connects to the main stem via a lateral weir at river mile 6.7 and junction at river mile 5.2.

The third flow split, designated “134th Avenue”, begins on the right bank of North Fork Clover Creek at the B Street Crossing. The reach rejoins Clover Creek a short 900 feet later just upstream of the mainstem crossing of 134th Street East.

The fourth flow split, designated “Pacific Avenue”, parallels the main stem of Clover Creek along its left bank between 134th Street East and Park Avenue South. The flow split is defined in the HEC-RAS model as a separate reach connected by two junctions at river miles 5.6 and 6.0.

The fifth flow split, referred to as the “Lakewood Split”, begins immediately downstream from Interstate 5. Here, water overtops the right bank near Cochise Lane Southwest and flows to the north toward 112th Street Southwest. During the modeled 1-percent-annual-chance flood, it appears that the water will not pass beyond 112th Street Southwest, but during the modeled 0.2-percent-annual-chance event, it will pass over 112th and continue north toward the Lakewood Towne Center. The flow split is defined by a series of lateral weirs, located along the right bank in the vicinity of Cochise Lane Southwest. Based on topographic data and anecdotal information, floodwaters in the Lakewood Split do not rejoin Clover Creek, but instead travel north toward the Lakewood Towne Center and Ponce de Leon Creek (City of Lakewood, 2003).

To model the hydraulic interaction of these reaches, the split flow optimization option was selected within HEC-RAS at the reach junctions and lateral weirs. This allowed the computer model to estimate the magnitude of each flow split by internally balancing water surface and/or energy values, these values are ‘hard coded’ into the hydraulic model used for floodplain simulations.

The starting water surface elevation for the backwater model was assumed to be 213.42 feet NAVD88 (210 feet NGVD29), consistent with the 1-percent-annual-chance flood elevation shown on the effective FEMA FIRM for Steilacoom Lake.

The model could not be thoroughly calibrated along its entire reach because reliable high water marks are not available. However, in several areas water surface elevations and inundation limits for the February 1996 and January 1997 floods were estimated from photographs and anecdotal eyewitness accounts. In general, channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen by engineering judgment and based on field observations of the creek and floodplain areas. The channel "n" values for the main channel and overbank of both the main stem and North Fork range from 0.03 to 0.15.

*Crescent Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 100).*

The HEC-RAS model for Crescent Creek includes 83 cross-sections. These cross-sections serve to represent the geometry of the channel and floodplain along the study reach. Since approximate methods were employed for the effective FIS study of Crescent Creek no previous cross-sectional data exists. As a result, 35 new channel cross-sections were surveyed along the study reach. At select locations, such as culvert or bridge crossings, surveyed channel cross-sections were copied upstream or downstream to represent similar channel geometry. This occurred at 21 locations. In reaches where access to the channel was unavailable, channel cross-sections were interpolated to augment the surveyed data and occurred at 27 locations. Topographic data provided by Pierce County, in the vicinity of Crescent Creek was used to extend these cross-sections to include the floodplain. The topographic data was developed from aerial photographs taken in 1998, with a contour interval of two feet.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised studies was the National Geodetic Vertical Datum of 1929 (NGVD29). With the finalization of the North American Vertical Datum of 1988 (NAVD88), most studies are being prepared using NAVD88 as the referenced vertical datum. The initial hydraulic analysis for the Crescent Creek FIS was conducted using the NGVD29 vertical datum; however, the vertical datum was adjusted to NAVD88 for the final submission. Precise conversion factors between the two vertical datums vary by location and can be obtained from the Army Corps of Engineers' CORPSCON utility (USACE, 1997). Specifically, NGVD29 elevations along the Crescent Creek study reach can be converted to NAVD88 elevations by adding 3.48 ft.

The hydraulic analysis of the Crescent Creek study reach includes 12 culvert crossings, 11 bridges, and 1 in-line weir structure. Within the HEC-RAS model, all culverts were modeled using the Highest Upstream

Energy method, while bridges were modeled either with the Energy, or Pressure and/or Weir methods, depending on level of inundation.

A split-flow reach and in-line weir structure were implemented between cross-sections 1 and 4 to model the main stem of Crescent Creek and expected overtopping of the roadway at Crescent Valley Drive, respectively.

The downstream boundary conditions for Crescent Creek consists of culvert structure, thus the normal depth starting water surface elevations recommended by FEMA (FEMA, 2003) were not implemented. Alternatively, a culvert structure was included in the HEC-RAS model and the culvert routine was used to determine headwater depths. To facilitate the computation, cross-section A was placed at the downstream end of the culvert structure within the Gig Harbor estuary. A fixed tailwater depth was then selected as the boundary conditions for the study reach. The mean higher high water tide level of 9.34 feet, NAVD 88 (5.86 feet, NGVD 29) was selected for all flood profiles modeled (USDC, 2003).

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the creek and floodplain areas. The channel "n" values for the main channel of Crescent Creek typically range from 0.04 to 0.055. Overbank "n" values range from 0.04 to 0.15. No reliable high water marks were available to calibrate the Crescent Creek model.

*Diru Creek Study*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 101).*

The HEC-RAS model for Diru Creek uses 147 cross-sections to represent the channel and floodplain. Cross-sections were developed from a combination of data sources. The floodplain information for each section was developed from Pierce County topographic data; a field survey (June 2004) was used for the channel information. In instances where the channel could not be surveyed, channel characteristics were estimated based on adjacent surveyed data and topographic mapping.

Manning's roughness coefficients for channels, based on field observations and engineering judgment, ranged from 0.015 to 0.065. A higher roughness value was generally used (from 0.055 to 0.065) to account for densely vegetated conditions in this intermittent stream. Overbank roughness values ranged from 0.02 to 0.12.

Pierce County provided digital topography created by Triathlon Inc. using aerial photographs taken in 1998. These contours were at 2-foot intervals and provided sufficient detail to estimate the extent of the floodplain.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or reviewed studies was the National Geodetic Vertical Datum of 1929 (NGVD29). With the finalization of the North American Vertical Datum of 1988 (NAVD88), most studies are being prepared using NAVD88 as the referenced vertical datum. Typically, NAVD88 elevations are approximately 3.5 feet higher than NGVD29 elevations; precise conversion factors between the two systems can be obtained from the National Geodetic Survey's VERTCON utility (NGS, 1994). This study was initially prepared using NGVD29 as the reference datum for the hydraulic modeling and base map preparation. However, the final Flood Insurance Rate Maps (FIRMs) and flood profiles have been converted to NAVD88.

Numerous culverts are located along the studied reach on Diru Creek, primarily at driveways and road crossing, but also at creek crossings in pastures. Culverts that were perceived to present significant controls on water surface elevations were coded into the HEC-RAS model. Two long culverts are located within the study reach: a 500-foot-long culvert just north of SR 512; and a 525-foot-long culvert along 66th Avenue East just south of 104th Street East. The hydraulic model includes 26 culverts. Since access to private property was limited, some private culverts in the study area may not have been identified. Where access was granted, culvert dimensions and elevations were surveyed as part of the June 2004 survey.

Roughness values ('n') used for modeling the culverts were chosen based on field observations and engineering judgment. Values depended on the culvert construction material and ranged from 0.011 to 0.013 for concrete culverts; corrugated metal pipe (CMP) culverts were given an 'n' value of 0.024.

Lateral weir structures were coded into the HEC-RAS model to estimate flow splits at culvert locations. Flow splits were defined for locations where predicted flow would overtop a culvert and the adjacent bank height is not sufficient to contain the flow. One lateral weir was used to estimate a split flow at the culvert just downstream of SR 512. Several lateral weirs were used to represent split flows at culverts along 66th Avenue East upstream of 96th Street East.

The downstream boundary of Diru Creek for this study is where the creek enters a steep narrow ravine. Since no backwater effects are expected in this area due to the steep drop in elevation, the starting water surface elevation was assumed to be normal depth for all flood profiles.

No calibration data was available for Diru Creek. Interviews with long-time residents and flooding problems documented in the *Clear/Clarks Creek Basin Plan* (Reference 93) provided anecdotal information that was used as a check for the model. Without specific calibration data it was not

possible to judge the accuracy of the water surface profiles. However, considerable care was taken to examine the modeling results for reasonableness based on the information that was available.

The hydraulics in Diru Creek are complex, and the creek overtops its banks and flows overland at several locations. Two split flow reaches have the potential to convey a significant amount of flow and were included in the HEC-RAS model:

- The split flow reach east of 66th Avenue East between Pipeline Road and 96th Street East is created as the creek overtops its banks at several driveway culverts and flows to the west. The overflow path is west through residential property, with flow collecting in a low spot behind the homes. The split flows continue north and rejoin the creek just north of 96th Street East.

- The 104th Street Overflow is north of SR 512, where the creek overtops a 12-inch culvert behind a residence. The split flow follows the edge of the property in a man-made ditch and passes under 66th Avenue East in a 12-inch culvert, entering a ditch on the east side of the road. This ditch runs north along 66th Avenue East, then east along 106th Street Court East. The split flow crosses 106th Street Court East through a culvert and flows north, rejoining Diru Creek at 104th Street East.

Due to the limitations of the HEC-RAS model in capturing the complexity of the hydraulics, flow splits were not included in the hydraulic model for two other locations, where the amount of flow involved appears to be less significant:

- According to residents, water can overtop the backyard ditch in the upper portion of the 104th Street Overflow. According to the hydraulic modeling and accounts from local residents, water overtops the north bank of the ditch and flows into the front yard of the house to the north of the creek. This overflow appears to be relatively small and flooding is less than 1 foot. This area was mapped, but is not included in the hydraulic model.

- According to local residents, the long culvert on 66th Avenue East often is overtopped, and water flows east down the hillside and into the lower portion of the 104th Street Overflow. This 12-inch culvert has a trash rack which, according to residents, can often be blocked by debris. It wasn’t clear if this blockage causes the flooding or if the culvert is undersized. Regardless, this appears to be a local drainage problem and was not included in the hydraulic modeling.

*East Fork of Clear Creek*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 102).*

The HEC-RAS model for the East Fork of Clear Creek uses 71 cross sections to represent the channel and floodplain. Cross sections were developed from a combination of data sources. The floodplain information for each section was developed from Pierce County topographic data; a field survey (August 2004) was used for the channel information. On occasion, the channel was inaccessible due to private property restrictions or the density of vegetation. In instances where the channel could not be surveyed, channel characteristics were estimated based on adjacent surveyed data and topographic mapping.

Dimensions of hydraulic structures were determined during the field survey. Pierce County provided digital topography created by Triathlon Inc. using aerial photographs taken in 1998. These contours were at 2-foot intervals and provided sufficient detail to estimate the extent of the floodplain.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or reviewed studies was the National Geodetic Vertical Datum of 1929 (NGVD29). With the finalization of the North American Vertical Datum of 1988 (NAVD88), most studies are being prepared using NAVD88 as the referenced vertical datum. Typically, NAVD88 elevations are approximately 3.5 feet higher than NGVD29 elevations; precise conversion factors between the two systems can be obtained from the National Geodetic Survey's VERTCON utility (NGS, 1994). This study was initially prepared using NGVD29 as the reference datum for the hydraulic modeling and base map preparation. However, the final Flood Insurance Rate Maps (FIRMs) and flood profiles have been converted to NAVD88.

Numerous culverts are located along the studied reach on the East Fork and the West Stem, primarily at driveways and road crossing. All of the culverts identified in the field or from the County's public drainage inventory were coded into the HEC-RAS model. The hydraulic model includes 13 culverts. Since access to private property was limited, some private culverts in the study area may not have been identified. Where access was granted, culvert dimensions and elevations were surveyed as part of the June 2004 survey.

Roughness values used for modeling the culverts were chosen based on field observations and engineering judgment. Values depended on the culvert construction material and ranged from 0.011 to 0.013 for concrete culverts; corrugated metal pipe (CMP) culverts were given an 'n' value of 0.024.

Manning's roughness coefficients for channels, based on field observations and engineering judgment, ranged from 0.055 to 0.075. The highest roughness values occur in the heavily overgrown upper watershed where the stream is intermittent and the channel is vegetated. Overbank roughness values ranged from 0.065 to 0.12.

The downstream boundary of the East Fork of Clear Creek is located where the creek enters a step narrow ravine. Since no backwater effects are expected in this area due to the steep drop in elevation, the starting water surface elevation was assumed to be normal depth for all flood profiles.

No calibration data was available for the East Fork of Clear Creek. Interviews with long-time residents and flooding problems documented in the *Clear/Clarks Creek Basin Plan* (Reference 93) provided anecdotal information that was used as a check for the model. Without specific calibration data it impossible to judge the accuracy of the water surface profiles. However, considerable care was taken to examine the modeling results for reasonableness based on the information that was available. In general, the HEC-RAS model results came close to reproducing the previously observed flooding problems on the creek. The actual cause of some of the flooding problems could be due to blockages or maintenance issues that are not accounted for in this study.

*Fennel Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 103).*

Fifty-nine cross-sections are used in the HEC-RAS model to represent conditions along the study reach. Many of the cross-sections were surveyed by nhc in December 2001, however some were digitally synthesized.

Two bridges and three culverts influence hydraulic conditions within the study reach. The bridges include the Kelly Lake Road bridge, which marks the upstream boundary of the study reach, and the McCutcheon Road bridge. The culvert crossings are at Old Sumner Buckley Highway, State Route 410, and Rhodes Lake Road. All bridges and culverts were surveyed by nhc to obtain elevation data and structural geometry for input into the hydraulic model.

In accordance with FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2002), the starting water surface elevation for the backwater model was assumed to be normal depth. The assumption of coincident peaks with the Puyallup River did not meet the acceptance criteria in the FEMA guidelines (FEMA, 2002). Backwater flooding from the Puyallup River will influence the lowest portion of Fennel Creek, but it is left for FEMA's Map Maintenance Partner (Michael Baker Jr. Inc.) to determine the base flood elevations that should be shown in the final FIS for this reach.

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the creek and floodplain areas. The channel "n" values for the main channel of Fennel Creek range from 0.04 to 0.06, and the overbank "n" values from 0.04 to 0.15.

No reliable high water marks are available to calibrate the Fennel Creek model. Anecdotal high water mark information was obtained from three residents living adjacent to the stream. The water levels recalled by these individuals for a major flood in February 1996 are no more than 5 inches different in elevation than that predicted by the hydraulic model. Additional anecdotal information collected from County road crews describes flooding at the McCutcheon Road bridge. Road crews estimate the flood depth at the crown of McCutcheon Road to have been approximately 6 inches during the 1996 storm; however a resident who lived adjacent to the bridge remembers that the water was more than 12 inches deep over the road. The hydraulic model supports the resident's observation showing McCutcheon Road overtopping by 14 inches at its deepest location during the 1996 flood. The February 1996 storm was approximately a 1-percent-annual-chance event.

*Lacamas Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 104).*

The HEC-RAS model for this study includes 107 cross-sections to represent the geometry of the channel and floodplain along the study reach. Of these cross-sections, 50 were obtained directly from the USGS E431 model created for the original FIS of Lacamas Creek. These cross-sections were assembled in the late-1970's by the USGS through a combination of field surveying and topography information obtained from aerial photos (FEMA, 1987). To supplement the information from the previous study, nhc surveyed an additional 56 cross-sections. This usually occurred at road crossings (e.g. bridges and culverts), although some mid-reach sections were resurveyed to account for substantial channel changes that have occurred since the previous study. The topographic maps provided by Pierce County, in the vicinity of Lacamas Creek, have a contour interval of five feet, thus they were only used in locations where survey data was limited.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised studies was the National Geodetic Vertical Datum of 1929 (NGVD29). With the finalization of the North American Vertical Datum of 1988 (NAVD88), most studies are being prepared using NAVD88 as the referenced vertical datum. The initial hydraulic analysis for the Lacamas Creek FIS was

conducted using the NGVD29 vertical datum; however, the vertical datum was adjusted to NAVD88 for the final submission. Precise conversion factors between the two vertical datums vary by location and can be obtained from the National Geodetic Survey's (NGS) VERTCON utility (NGS,1994). Specifically, NGVD29 elevations along the Lacamas Creek study reach can be converted to NAVD88 elevations by adding 3.42 feet.

The hydraulic analysis of Lacamas Creek included 8 culvert crossings and 13 bridge structures. At least two other bridges were observed during field visits; however, they were left out of the analysis because it was assumed they had negligible effect on the stream flow. Within the HEC-RAS model, all culverts were modeled using the Highest Upstream Energy method, while bridges were modeled either with the Energy, or Pressure and/or Weir methods, depending on level of inundation. Skews were also applied to bridge or culvert crossings when deemed necessary.

The high road embankment and relatively low capacity of the culverts at State Route 507 result in significant overbank flow. When flooding occurs at this location on Lacamas Creek, it is diverted away from the main stem, and essentially acts as a separate overflow channel. To model this in HEC-RAS, a diversion reach was created. This reach begins on the main stem of Lacamas Creek, upstream of cross-section G, flows to the west and then north, crossing SR 507, and rejoining the main stem downstream of cross-section C. In total there are 6 cross-sections (labeled 1-6) and an inline weir defining the reach. As a conservative assumption, all diverted flow was returned to Lacamas Creek rather than continuing as overland flow directly to Muck Lake.

In accordance with FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2002), the starting water surface elevation for the backwater model was assumed to be normal depth. Backwater flooding from Muck Lake will influence the lowest portion of Lacamas Creek.

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the creek and floodplain areas. The channel "n" values for the main channel of Lacamas Creek range from 0.035 to 0.06, and the overbank "n" values from 0.05 to 0.13.

No reliable high water marks are available to calibrate the Lacamas Creek model. Anecdotal high water mark information was obtained from residents living adjacent to the stream, and Pierce County maintenance personnel. However, this information was qualitative in nature; therefore, it was only used as partial verification of model accuracy.

*Mashel River Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 105).*

The geometry of the river channel and floodplain are represented within the HEC-RAS model by 28 cross sections. The source of the channel and overbank geometry at each cross section is described below. It was not within the scope of this study to survey new cross sections along the study reach; therefore, 22 of the sections were taken from an existing E431 backwater model of the study reach. The E431 model was created by the USGS in the late 1970s for the original FEMA floodplain mapping study. Of the remaining six cross sections, five (cross sections K, L, U, V, and W) were surveyed by nhc in March and May 2002. The sixth (cross section M), located on the upstream side of the S.R. 161 bridge, is a copy of the cross section nhc surveyed at the downstream face with its elevations adjusted to account for the channel slope. Although the cross sections taken from the E431 model are over 20 years old, they appear to provide an acceptable representation of the current channel geometry at most locations along the study reach. This was determined through field inspections and by comparing several of the newly surveyed cross sections with sections measured at approximately the same location as those for the E431 study. There were two cross sections from the E431 model (labeled N and X in this study) where nhc adjusted the channel geometry using approximate methods. These adjustments were made to account for changes in the channel geometry that were identified by comparing the 1989/1994 and 1998 digital orthophotos and later confirmed by residents and public works officials during interviews.

NHC originally planned to use only the channel portion of each E431 cross section and to update the floodplain geometry using the 1998 topographic contour maps provided by Pierce County. However, this was not done because spot surveys and field inspections revealed that the five-foot contour lines were frequently unreliable. This coarse contour interval does not provide an accurate and/or detailed enough representation of the Mashel River floodplain, particularly in areas where there is dense vegetation. For example, on the left bank of the uppermost portion of the reach, contour lines differ by as much as 10 to 15 feet from the previous E431 overbank geometry. As a result, nhc used the floodplain geometry from the E431 model at most of the cross sections. Areas where the contour data differ significantly from the E431 data are noted on the work map. The contour lines had to be used to reconstruct one or both overbanks of several cross sections because changes have occurred at those locations since the E431 cross sections were surveyed. These cross sections (I, J, K, N, W, X) are identified and are located near the Eatonville Sewer Plant, the George Smallwood Park, and approximately 650 feet upstream of the Eatonville Cutoff Road.

Three bridges influence hydraulic conditions along the study reach. These include a private driveway bridge the State Route 161 bridge and the Eatonville Cutoff Road bridge. The geometry of the private driveway bridge was represented in the model using data from the E431 model. State Route 161 and Eatonville Cutoff Road bridges were surveyed by nhc. Within the HEC-RAS model, all bridges were modeled using the Energy method.

In accordance with FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2002), the starting water surface elevation for each profile was estimated using normal depth procedures. Normal depth at the downstream boundary of the model, cross section A, was calculated using an estimated energy slope of 0.01.

Channel and overbank roughness values (Manning's "n") used in the hydraulic model were estimated based upon engineering judgment and reference to classical publications such as Chow (1959), Barnes (1967); and Hicks and Mason (1991). The "n" values for the main channel range from 0.055 to 0.06, and for the overbank from 0.08 to 0.15.

No reliable high water marks are available to calibrate the Mashel River model. Anecdotal high water mark information was obtained from several sources including residents living adjacent to the river, Eatonville Public Works employees, and Pierce County officials. These individuals could remember flooding dating back to the 1960s as well as the recent February 1996 flood. Although precise high-water marks could not be determined based upon their eyewitness accounts, water levels and inundation limits could be approximated at several locations. This information, which is summarized in the Technical Data Support Notebook, was considered by nhc during the development of the HEC-RAS model.

*Morey Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 106).*

The HEC-RAS model for the study reach includes 34 cross-sections that define the geometry of the channel and floodplain. nhc surveyed the channel portion of 30 of these as part of the Spanaway Creek FIS in the spring of 2002 and this FIS in the fall of 2003. The four remaining sections were interpolated from the surveyed cross-sections. With the aide of the HEC GEO-RAS 3.1.1 (HEC, 2003) computer program, all 34 sections were extended to include the floodplain using topographic maps provided by the County. The maps, which were created by the NIES Mapping Group Inc. are based upon 1998 aerial photographs and have a contour interval of two feet. In several locations of dense vegetation, elevations on the topographic maps varied slightly from survey data (2 +/- feet). In these locations, floodplain elevations were adjusted using survey data and field observations.

Within the study reach, landowners have made numerous modifications to the stream by adding small pedestrian/auto bridges, diversion weirs, and floodplain ponds. During field surveys and reconnaissance, 5 bridges, 2 culverts and 4 inline weirs were noted within the study reach, all of these were coded into the HEC-RAS model. The most significant hydraulic structure is a concrete weir with a 10 foot drop at outlet from Morey Pond (see Photo 3). Two additional private ponds were noticed to have been

constructed in the floodplain adjacent to the stream. Both of these are separated from the channel by a low earthen berm and are connected to the stream by a small culvert or control. Within the HEC-RAS model, it was assumed that the portion of each pond below the top elevation of the berm would not convey flow during an overbank event. These areas were assumed to be “dead storage” and are coded as an ineffective area or are not included in the cross-section geometry.

In accordance with FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2003), the starting water surface elevation for Morey Creek was established by assuming critical depth downstream of the outlet from Morey Pond. Backwater flooding from Clover Creek will not influence Morey Creek above the Morey Pond outlet weir.

There have not been any quality high-water marks collected for any large historic events on Morey Creek but some data from a moderate event during the study and anecdotal information from a recent large event in 1996 provided some level of calibration. Initial estimates of Manning’s “n” values were made for the channel and floodplain based upon engineering judgment and reference to classical publications (Chow, 1959; Barnes, 1967; and Hicks and Mason, 1991). Water surface elevations were documented throughout the study area during the survey that began on October 21st, 2003 and ended on October 23rd, 2003. The survey began the day following the peak of an event that occurred at approximately 18:00 on October 20th, 2003. Though the event magnitude was significant for many streams in the area there was only minor flooding visible the day following the storm on Morey Creek. The discharge from the outlet of Morey Pond was estimated at 10 cfs using weir flow calculations on the morning of the October 21st, 2003. During the three days of surveying that followed the event the discharge appeared to decrease only slightly (min 5cfs). The initial Manning’s “n” values were modified slightly to match the water surface elevations surveyed between October 21$^{st}$ and 23rd, 2003. The 1996 event is the largest event in record. The anecdotal information from this event was used to verify that the model accurately predict inundation of the field to the east of 2004 138th Street South for the events modeled with discharges greater than the 1996 event.

*North Fork of Clover Creek Study*

See Clover Creek Study.

*North Fork of Clover Creek Tributaries Study*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 107).*

The HEC-RAS model for the North Fork Tributaries uses 174 cross-sections to represent the channel and floodplain. Cross-sections were developed from a combination of data sources. The floodplain information for each section was developed from Pierce County topographic data; a field survey (June 2004) was used for the channel information.

Manning's roughness coefficients for channels, based on field observations and engineering judgment, ranged from 0.03 to 0.04. A higher roughness value was generally used on overbanks to account for densely vegetated conditions. Overbank roughness values ranged from 0.02 to 0.10.

Pierce County provided digital topography created by Triathlon Inc. using aerial photographs taken in 1998. These contours were at 2-foot intervals and provided sufficient detail to estimate the extent of the floodplain.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or reviewed studies was the National Geodetic Vertical Datum of 1929 (NGVD29). With the finalization of the North American Vertical Datum of 1988 (NAVD88), most studies are being prepared using NAVD88 as the referenced vertical datum. Typically, NAVD 88 elevations are approximately 3.5 feet higher than NGVD29 elevations; precise conversion factors between the two systems can be obtained from the National Geodetic Survey's VERTCON utility (NGS, 1994). This study was initially prepared using NGVD29 as the reference datum for the hydraulic modeling and base map preparation. However, the final Flood Insurance Rate Maps (FIRMs) and flood profiles have been converted to NAVD88. All of Pierce County's existing topographic information was referenced to NGVD29, and the conversion was easiest to make for the flood profiles alone rather than for the entire topographic database.

Numerous culverts are located along the studied reaches, primarily at driveways and road crossing, but also at creek crossings in pastures. Culverts that were perceived to present significant controls on water surface elevations were coded into the HEC-RAS model. In total, 35 culverts and 1 bridge were included in the North Fork Tributaries hydraulic model. Where access was granted, culvert dimensions and elevations were surveyed as part of the June 2004 survey.

Roughness values ('n') used for modeling the culverts were chosen based on field observations and engineering judgment. Values depended on the culvert construction material and ranged from 0.011 to 0.013 for concrete

culverts; corrugated metal pipe (CMP) culverts were given an ‘n’ value of 0.024.

The two regional detention ponds place hydraulic controls on upstream and downstream reaches during flood events. To model the E-1 pond, the rating curve of the outlet structure from the pond was used to establish water surface elevations for each flood event. Less information was available for the W-1 pond. Conservative estimates of the water surface elevation for each flood event were based on the plans for the outlet control structure and the elevation of the emergency spillway. These water surface elevations were entered as known water surface elevations in the model.

Starting water surface elevations were based on water surface elevations generated for the North Fork Clover Creek floodplain study. Due to the relatively small watershed size, it was assumed that the flood events on the North Fork Clover Creek and the North Fork Tributaries would be coincident. Therefore, the 10-, 2-, 1-, and 0.2-percent-annual-chance flood profiles on North Fork Clover Creek were used to set the 10-, 2-, 1-, and 0.2-percent-annual-chance starting water surface elevations on the North Fork Tributaries, respectively.

No calibration data was available for the North Fork Tributaries. Interviews with long-time residents and flooding problems documented in the *Clover Creek Basin Plan* provided anecdotal information that was used as a check for the model. Without specific calibration data it was not possible to judge the accuracy of the water surface profiles. However, considerable care was taken to examine the modeling results for reasonableness based on the information that was available.

Four potential split flows were identified on the North Fork of Clover Creek—one upstream of the Burlington Northern Santa Fe Railroad culvert on Tributary 1, a second where Tributary 5 west crosses Golden Given Road, a third at SR 512 on Tributary 5 West, and a fourth between Bingham Road and 40th Avenue East.

Model results show that the split flow at the railroad crossing only occurs for very extreme flood events. During the 0.2-percent-annual-chance flooding event, flow can overtop the right bank and enter Tributary 2 via an existing drainage ditch. Split flow was modeled here for the 0.2-percent-annual-chance event, but not the 1-percent-annual-chance existing or future conditions floods. An internal HEC-RAS flow split optimization routine was used to compute the flow frequencies at which split flow would occur and the amount of flow that would be lost.

Flooding would also overtop the railroad embankment to the south (left bank) and inundate low-lying areas west and downstream of the railroad. Some flow could split off and return to Tributary 1 near 136th Street Court East and Waller Road East through a narrow depression. This potential split flow was not modeled, however, because the flow through this

depression is estimated to be between 1 and 2 cubic feet per second (cfs), which is less than 1 percent of the total flow in the creek and represents shallow flooding of a depth less than 1 foot.

At Golden Given Road the 5th Tributary of Clover Creek makes a 90 degree turn into a roadside ditch before crossing under the road. At this 90 degree turn, high flows would overtop the road, directing a substantial portion of the flow towards and around a cluster of houses and buildings on the other side of the street. This flow split was modeled by creating a new reach to represent this flow path to the point where it converges back into the stream further to the east. The road was represented as a weir at the upstream end of the new reach, while the culvert was modeled as a culvert with a artificially heightened roadway embankment. This would force all flow overtopping the roadway embankment to be directed into the split flow reach. The split flow optimization routine in HEC-RAS was used to compute the flow split ratios.

The flow split upstream of the SR 512 culvert could occur during both the 1-percent-annual-chance and 0.2-annual-chance flood events. Survey and LIDAR data indicate that once flow overtops the right bank, flow could continue westward into one of the basins tributary to the main stem of Clover Creek. The main stem reaches are not included in the HEC-RAS model. Flow loss could be as much as 20 cfs (15 percent of the total flow) for the 1-percent-annual-chance event and as much as 55 cfs (32 percent of the total flow) for the 0.2-percent-annual-chance event, based on an approximation using Manning's equation. Detailed survey data is not available further to the west (downstream) of the flow split. The split flow would eventually end up in another tributary of the main stem of Clover Creek, but its actual path is unclear without a further detailed assessment and additional survey information. HEC-RAS runs with and without the flow loss at the highway culvert showed that the difference in flood elevations downstream was generally less than 0.5 feet. Thus, not reducing the flow in the downstream reaches provides a more conservative estimate of flood elevations, yet does not significantly overestimate flooding. Therefore, in the model, all flow remains in the creek and the floodplain maps reflect this.

The flow split between Bingham Road and 40th Avenue East occurs in an area where the channel appears to have been heavily modified. This area is relatively flat and several channels appear to be cut through the area to facilitate drainage. The flow split reflects conditions that were observed in the field.

*Puyallup River Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 108).*

Approximately three hundred twenty-five cross-sections are used in the HEC-RAS model to represent conditions along the study reach. Pierce

County provided geometry data for most of the cross-sections used in the model. These were surveyed between 1996 and 2001 by consultants working for Pierce County. In several places additional cross-sections were needed. These were obtained from other sources or measured by nhc. Each surveyed cross-section typically only included the river channel and levees. The floodplain was not surveyed; therefore, the floodplain was added to each cross-section using digital topographic data provided to nhc by the County. The topography was developed by Triathlon Ltd. using aerial photographs taken in 1998.

Excluding the earthen levees described previously, bridges are the main structures that affect flow conditions along the Puyallup River. Twenty-four bridges span the Puyallup River within the study reach. Each bridge is included in the hydraulic model except the SR-509 crossing at RM 1.82. It is high and long enough that it does not influence hydraulic conditions on the river. The data used in the model to represent the geometry of each bridge was taken from a combination of field survey notes, bridge plan drawings, and photographs.

In accordance with FEMA guidelines, the starting water surface elevation for each flood profile was set equal to the Mean Higher High Water elevation (MHHW) of Commencement Bay. The mean higher high water elevation, as published by the National Oceanic and Atmospheric Administration, for station 9446484 “Tacoma, Commencement Bay” is equivalent to 9.16 feet NAVD88 (5.67 feet NGVD29).

Channel and overbank roughness factors (Manning’s “n” values) used in the hydraulic computations were chosen using engineering judgment and were based on field observations, orthophotos, calibration to high water marks (HWMs), published data, and Manning’s “n” values from the effective FIS. The “n” values for the main channel of the Puyallup steadily increase from 0.027 near the mouth to 0.05 near Electron, WA. Overbank “n” values range from 0.03 to 0.12 depending on the land cover type.

The hydraulic model was calibrated to high water marks and aerial photographs of flooding that occurred in February 1996, the post-Mud Mountain Dam flood of record. The hydrologic analysis completed for this study indicates that the 1996 event had a recurrence interval of approximately 60 years in the reach of river downstream of the White River confluence and greater than 100 years upstream. Numerous surveyed high water marks from the 1996 event were collected by the USGS and provided to nhc by Pierce County. The County also had aerial photographs taken of the river on February 9, 1996, the day after the peak of the flood. In many areas, these photographs reveal the extent of flooding and the locations where levees failed or were overtopped.

In order to calibrate the model to match the high water marks, discharges corresponding to the peak of the February 1996 event were used in the model. These flows were based on the recorded peak discharges at four USGS stream-gages on the Puyallup River: 12092000 near Electron,

12093500 near Orting, 12096500 at Alderton, and 12101500 at Puyallup. The peak discharges at these gages for the February 1996 flood were reported as 16,000, 18,300, 41,500 and 46,700 cfs respectively. For the calibration exercise, at several locations between these gages, discharges were adjusted to reflect significant inflow from tributaries.

Given the complex nature of the levee system on the Puyallup River and the assorted levee failures that occurred during the 1996 event, it was challenging to calibrate the model in places. Each high water mark was assessed to determine its location and whether it represented flooding conditions inside or outside the levees. Attempts were made to physically locate each HWM in the field, allowing the modelers to assess the flooding conditions that created each mark and subsequently whether it was appropriate to use it for calibration.

The final model calibration is of the highest possible precision given the complexities of the system and the limitations of a one-dimensional model. Ultimately, 38 HWMs, including two marks from USGS gaging stations, were used to calibrate the model.

It must be recognized that although the model is “calibrated”, it cannot provide precise water surface elevations at all locations. Rather, the water surface elevations produced by the model should be considered as a “best estimate” with the understanding that actual levels could be higher or lower.

Once the HEC-RAS model was calibrated, it was used to evaluate the hydraulic characteristics of the 30-mile study reach. Because the Puyallup River is leveed in many areas, the first task was to determine if the existing levees meet FEMA freeboard requirements. FEMA typically requires levees to maintain a minimum of three feet of freeboard above the 1-percent-annual-chance flood water surface profile. This increases to four feet near bridges and three and one-half feet at the upstream end of a levee. Using the HEC-RAS model, it was determined that none of the levees in the study reach meet the FEMA freeboard requirements. This includes the flood control levees downstream of RM 8.0 and between RM 21.5 and RM 20.8 (left bank) which are shown to contain the 100-year flood on the existing FEMA FIRMs. Therefore, in order to meet FEMA evaluation requirements, water surface profiles were computed for three different scenarios: (1) with consideration of both left and right bank levees; (2) without consideration of the right bank levee; and (3) without consideration of the left bank levee.

The following sections of the report describe the results of the hydraulic analysis. For convenience, the study reach has been divided into three reaches: the upper, middle and lower Puyallup River. The hydraulic analysis is described first for the upper Puyallup River, followed by the middle and lower reaches. For each reach, levee freeboard is discussed first, followed by the 1-percent-annual-chance floodplain.

*Upper Puyallup River (RM 30.1 Downstream to Carbon River Confluence)*

Earthen levees exist along portions of both banks. Along the left bank, a levee extends from RM 24.1 downstream to the mouth of Horse Haven Creek (RM 19.3). The existing FIRM shows that a section of this levee was certified to provide 1-percent-annual-chance flood protection. This certified levee is coincident with Leach Avenue in Orting and extends from the Calistoga Bridge (RM 21.5) downstream approximately one mile. This study reveals that neither the certified section nor the uncertified section of the levee meets FEMA freeboard requirements. Therefore, in accordance with FEMA guidelines, the left bank levee was failed and the floodplain was delineated as if the levee does not exist. For this scenario, it was assumed that the right bank levee remained in place, a requirement by FEMA.

On the right bank, a flood control levee is maintained from the Orville Bridge (RM 25.8) to the confluence with the Carbon River (RM 17.5). No part of the right bank levee was previously certified as providing protection from the 1-percent-annual-chance event. Analysis of the right bank levee shows that it fails to meet FEMA freeboard requirements. As such, the right bank levee was failed and the floodplain was delineated as if the levee did not exist. For this scenario, it was assumed that the left bank levee remained in place, a requirement by FEMA.

These assumptions result in severe flooding throughout much of the Town of Orting and surrounding lands. Residents may feel this is unrealistic because the February 1996 event did not flood many of these areas due to the protection provided by levees and community flood fighting efforts. Unfortunately, FEMA cannot recognize the protection offered by the levees since they do not meet FEMA certification requirements.

*Rody Creek Study*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 109).*

The HEC-RAS model for the Rody Creek uses 121 cross-sections to represent the channel and floodplain. Cross-sections were developed from a combination of data sources. The floodplain information for each section was developed from Pierce County topographic data; a field survey (August 2004) was used for the channel information. On occasion, the channel was inaccessible due to either private property restrictions or due to the density of vegetation. In instances where the channel could not be surveyed, channel characteristics were estimated based on adjacent surveyed data and topographic mapping.

Dimensions of hydraulic structures were determined during the field survey. Pierce County provided digital topography created by Triathlon Inc. using aerial photographs taken in 1998. These contours were at 2-

foot intervals and provided sufficient detail to estimate the extent of the floodplain.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or reviewed studies was NGVD29. With the finalization of the NAVD88, most studies are being prepared using NAVD88 as the referenced vertical datum. Typically, NAVD88 elevations are approximately 3.5 feet higher than NGVD29 elevations; precise conversion factors between the two systems can be obtained from the National Geodetic Survey's VERTCON utility (NGS, 1994). This study was initially prepared using NGVD29 as the reference datum for the hydraulic modeling and base map preparation.

However, the final Flood Insurance Rate Maps (FIRMs) and flood profiles have been converted to NAVD88.

Numerous culverts are located along the studied reach on Rody Creek, primarily at driveways and road crossing. All of the culverts identified in the field or from the County's public drainage inventory were coded into the HEC-RAS model. The hydraulic model includes 25 culverts. Since access to private property was limited, some private culverts in the study area may not have been identified. Where access was granted, culvert dimensions and elevations were surveyed as part of the June 2004 survey.

Manning's roughness coefficients for channels, based on field observations and engineering judgment, ranged from 0.035 to 0.050. The highest roughness values are for the heavily overgrown middle watershed where the stream traverses a forested reach and the channel is vegetated. Overbank roughness values ranged from 0.035 to 0.075.

The downstream boundary of Rody Creek is Clarks Creek. The flood stage of Clarks Creek is influenced by the nearby Puyallup River. The tailwater for the Rody Creek analysis was set to the water surface elevation in Clarks Creek and assume coincident flooding events on both creeks. The WSELs were taken from the Clarks Creek Draft FIS as of June 2005 and are referenced to NAVD88.

No calibration data was available for the Rody Creek model. Interviews with long-time residents and flooding problems documented in the *Clear/Clarks Creek Basin Plan* provided anecdotal information that was used as a check for the model. Without specific calibration data it impossible to judge the accuracy of the water surface profiles.

However, considerable care was taken to examine the modeling results for reasonableness based on the information that was available. In general, the HEC-RAS model results came close to reproducing the previously observed flooding problems on the creek. The actual cause of

some of the flooding problems could be blockages or maintenance issues that are not accounted for in this study.

*South Prairie Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 110).*

The HEC-RAS model for the South Prairie Creek study uses 62 cross-sections to represent the geometry of the channel and floodplain. Each cross section was created using data obtained from several different sources. The stream channel portion of each cross section was obtained from data included in an existing WSPRO water surface profile computer model of South Prairie Creek that was created by the USGS (USGS, 1998). The USGS surveyed many of the cross sections in 1996/97; however, they did not survey all of them. A number of the cross sections contain data that was collected in the late 1970's for the original Flood Insurance Study. The floodplain portion of each cross section was obtained from existing paper topographic maps of the valley, provided by Pierce County and created by Walker and Associates. The maps have a scale of 1 inch equals 50 feet, are dated March 23, 1987, and we believe were produced from aerial photographs taken in 1986. With only a few exceptions where the stream channel has changed location, these contour maps provide excellent detail. Pierce County also provided digital topography created by Triathlon Inc. using aerial photographs taken in 1998. Most of these contours were spaced at five foot intervals, too coarse to be useful.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised studies was NGVD29. With the finalization of NAVD88, most studies are being prepared using NAVD88 as the referenced vertical datum. The initial hydraulic analysis for the South Prairie Creek FIS was conducted using the NGVD29 vertical datum; however, the vertical datum was adjusted to NAVD88 for the final submission. Precise conversion factors between the two vertical datums vary by location and can be obtained from the National Geodetic Survey's (NGS) VERTCON utility (NGS,1994). Specifically, NGVD29 elevations along the South Prairie Creek study reach can be converted to NAVD88 elevations by adding 3.51 ft.

During the course of developing the HEC-RAS model for this study, we determined that the elevation reference marks (ERMs) that were set for the original FIS were incorrect. Although we couldn't find many of the original ERMs, we resurveyed those that we could find, tying them in to active bench marks maintained by Pierce County.

The USGS also discovered this error when they surveyed high-water marks left from the February 1996 flood. The USGS surveyed several of

the high-water marks twice, once referencing them to the FEMA ERMs and the other to active Pierce County benchmarks. They found that elevations referenced to the FEMA ERMs were in the range of 1.0 to 1.5 feet too high. As a result of both the USGS and our findings, we have lowered the channel bed elevations of the cross sections that were referenced to the original FEMA ERMs. The elevations of the floodplain portion of each cross section were not adjusted because these were taken from the 1987 topographic maps which appear to provide accurate elevations.

The only structures that have a significant effect on hydraulic conditions are six bridges. Highway 162 crosses the stream five times and an abandoned railroad once. All six bridges are coded in the HEC-RAS model. For each highway bridge, structure dimensions were obtained from as-built plans obtained from the Washington State Department of Transportation. For the abandoned railroad bridge, structure dimensions and elevations were obtained from the existing USGS WSPRO model (USGS, 1998). The only other significant hydraulic feature along the study reach is the earthen berm that has been constructed along the right bank immediately upstream from the Highway 162 bridge near Spring Site Road (RM 3.5). The dimensions and elevations of the berm were obtained from the USGS WSPRO model report (USGS, 1998). It will affect flow conditions, but the impact will be minor because the water will flow around the berm because it is not tied into high ground at the upstream end.

The starting water surface elevation at the mouth of South Prairie Creek is assumed to equal the water surface elevation on the Carbon River assuming coincident peak flows on both systems. Analysis of the timing of flood peaks on South Prairie Creek and the Carbon River reveal that there is a high likelihood that the two systems will peak at about the same time.

Channel and overbank roughness factors (Manning's "n" values) used in the model were first estimated using engineering judgment. These values were then refined by calibrating the model to reproduce highwater marks left by the February 8, 1996 flood. Shortly after the February 1996 flood, the USGS set and surveyed a series of high water marks at key locations along the study reach. The instantaneous peak discharge during the event was estimated to be 8,170 cfs at the USGS gage near the upstream end of the study reach. At the mouth, the peak flow was estimated to be 9,045 cfs. Model parameters were further refined by calibrating to a second flood that occurred on February 15, 1995. It had estimated peak discharges of 2,680 cfs and 2,970 cfs at upstream and downstream ends of the study reach, respectively. It is our opinion that the HEC-RAS model does a very good job reproducing the observed high water mark elevations for both calibration events; therefore, it should yield reasonable water surface profiles for the 10-, 2-, 1- and 0.2-percent-annual-chance floods.

The only significant uncertainty in the accuracy of the model occurs in the vicinity of several Highway 162 bridges. The earthen approach fills that elevate the highway to the bridge deck create obstructions which the flow must pass around. These create split flow conditions which are difficult to model using the one-dimensional HEC-RAS program. Using engineering judgment we have modified the HEC-RAS model to account for these complex conditions. The channel “n” values for the main channel of South Prairie Creek range from 0.035 to 0.037, and the overbank “n” values from 0.04 to 0.15.

As noted in the preceding section, there are several places along the study reach where flow splits occur. We have coded these split flow reaches into the HEC-RAS model. Starting at the upstream end of the study reach and moving downstream, the first major split occurs just upstream from the highway bridge near Spring Site Road (RM 3.5). During a 1-percent-annual-chance flood nearly one-third of the total flow will overtop the right bank upstream from the bridge and flow across Spring Site Road. Most of this water will flow across open dairy pasture and re-enter the stream approximately one-half mile downstream. The second split is almost insignificant. It is just upstream from the third highway bridge along the left bank. During a 1-percent-annual-chance flood, several hundred cubic feet per second will overtop the left bank and flow over cropland before rejoining the channel about one-half mile downstream. The third major split occurs just upstream from the Highway 162 Bridge near River Mile 1. Here water overtops the right bank and flows across the highway, rejoining the main channel about one-tenth of a mile downstream from the bridge. The forth split occurs along the left bank just upstream from the same bridge. Here water overtops the left bank and flows through a narrow corridor between the highway and an old railroad fill. This flow joins the main channel about one-half mile downstream.

*Spanaway Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 111).*

The HEC-RAS model for the study reach includes 71 cross-sections that define the geometry of the channel and floodplain. nhc surveyed the channel portion of 46 of these in the spring of 2002. Nineteen of the remaining sections were interpolated from the surveyed cross-sections, five were copied from an existing 1980 USGS E431 model created in 1980 for the original FIS study (FEMA, 1987) and one was synthesized directly from topographic contour data. With the aide of the BOSS RiverCAD 2000 (BOSS, 2000) computer program, all 71 sections were extended to include the floodplain using topographic maps provided by the County. The maps, which were created by the NIES Mapping Group Inc. are based upon 1998 aerial photographs and have a contour interval of two feet. In several locations of dense vegetation, elevations on the topographic maps varied slightly from survey data (2 +/- feet). In these

locations, floodplain elevations were adjusted using survey data and field observations.

Within the study reach, landowners have made numerous modifications to the stream by adding small pedestrian/auto bridges, diversion weirs, and floodplain ponds. During field surveys and reconnaissance, 23 bridges and 6 inline weirs were noted within the study reach. Only 15 bridges and 3 weirs were coded into the HEC-RAS model. The remaining 8 bridges and 3 weirs were determined by inspection to be small enough that they would not affect flooding or they would wash away during a flood. The most significant hydraulic structure is a concrete weir with an 8 foot drop at RM 2.12. At least 12 private ponds have been constructed in the floodplain adjacent to the stream. Most of these are typically separated from the channel by a low earthen berm and are connected to the stream by a small culvert. Within the HEC-RAS model, it was assumed that the portion of each pond below the top elevation of the berm would not convey flow during an overbank event. These areas were assumed to be "dead storage" and are coded as a "conveyance obstruction" in the model.

A section of Spanaway Loop Road was coded as a lateral weir in the model. During the 1996 flood, Spanaway Creek was reported to have come very close to overtopping Spanaway Loop Road just north of 138th Street. Therefore, the road profile was surveyed and a 520-foot long lateral weir was coded in the model between RM 1.386 to 1.485.

At RM 1.65 the stream splits into two channels and part of the flow permanently leaves Spanaway Creek and enters Morey Creek. This flow split is coded in the HEC-RAS model and the split flow optimization routine is used to automatically determine the discharge lost to Morey Creek for each flood event. A short reach of Morey Creek that begins 235 feet downstream from Spanaway Loop Road and extends upstream to Spanaway Creek is included the HEC-RAS model. The cross sections used to represent Morey Creek were created using a combination of field survey data collected by nhc and topographic mapping. Morey Creek joins Clover Creek a short distance downstream from the Spanaway Creek confluence with Clover Creek.

In accordance with FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2002), the starting water surface elevation for both Spanaway and Morey Creeks were established by assuming normal depth at the downstream most cross-section. Energy slopes were estimated from topographic mapping and were set to approximately 0.014 and 0.0005 for Spanaway and Morey Creeks respectively. Backwater flooding from Clover Creek is not expected to influence Spanaway Creek above the study reach boundary at Spanaway Loop Road nor at the first coded cross section for Morey Creek.

It was impossible to truly calibrate all reaches of the HEC-RAS model because only two quality high-water marks could be identified and

surveyed. These marks were identified by local residents and were left by the 1996 flood. In addition, however, local residents provided a significant amount of anecdotal information on flood levels which helped considerably in the calibration process. Based upon the results of the HSPF modeling, it is estimated that the 1996 flood had a peak discharge of 67 cfs below the Morey Creek dis-tributary. This corresponds approximately to a 1-percent-annual-chance flood. The two high water marks surveyed are located at RM 1.24 on a deck post of a house at 14027 Spanaway Loop Road (elevation 295.1 feet NGVD29/298.52 feet NAVD88) and at RM 1.03 on a swing set in the front yard of a house at 1513 138th Street South (elevation of 291.6 feet NGVD29 / 295.02 feet NAVD88). Initial estimates of Manning's "n" values were made for the channel and floodplain based upon engineering judgment and reference to classical publications (Chow, 1959; Barnes, 1967; and Hicks and Mason, 1991). Without any modifications to the initial n-value estimates, the computed water surface elevations matched the high-water marks within one tenth of one foot. Further analyses revealed that the high-water marks were not sensitive to changes in Manning's "n" values within the range deemed reasonable. Following this step, Manning's "n" values and expansion/contraction coefficients were further refined to ensure that the model reproduced water levels for the 1996 flood that were similar to those described by local residents and Pierce County road maintenance personnel.

*Swan Creek Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 112).*

The HEC-RAS model for Swan Creek includes 61 cross-sections. These cross-sections serve to represent the geometry of the channel and floodplain along the study reach. Because no cross-section data exists for Swan Creek, nhc surveyed new channel cross-sections. In reaches where access to the channel was unavailable, channel cross-sections were interpolated to augment the surveyed data and occurred at 6 locations. Topographic data provided by Pierce County, in the vicinity of Swan Creek, was used to extend both the surveyed and interpolated channel cross-sections to include the floodplain. The topographic data was developed in 1998, with a contour interval of two feet.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised studies was NGVD29. With the finalization of NAVD88, most studies are being prepared using NAVD88 as the referenced vertical datum. The initial hydraulic analysis for the Swan Creek FIS was conducted using the NGVD29 vertical datum; however, the vertical datum was adjusted to NAVD88 for the final submission. Precise conversion factors between the two vertical datums vary by location and can be obtained from the Army

Corps of Engineers' CORPSCON utility (USACE, 1997). Specifically, NAVD88 elevations along the Swan Creek study reach can be converted to NGVD29 elevations by subtracting 3.46 feet.

The hydraulic analysis of the Swan Creek study reach includes 9 culverts and 1 bridge crossing. Within the HEC-RAS model, all culverts were modeled using the Highest Upstream Energy method, while bridges were modeled either with the Energy, or Pressure and/or Weir methods, depending on level of inundation.

The downstream limit of the Swan Creek HEC-RAS model is located at the inlet of a culvert structure at 64th Street East. This location was chosen because it provided a convenient point to join flood profiles from the effective FIS (FEMA, 1987). Although water surfaces at the selected downstream limit are likely controlled by the culvert structure located immediately downstream, a normal depth starting water surface elevation was implemented. Because the channel in this location is deeply entrenched within a narrow gully, the floodplain extents are not significantly affected by starting water surface elevation. Thus, the choice of using the normal depth boundary condition is considered reasonable in this situation.

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the creek and floodplain areas. The channel "n" values for the main channel of Swan Creek typically range from 0.04 to 0.045. Overbank "n" values range from 0.05 to 0.10. No reliable high water marks are available to calibrate the Swan Creek model.

*Wapato Creek I & II Study*

*For Tables, Photos, and Appendices related to this study, please see the nhc report (Reference 113).*

The HEC-RAS model for Wapato I includes 137 cross-sections, while the Wapato II model includes 25 cross-sections. These cross-sections serve to represent the geometry of the channel and floodplain along each of the study reaches. To account for the substantial development in the drainage basin since the previous FIS study, nhc surveyed new channel cross sections along most of Wapato I and II. The topographic data provided by Pierce County, in the vicinity of Wapato Creek was used to extend these cross sections to include the floodplain. This data was developed in 1998, with a contour interval of two feet. In several areas where the channel has not changed significantly, cross sections from the original FIS study were imported into the new HEC-RAS model. These cross-sections were assembled in the late-1970's by the USGS through a combination of field surveying and topography information obtained from aerial photos (FEMA, 1987).

In several locations, flood water splits from the stream corridor and flows across flat lands which were once part of the Puyallup River floodplain. In these areas, flow depths were estimated using approximate methods which are based upon cross sections estimated from the topographic maps provided by Pierce County.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised studies was NGVD29. With the finalization of NAVD88, most studies are being prepared using NAVD88 as the referenced vertical datum. The initial hydraulic analyses for the Wapato Creek FIS were conducted using the NGVD29 vertical datum; however, the vertical datum was adjusted to NAVD88 for the final submission. Precise conversion factors between the two vertical datums vary by location and can be obtained from the National Geodetic Survey's (NGS) VERTCON utility (NGS,1994). Specifically, NGVD29 elevations along the Wapato Creek study reaches can be converted to NAVD88 elevations by adding 3.49 feet.

The hydraulic analysis of the Wapato I study reach includes 32 culvert crossings, 17 bridge structures, and 2 lateral weir structures. The Wapato II study reach includes 7 culvert crossings and 2 inline weir structures. Within the HEC-RAS model, all culverts were modeled using the Highest Upstream Energy method, while bridges were modeled either with the Energy, or Pressure and/or Weir methods, depending on level of inundation. Skews were also applied to bridge or culvert crossings when deemed necessary.

Approximate methods were utilized on Wapato Creek at locations of overtopping; namely, at the lateral weirs on Wapato I, located in the City of Fife, and the downstream of the study limits on Wapato II. In either case, overflow discharges were computed by HEC-RAS using a weir routine and normal flow calculations were then performed to estimate the average water depths in representative cross-sections within the overflow region. For Wapato II, storage-volume at a downstream depression was also estimated.

The downstream boundary conditions for both Wapato I and Wapato II consist of long culvert structures, thus the normal depth starting water surface elevations recommended by the FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2002) were not implemented. Alternatively, the culvert structures were included in the HEC-RAS model and the culvert routines were used to determine headwater depths. To facilitate the computation, "estimated" cross-sections, within the Blair Waterway and the Puyallup River were placed at the downstream end of each of the culvert structures. Tailwater depths were then selected as boundary conditions for each study reach. For the Wapato I study reach, the mean higher high water tide level of 5.67 feet,

NGVD29 was selected for all flood profiles modeled. For the Wapato II study reach, coincident flood peaks were assumed between the study reach and the Puyallup River.

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the creek and floodplain areas. The channel "n" values for the main channel of Wapato Creek typically range from 0.04 to 0.07, with values up to 0.15 used in locations with heavily vegetated channels or apparent obstructions. Overbank "n" values range from 0.05 to 0.15. In several locations "n" values as low as 0.025 are used to represent sections of the channel that are either concrete lined or have manicured lawns right up to the edge of the low water channel. No reliable high water marks are available to calibrate the Wapato Creek models.

*White River Study*

*For Tables, Photos, and Appendices related to this study, please see NHC Report (Reference 114).*

The HEC-RAS model for this study includes 39 cross sections to represent the geometry of the channel and floodplain along the study reach. In- channel portions of the cross sections were surveyed by the Corps of Engineers or King County in 2002 or 2001, respectively. Overbank portions of the cross sections were cut from topographic mapping using the HEC-GeoRAS GIS processing tool (Version 3.0; COE, 2004). The most recent available topographic mapping at a two-foot contour interval (1998) was obtained from Pierce County for this purpose.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised studies was NGVD29. With the finalization of NAVD88, most studies are being prepared using NAVD88 as the referenced vertical datum. At Pierce County's request, this study was originally referenced to NGVD29, in an exception approved by FEMA. This FIS and associated flood hazard mapping have been converted to NAVD88 by adding 3.93 feet to all elevations. Topographic mapping and survey data associated with this study are still referenced to NGVD29.

The hydraulic model of the lower White River included six bridge structures, including the recently constructed Tacoma Avenue bridge, which is not shown in the most recent available topography and aerial photographs. A utility crossing at RM 3.42 (cross section W) was left out of the analysis because it has negligible effect on the stream flow. Bridges were modeled with either the Energy or Pressure and/or Weir methods depending on level of inundation.

The bridge data and road profile for 8th Street East were taken from preliminary plans for the 8th Street East Corridor Improvement Project, currently under construction by Pierce County.

In the vicinity of the 8th Street East crossing, high ground separating the channel and left floodplain creates a split flow condition. In order to properly model overtopping of the road and to maintain consistent flows in the channel and floodplain, left overbank flow between cross sections AL and AA was routed in a separate reach. The split flow reach consists of ten cross sections, representing the left overbank portions of the main channel cross sections, and an inline weir representing 8th Street. The flow balance between the main channel and the split flow reach was determined using the Split Flow Optimization option in HEC-RAS.

In accordance with FEMA Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2002), the starting water surface elevation for the backwater model was assumed to be a coincident peak (or 50,000 cfs - see discussion below) on the Puyallup River. Although White River flows are regulated at Mud Mountain Dam, analysis of several large events indicates that, while most flood volume is released after the peak on the Puyallup River, White River flows typically are at or near peak level at the time of the Puyallup River peak, before releases are curtailed. The observed flow pattern is consistent with the operations procedure for Mud Mountain Dam aimed at controlling peaks on the Puyallup to 50,000 cfs.

Hydrologic analysis for the Puyallup River, also being modeled by nhc, indicates that peak flows on the Puyallup below the White River do not exceed 50,000 cfs until above the 1-percent-annual-chance event. Consequently, starting water surface elevations for the White River are equal to corresponding peak Puyallup River elevations (from nhc's updated Puyallup River model) for the 10-, 2-, and 1-percent-annual-chance events, and the 0.2-percent-annual-chance starting water surface elevation corresponds to the Puyallup River elevation for a flow of 50,000 cfs at the White River confluence.

As a result, the 0.2-percent-annual-chance water surface at the mouth of the White River does not correspond with the 0.2-percent-annual-chance water surface on the Puyallup River. Because the 0.2-percent-annual-chance water surface is higher on the Puyallup River, the 0.2-percent-annual-chance floodplain in the vicinity of the mouth of the White River will be delineated based on the 0.2-percent-annual-chance flood on the Puyallup.

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the river and floodplain areas. The channel "n" values for the main channel of the White River range from 0.037 to 0.05, and the overbank "n" values from 0.05 to 0.085.

High water marks (HWMs) from the February 1996 event were obtained from Pierce County to verify model accuracy. Of the six HWMs that could

be located with sufficient accuracy, two reported debris lines fell several feet below the modeled February 1996 water surface and were determined to be from a smaller storm that occurred immediately before the HWM survey was performed (in April 1996). A third mark, at the State Route 410 bridge, was inconsistent with nearby HWMs on the White and Puyallup rivers. Note that model calibration was performed using elevations referenced to NGVD29 for comparison to surveyed HWMs.

The model was calibrated to within 0.5 feet of two of the three valid HWMs and within 1.0 feet of the third. The latter mark is located in a section with a highly mobile bed, and the calibration assumed significant bed scour relative to the current channel bed elevations. It should also be noted that significant bank erosion was reported to nhc by a landowner just upstream of the high water mark section, and the bank where the HWM was located has been altered since 1996 with the modification of the utility crossing.

*Woodland Creek Study*

*For Tables, Photos, and Appendices related to this study, please see Tetra Tech Report (Reference 115).*

The HEC-RAS model for Woodland Creek uses 86 cross sections to represent the channel and floodplain. Cross sections were developed from a combination of data sources. The floodplain information for each section was developed from Pierce County topographic data; a field survey (June 2004) was used for the channel information. In instances where the channel could not be surveyed, channel characteristics were estimated based on adjacent surveyed data and topographic mapping.

Manning's roughness coefficients for channels, based on field observations and engineering judgment, ranged from 0.015 to 0.065. A higher roughness value was generally used (from 0.055 to 0.065) to account for densely vegetated conditions in this intermittent stream. Overbank roughness values ranged from 0.02 to 0.12.

Pierce County provided digital topography created by Triathlon Inc. using aerial photographs taken in 1998. These contours were at 2-foot intervals and provided sufficient detail to estimate the extent of the floodplain.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or reviewed studies was NGVD29. With the finalization of NAVD88, most studies are being prepared using NAVD88 as the referenced vertical datum. Typically, NAVD88 elevations are approximately 3.5 feet higher than NGVD29 elevations; precise conversion factors between the two systems can be obtained from the National Geodetic Survey's VERTCON utility (NGS, 1994). This study was initially prepared using NGVD29 as the reference datum for the hydraulic modeling and base map preparation. However,

the final Flood Insurance Rate Maps (FIRMs) and flood profiles have been converted to NAVD88.

Numerous culverts are located along the studied reach on Woodland Creek, primarily at driveways and road crossing, but also at creek crossings in pastures. Culverts that were perceived to present significant controls on water surface elevations were coded into the HEC-RAS model. The hydraulic model includes 19 culverts. Since access to private property was limited, some private culverts in the study area may not have been identified. Where access was granted, culvert dimensions and elevations were surveyed as part of the June 2004 survey.

Two long culverts were modeled in the computer program CulvertMaster as well was in the HEC-RAS model: a culvert at 104th Street East that daylights near 99th Street East; and a culvert that starts inside the Washington State University Agricultural Experiment Station and ends at Clarks Creek. Roughness values used for modeling the culverts were chosen based on field observations and engineering judgment. Values depended on the culvert construction material and ranged from 0.011 to 0.013 for concrete culverts; corrugated metal pipe (CMP) culverts were given an ‘n’ value of 0.024.

Three lateral weir structures were coded into the HEC-RAS model to estimate flow splits at culvert locations. A flow split was determined to occur where flows overtopped a culvert and the adjacent bank height was not sufficient to contain the flow. Two lateral weirs were used to represent split flows at culverts along 72nd Avenue East downstream of 96th Street East. The third lateral weir was used represent the split flow at the 80th Street East culvert.

The starting water surface elevations are based on the water surface elevations generated for the Clarks Creek floodplain study. Due to the relatively small watershed size, it was assumed that flood events on Woodland Creek and Clarks Creek would be nearly coincident. Therefore, the 10-, 2-, 1-, and 0.2-percent-annual-chance flood profile on Clarks Creek was used to set the 10-, 2-, 1-, and 0.2-annual-chance-flood starting water surface elevations on Woodland Creek, respectively.

No calibration data was available for Woodland Creek. Interviews with long-time residents and flooding problems documented in the *Clear/Clarks Creek Basin Plan* provided anecdotal information that was used as a check for the model. Without specific calibration data it impossible to judge the accuracy of the water surface profiles. However, considerable care was taken to examine the modeling results for reasonableness based on the information that was available.

Flow splits were modeled at two culverts along 72nd Avenue East south of 96th Street East and at the 80th Street East culvert. At all three locations, a flow split occurs for flows at the 10-year recurrence interval and above.

At 80th Street East, the split flow is complex and likely has several components. Because the culvert is located on a road that slopes to the east, a portion of the flow overtopping the culvert would likely run along the length of the road to two locations where flow may be able to cross the road and flow overland through the agricultural fields of the Washington State University Experimental Station. The HEC-RAS model was only able to model one split flow path; and the primary split flow path as determined by field observation was chosen to be modeled. Flow in the secondary flow path was observed to carry a much smaller amount of flow, predominantly as shallow sheet flow. The secondary flow path joins the primary flow path after approximately 250 feet. The split flow travels north through the WSU property, where it is collected in the WSU drainage system and returned to the creek in the wetland upstream of Pioneer Way. The total length of the flow split is approximately 2,000 feet.

Along 72nd Avenue East, it appears that the creek has been modified from its original drainage course. The creek currently flows in a ditch along 72nd Avenue East. The ditch is perched above a low spot to the east that appears to be the location of the original channel. When the creek overtops two of the driveway culverts, the water flows to the east and collects in the low spot before flowing downstream to the north. The split flow path is complicated by the houses that have been built in the area, but the split rejoins the creek in the pasture area downstream of 93rd Street East.

There were no new coastal hydraulic analyses performed for this initial countywide FIS report.

## 3.3 Wave Height Analyses

Since extreme tides would most likely be associated with a severe winter storm, the probability of the extreme tides and heavy wind wave action occurring simultaneously is likely. Under these circumstances, the possibility of wave damage should be considered when determining the flood potential.

The Pierce County Coastal Flood Hazard Analysis and Mapping study included field reconnaissance to determine representative shoreline reaches, a wave model for the intricate geometry of the Pierce County shoreline on the south end of Puget Sound, coastal analysis in the form of transect analysis, including wave setup, wave run-up and overtopping, and statistical analysis to determine base flood elevations and coastal flood hazard zone mapping.

Wave setup can be a significant contributor to the total water level at the shoreline and was included in the determination of coastal base flood elevations. Wave setup is defined as the increase in total stillwater elevation against a barrier caused by the attenuation of waves in shallow water. Wave setup is based upon wave breaking characteristics and profile slope.

Wave setup for Pierce County was calculated using the Direct Integration Method (DIM). The DIM setup equations can be found in the Pacific Coast Guidelines Equations (Reference 116). Static setup was calculated for the each run at every transect based on the near-shore slope, as defined in the guidelines and specifications. Since Puget Sound is considered to be a sheltered water body, the dynamic component of setup was not included in the total water level, as it is said to be negligible in the sheltered waters guidance (Reference 117).

Wave run-up is the uprush of water caused by the interaction of waves with the area of shoreline where the stillwater hits the land or other barrier intercepting the stillwater level. The wave run-up elevation is the vertical height above the stillwater level ultimately attained by the extremity of the up-rushing water. Wave run-up at a shore barrier can provide flood hazards above and beyond those from stillwater inundation.

Wave run-up for Pierce County was calculated for all 150 storm events at each transect; however the method varied based on the type of shoreline. The Type 1 (beaches) spreadsheet used the DIM method for calculating run-up, as detailed in the guidelines. The DIM run-up equations are contained in the Pacific Coast Guidelines as Equations (Reference 116) DIM is a one-dimensional empirical method that calculates static and dynamic setup, with the option to include the influence of variable spectral width in the dynamic setup. The setup parameters calculated through DIM are then statistically added to the incident run-up on a natural beach. It was decided to use the beach face slope in the calculation of run-up rather than the near-shore slope used in the setup calculation. This was due to the steeper nature of beach face slopes where the run-up will occur, compared to the near-shore slope taken from twice the breaking depth as suggested in the guidelines.

Run-up on barriers is dependent on wave height and steepness, as well as structure geometry. The Type 2 (barriers) spreadsheets calculated run-up using the TAW (Technical Advisory Committee for Water Retaining Structures) method, based on the structure slope, Iribarren number and reduction factors developed by Battjes (1974), Van der Meer (1988), and de Waal & Van der Meer (1992). The reduction factors include surface roughness, influence of a berm, porosity and oblique wave incidence. The Van der Meer relationship is recommended for use due to its wide range of applicability and long established international acceptance. The run-up equation recommended for use as the TAW methodology (Reference 119) is provided in the Pacific Coast Guidelines (Reference 116).

The Type 3 spreadsheet for vertical walls was set up to calculate run-up using methods provided in the Shore Protection Manual as there was no clear guidance in the Pacific Coast Guidelines for run-up on vertical structures (Reference 118). The SPM nomograph for run-up on vertical walls was digitized and an interpolation routine was set up to extract run-up from this digitization (Reference 118).

Once the wave setup and wave run-up were computed for each run, the total water level elevation could be calculated for all 150 storms for each transect. The total water level is the combination of still water, including storm surge for each event, and wave components, including setup and run-up.

Overtopping occurs when a barrier crest is lower than the potential run-up elevation, or total water level. Since a barrier is required, overtopping was only calculated for Type 2 (steeply sloping barriers) and Type 3 (vertical structure) spreadsheets. Equations were provided in the guidelines for both Type 2 and 3 shorelines. For Type 2 spreadsheets, overtopping was calculated using the Van der Meer method (Reference 119). The Besley and Allsop method was used for overtopping calculations in the Type 3 spreadsheets (Reference 119).

The total water level (TWL) is comprised of the still water elevation including storm surge, in addition to the wave setup and run-up components. A TWL is calculated for each of the 150 storm event runs at each transect, with a total of 112 transects. Using the top 100 TWLs, the Generalized Pareto Distribution statistical method was applied to each transect to extrapolate the extreme water levels for the 1- and 0.2-percent-annual-chance total water levels. These extreme water levels were then used to determine the coastal flood hazard zones for the coastal mapping.

Once the water level modeling for the Puget Sound study was complete, background data was collected and transect locations were finalized, a detailed wave model for the complex geometry of Pierce County was generated in order to extract transect wave data at the south end of Puget Sound, where waves from the Pacific Coast cannot penetrate. Simulating Waves Near-shore (SWAN) modeling methodology was the chosen model for this sheltered area of Pierce County.

Extreme water levels were used to map the coastal flood hazard zones as detailed in the Pacific Coast Guidelines (Reference 116). VE zones were mapped at the shoreline when the wave action was large enough that there could be structural damage due to wave energy. Coastal AE zones were mapped when the wave height and run-up were too low to meet the velocity zone criteria, which occurred on the more sheltered and more gradually sloping shorelines of Pierce County. The base flood elevation for these zones was dependent on the 1-percent-annual-chance total water level determined through the transect analysis spreadsheets and statistical analysis for each transect.

In Pierce County, the majority of the shoreline consisted of steeply sloping bluffs. For this shoreline type, the VE zone (or coastal AE zone) was mapped to the elevation contour that matches the BFE calculated by the analysis on the shoreline at that transect. In the case of an overtopped barrier, the BFE was limited to three feet above the structure, if necessary, and the flood zone elevations behind the barrier were stepped down. Mapping procedures for a shoreline consisting of primary frontal dune are

not discussed here because primary frontal dunes were not found to be present in Pierce County.

Once the landward extent of the flood zones was determined for each transect, it was necessary to interpolate between transects to create a smooth and continuous coastal flood hazard zone extent. The smooth transition between transects had to be made based on engineering judgment in evaluating shoreline type, topography, land cover and upland development type.

The Limit of Moderate Wave Action (LiMWA) is determined and defined as the location of the 1.5-foot wave. Typical constructions in areas of wave heights less than 3-feet high have experienced damage, suggesting that construction requirements within some areas of the AE zone should be more like those requirements for the VE zone. Testing and investigations have confirmed that a wave height greater than 1.5 feet can cause structure failure. The LiMWA was determined for all areas subject to significant wave attack in accordance with “Procedure Memorandum No. 50 - Policy and Procedures for Identifying and Mapping Areas Subject to Wave Heights Greater than 1.5 feet as an Informational Layer on FIRMs” (Reference 117).

The effects of wave hazards in the Zone AE areas (or shoreline in areas where VE Zones are not identified) and the limit of the LiMWA boundary are similar to, but less severe than, those in Zone VE where 3-foot breaking waves are projected during a 1-percent-annualchance flooding event. The effects of wave hazards in the Zone AE areas (or shoreline in areas where VE Zones are not identified) and the limit of the LiMWA boundary are similar to, but less severe than, those in Zone VE where 3-foot breaking waves are projected during a 1-percent-annual-chance flooding event.

Transects were placed perpendicular to the mean shoreline or parallel to the mean direction of wave propagation. Table 5a, “Transect Data Location,” shows the stillwater elevations of the 112 transects along Pierce County.

Table 5a. Transect Data Location

| TRANSECT | STILLWATER ELEVATIONS 10-PERCENT-ANNUAL-CHANCE | 2-PERCENT-ANNUAL-CHANCE | 1-PERCENT-ANNUAL-CHANCE | 0.2-PERCENT-ANNUAL-CHANCE | TOTAL WATER LEVEL[1] 1-PERCENT-ANNUAL-CHANCE | ZONE | BASE FLOOD ELEVATION (FEET NAVD88[2,3]) |
|---|---|---|---|---|---|---|---|
| 1 | * | * | 10.9 | 11.0 | 13.0 | AE | 15 |
| 2 | * | * | 11.4 | 11.7 | 14.3 | AE | 17 |
| 3 | * | * | 12.8 | 13.0 | 15.2 | AE | 17 |
| 4 | * | * | 14.5 | 15.2 | 17.8 | VE | 20 |
| 5 | * | * | 12.7 | 12.5 | 14.9 | AE | 17 |
| 6 | * | * | 12.8 | 12.8 | 14.9 | AE | 17 |
| 7 | * | * | 11.1 | 11.5 | 13.4 | AE | 15 |
| 8 | * | * | 12.9 | 13.5 | 14.6 | AE | 16 |
| 9 | * | * | 12.7 | 13.3 | 14.6 | AE | 16 |
| 10 | * | * | 12.7 | 13.2 | 14.5 | AE | 16 |
| 11 | * | * | 12.5 | 12.2 | 13.7 | AE | 15 |
| 12 | * | * | 11.9 | 11.7 | 12.8 | AE | 14 |
| 13 | * | * | 14.1 | 14.2 | 17.1 | VE | 20 |
| 14 | * | * | 15.2 | 15.8 | 18.6 | VE | 21 |
| 15 | * | * | 15.4 | 15.9 | 18.5 | VE | 21 |
| 16 | * | * | 13.7 | 13.8 | 16.1 | VE | 18 |
| 17 | * | * | 13.9 | 14.3 | 17.4 | VE | 20 |
| 18 | * | * | 15.8 | 16.5 | 19.7 | VE | 23 |
| 19 | * | * | 9.9 | 10.5 | 13.6 | VE | 17 |
| 20 | * | * | 15.3 | 16.0 | 18.8 | VE | 21 |
| 21 | * | * | 14.6 | 15.0 | 17.4 | VE | 19 |

[1] Including stillwater elevation and effects of wave setup
[2] Due to map scale limitations, BFEs shown on the FIRM may represent average elevation for the zone depicted
[3] North American Vertical Datum of 1988 (NAVD88)
* Data not available

Table 5a. Transect Data Location (continued)

| TRANSECT | STILLWATER ELEVATIONS 10-PERCENT-ANNUAL-CHANCE | 2-PERCENT-ANNUAL-CHANCE | 1-PERCENT-ANNUAL-CHANCE | 0.2-PERCENT-ANNUAL-CHANCE | TOTAL WATER LEVEL[1] 1-PERCENT-ANNUAL-CHANCE | ZONE | BASE FLOOD ELEVATION (FEET NAVD88[2,3]) |
|---|---|---|---|---|---|---|---|
| 22 | * | * | 15.8 | 15.8 | 18.0 | VE | 20 |
| 23 | * | * | 14.8 | 15.3 | 17.6 | VE | 19 |
| 24 | * | * | 15 | 15.4 | 17.3 | VE | 19 |
| 25 | * | * | 13.3 | 13.4 | 16.1 | VE | 19 |
| 26 | * | * | 14.8 | 15.4 | 17.7 | VE | 19 |
| 27 | * | * | 14.9 | 15.8 | 17.8 | VE | 19 |
| 28 | * | * | 14.1 | 14.3 | 16.2 | VE | 18 |
| 29 | * | * | * | * | * | AE | * |
| 30 | * | * | 15.8 | 16.0 | 18.2 | VE | 20 |
| 31 | * | * | 14.7 | 15.3 | 17.6 | VE | 20 |
| 32 | * | * | 15.9 | 16.2 | 18.3 | VE | 20 |
| 33 | * | * | 12.8 | 12.8 | 14.1 | AE | 15 |
| 34 | * | * | 15.5 | 16.2 | 18.7 | VE | 21 |
| 35 | * | * | 15.5 | 15.6 | 18.1 | VE | 20 |
| 36 | * | * | 14.6 | 15.1 | 17.5 | VE | 20 |
| 37 | * | * | 14.7 | 15.5 | 17.8 | VE | 20 |
| 38 | * | * | 13.2 | 13.4 | 15.3 | AE | 17 |
| 39 | * | * | 14.1 | 14.6 | 16.5 | VE | 18 |
| 40 | * | * | 13.9 | 14.6 | 17.7 | VE | 21 |
| 41 | * | * | 14.2 | 14.9 | 17.6 | VE | 20 |
| 42 | * | * | 11.2 | 11.6 | 14.4 | AE | 17 |
| 43 | * | * | 13.8 | 14.4 | 17.7 | VE | 21 |

[1] Including stillwater elevation and effects of wave setup
[2] Due to map scale limitations, BFEs shown on the FIRM may represent average elevation for the zone depicted
[3] North American Vertical Datum of 1988 (NAVD88)
* Data not available

Table 5a. Transect Data Location (continued)

| TRANSECT | STILLWATER ELEVATIONS 10-PERCENT-ANNUAL-CHANCE | 2-PERCENT-ANNUAL-CHANCE | 1-PERCENT-ANNUAL-CHANCE | 0.2-PERCENT-ANNUAL-CHANCE | TOTAL WATER LEVEL[1] 1-PERCENT-ANNUAL-CHANCE | ZONE | BASE FLOOD ELEVATION (FEET NAVD88[2,3]) |
|---|---|---|---|---|---|---|---|
| 44 | * | * | 10.9 | 10.9 | 13.9 | VE | 17 |
| 45 | * | * | 14.9 | 15.0 | 18.1 | VE | 21 |
| 46 | * | * | 15.1 | 15.9 | 18.8 | VE | 21 |
| 47 | * | * | 15.7 | 16.1 | 18.4 | VE | 20 |
| 48 | * | * | 14.9 | 15.4 | 17.5 | VE | 19 |
| 49 | * | * | 15.8 | 16.2 | 18.4 | VE | 20 |
| 50 | * | * | 15.5 | 16.0 | 18.6 | VE | 21 |
| 51 | * | * | 15.4 | 15.5 | 18.0 | VE | 20 |
| 52 | * | * | 14.9 | 15.2 | 17.3 | VE | 19 |
| 53 | * | * | 12.3 | 11.9 | 14.6 | AE | 17 |
| 54 | * | * | 13.3 | 13.6 | 16.4 | VE | 19 |
| 55 | * | * | 15.2 | 15.6 | 18.4 | VE | 21 |
| 56 | * | * | 15.6 | 15.9 | 18.3 | VE | 20 |
| 57 | * | * | 12.4 | 12.9 | 15.5 | AE | 18 |
| 58 | * | * | 14.7 | 15.1 | 17.5 | VE | 19 |
| 59 | * | * | 15.5 | 15.9 | 18.4 | VE | 20 |
| 60 | * | * | 15.6 | 15.5 | 17.9 | VE | 20 |
| 61 | * | * | 15.6 | 15.7 | 18.1 | VE | 20 |
| 62 | * | * | 13.6 | 13.8 | 16.1 | VE | 18 |
| 63 | * | * | 14.6 | 15.4 | 17.8 | VE | 20 |
| 64 | * | * | 15.6 | 16.2 | 18.6 | VE | 21 |
| 65 | * | * | 15.4 | 15.7 | 18.3 | VE | 21 |

[1] Including stillwater elevation and effects of wave setup
[2] Due to map scale limitations, BFEs shown on the FIRM may represent average elevation for the zone depicted
[3] North American Vertical Datum of 1988 (NAVD88)
* Data not available

Table 5a. Transect Data Location (continued)

| TRANSECT | STILLWATER ELEVATIONS 10-PERCENT-ANNUAL-CHANCE | 2-PERCENT-ANNUAL-CHANCE | 1-PERCENT-ANNUAL-CHANCE | 0.2-PERCENT-ANNUAL-CHANCE | TOTAL WATER LEVEL[1] 1-PERCENT-ANNUAL-CHANCE | ZONE | BASE FLOOD ELEVATION (FEET NAVD88[2,3]) |
|---|---|---|---|---|---|---|---|
| 66 | * | * | 14.4 | 14.7 | 16.2 | VE | 18 |
| 67 | * | * | 13.1 | 13.4 | 15.4 | AE | 17 |
| 68 | * | * | 14.8 | 15.3 | 17.5 | VE | 19 |
| 69 | * | * | 14.3 | 14.8 | 16.5 | VE | 18 |
| 70 | * | * | 15.0 | 15.2 | 17.2 | VE | 19 |
| 71 | * | * | 14.4 | 14.9 | 16.5 | VE | 18 |
| 72 | * | * | 12.8 | 12.5 | 14.7 | AE | 17 |
| 73 | * | * | 14.1 | 14.5 | 17.4 | VE | 20 |
| 74 | * | * | 16.0 | 16.4 | 19.4 | VE | 22 |
| 75 | * | * | 14.9 | 15.5 | 18.7 | VE | 22 |
| 76 | * | * | 14.6 | 15.2 | 18.7 | VE | 22 |
| 77 | * | * | 13.7 | 13.7 | 17.0 | VE | 20 |
| 78 | * | * | 14.7 | 14.4 | 17.8 | VE | 21 |
| 79 | * | * | 16.7 | 16.8 | 20.2 | VE | 23 |
| 80 | * | * | 16.7 | 16.6 | 20.0 | VE | 23 |
| 81 | * | * | 14.6 | 14.7 | 18.1 | VE | 21 |
| 82 | * | * | 12.2 | 12.3 | 15.2 | AE | 18 |
| 83 | * | * | 14.2 | 14.4 | 17.3 | VE | 20 |
| 84 | * | * | 15.2 | 15.3 | 18.2 | VE | 21 |
| 85 | * | * | 14.9 | 14.7 | 17.8 | VE | 21 |
| 86 | * | * | 15.9 | 16.3 | 19.4 | VE | 22 |
| 87 | * | * | 15.5 | 15.8 | 18.2 | VE | 20 |

[1] Including stillwater elevation and effects of wave setup
[2] Due to map scale limitations, BFEs shown on the FIRM may represent average elevation for the zone depicted
[3] North American Vertical Datum of 1988 (NAVD88)
* Data not available

Table 5a. Transect Data Location (continued)

| TRANSECT | STILLWATER ELEVATIONS 10-PERCENT-ANNUAL-CHANCE | 2-PERCENT-ANNUAL-CHANCE | 1-PERCENT-ANNUAL-CHANCE | 0.2-PERCENT-ANNUAL-CHANCE | TOTAL WATER LEVEL[1] 1-PERCENT-ANNUAL-CHANCE | ZONE | BASE FLOOD ELEVATION (FEET NAVD88[2,3]) |
|---|---|---|---|---|---|---|---|
| 88 | * | * | 15.1 | 14.9 | 17.8 | VE | 20 |
| 89 | * | * | 18.2 | 18.1 | 21.0 | VE | 23 |
| 90 | * | * | 15.1 | 15.2 | 18.1 | VE | 21 |
| 91 | * | * | 14.5 | 14.6 | 16.1 | VE | 17 |
| 92 | * | * | 14.5 | 14.6 | 16.2 | VE | 17 |
| 93 | * | * | 14 | 14.7 | 17.7 | VE | 20 |
| 94 | * | * | 11.1 | 11.4 | 14.4 | AE | 17 |
| 95 | * | * | 13.8 | 13.8 | 17.0 | VE | 20 |
| 96 | * | * | 14.5 | 14.5 | 16.0 | VE | 17 |
| 97 | * | * | 13.6 | 13.7 | 15.2 | AE | 16 |
| 98 | * | * | 14.4 | 14.8 | 16.4 | VE | 18 |
| 99 | * | * | 14.4 | 14.7 | 16.3 | VE | 18 |
| 100 | * | * | 14 | 14.7 | 16.7 | VE | 18 |
| 101 | * | * | 14.3 | 14.8 | 16.5 | VE | 18 |
| 102 | * | * | 13 | 13.8 | 15.7 | AE | 17 |
| 103 | * | * | 15.7 | 15.5 | 17.8 | VE | 20 |
| 104 | * | * | 16 | 16.0 | 19.0 | VE | 22 |
| 105 | * | * | 14.7 | 14.9 | 17.1 | VE | 19 |
| 106 | * | * | 14.2 | 14.3 | 16.2 | VE | 18 |
| 107 | * | * | 14.3 | 14.6 | 16.3 | VE | 18 |
| 108 | * | * | 11.3 | 11.3 | 13.0 | AE | 15 |
| 109 | * | * | 14.7 | 15.4 | 17.8 | VE | 20 |

[1] Including stillwater elevation and effects of wave setup
[2] Due to map scale limitations, BFEs shown on the FIRM may represent average elevation for the zone depicted
[3] North American Vertical Datum of 1988 (NAVD88)
* Data not available

Table 5a. Transect Data Location (continued)

| TRANSECT | STILLWATER ELEVATIONS 10-PERCENT-ANNUAL-CHANCE | 2-PERCENT-ANNUAL-CHANCE | 1-PERCENT-ANNUAL-CHANCE | 0.2-PERCENT-ANNUAL-CHANCE | TOTAL WATER LEVEL[1] 1-PERCENT-ANNUAL-CHANCE | ZONE | BASE FLOOD ELEVATION (FEET NAVD88[2,3]) |
|---|---|---|---|---|---|---|---|
| 110 | * | * | 15.4 | 15.5 | 18.1 | VE | 20 |
| 111 | * | * | 14 | 14.5 | 16.5 | VE | 18 |
| 112 | * | * | 15 | 14.7 | 16.8 | VE | 19 |

[1] Including stillwater elevation and effects of wave setup
[2] Due to map scale limitations, BFEs shown on the FIRM may represent average elevation for the zone depicted
[3] North American Vertical Datum of 1988 (NAVD88)
* Data not available

Figure 1 is a profile for a hypothetical transect showing the effects of energy dissipation on a wave as it moves inland. This figure shows the wave elevations being decreased by obstructions, such as buildings, vegetation, and rising ground elevations and being increased by open, unobstructed wind fetches. Actual wave conditions may not necessarily include all of the situations shown in Figure 1.

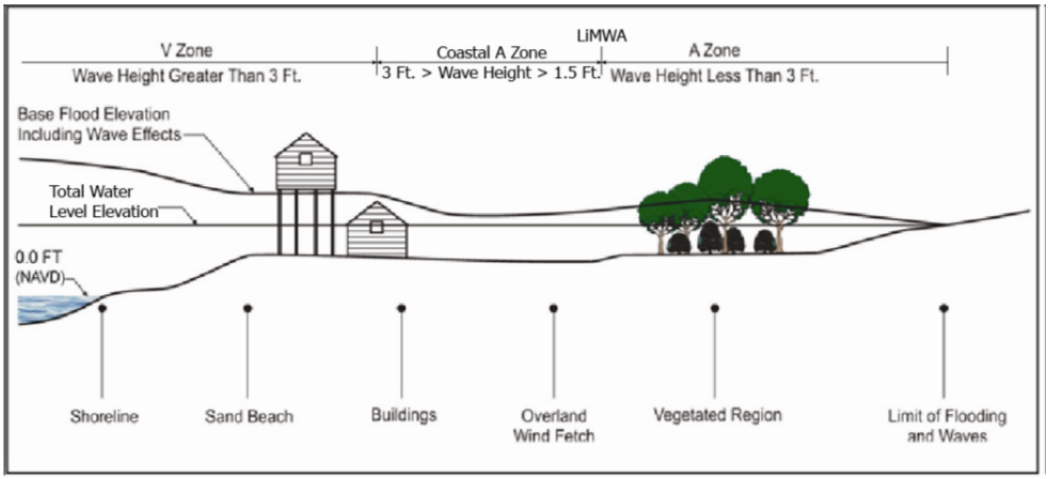


**Figure 1 - Transect Schematic**

Table 5b provides a description of the transect locations, the 1-percent-annual-chance coastal stillwater elevations, and the maximum 1-percent-annual-chance wave crest elevations.

Figure 2, “Transect Location Map”, illustrates the location of the transects for Pierce County.

Table 5b. Transect Descriptions

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| **Transects** | **Location** | **Stillwater 1-percent-annual-chance** | **Maximum Wave Crest 1-percent-annual-chance[1]** |
| 1 | At the shoreline of Commencement Bay, in the Unincorporated Areas of Pierce County, extending from approximately 1,518 feet northeast from Arthur Street Northeast | 10.9 | 13 |
| 2 | At the shoreline of Commencement Bay, in the Unincorporated Areas of Pierce County, extending from approximately 2,029 feet northwest from Poe Avenue Northeast | 11.4 | 14 |
| 3 | At the shoreline of Commencement Bay, in the Unincorporated Areas of Pierce County, extending from approximately 2,229 feet northwest from Wana-Wana Place Northeast | 12.8 | 15 |
| 4 | At the shoreline of Commencement Bay, in the Unincorporated Areas of Pierce County, extending from approximately 1,711 feet west from La-Hal-Da Avenue Norteast | 14.5 | 17 |
| 5 | At the shoreline of Commencement Bay, in the Unincorporated Areas of Pierce County, extending from approximately 2,754 feet southwest from State Highway 509 | 12.7 | 15 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 6 | At the shoreline of Commencement Bay, City of Tacoma, extending from approximately 3,179 feet southwest from State Highway 509 | 12.8 | 15 |
| 7 | At the shoreline of Commencement Bay, in the City of Tacoma, extending from approximately 14,836 feet northwest from North Frontage Road | 11.1 | 13 |
| 8 | At the shoreline of Commencement Bay, in the City of Tacoma, extending from approximately 1,071 feet northeast from North Schuster Parkway | 12.9 | 14 |
| 9 | At the shoreline of Commencement Bay, in the City of Tacoma, extending from approximately 1,625 feet northeast from North Junett Street | 12.7 | 14 |
| 10 | At the shoreline of Commencement Bay, City of Tacoma, extending from approximately 1,337 feet northeast from North Waterview Street | 12.7 | 14 |
| 11 | At the shoreline of Commencement Bay, in the Town of Ruston, extending from approximately 3,724 feet northeast from North Commercial Street | 12.5 | 14 |
| 12 | At the shoreline of Commencement Bay, in the City of Tacoma, extending from approximately 1,456 feet northeast from North Owen Beach Road | 11.9 | 13 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 13 | At the shoreline of Puget Sound, in the City of Tacoma, extending from approximately 876 feet southwest from North Salmon Beach Road | 14.1 | 17 |
| 14 | At the shoreline of Puget Sound, in the City of Tacoma, extending from approximately 1,270 feet northwest from Vista View Drive | 15.2 | 18 |
| 15 | At the shoreline of Puget Sound, in the City of Tacoma, extending from approximately 1,262 feet northeast from North Sunset Street | 15.4 | 18 |
| 16 | At the shoreline of Puget Sound, in the City of Tacoma, extending from approximately 3,439 feet northeast from South 19th Street | 13.7 | 16 |
| 17 | At the shoreline of Puget Sound, in the City of University Place, extending from approximately 1,364 feet northwest from Vista View Drive | 13.9 | 17 |
| 18 | At the shoreline of Puget Sound, in the City of University Place, extending from approximately 1,839 feet west from Soundview Drive West | 15.8 | 19 |
| 19 | At the shoreline of Puget Sound, in the City of University Place, extending from approximately 1,601 feet northwest from Paradise Avenue West | 9.9 | 13 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 20 | At the shoreline of Puget Sound, in the City of University Place, extending from approximately 3,273 feet west from Chambers Creek Road West | 15.3 | 18 |
| 21 | At the shoreline of Puget Sound, in the Town of Steilacoom, extending from approximately 2,027 feet northwest from Puget Drive | 14.6 | 17 |
| 22 | At the shoreline of Puget Sound, in the Town of Steilacoom, extending from approximately 2,299 feet northwest from Rainier Street | 15.8 | 18 |
| 23 | At the shoreline of Puget Sound, in the Town of Steilacoom, extending from approximately 4,037 feet northwest from Union Avenue | 14.8 | 17 |
| 24 | At the shoreline of Puget Sound, in the Town of Steilacoom, extending from approximately 1,226 feet northwest from Chapman Loop | 15.0 | 17 |
| 25 | At the shoreline of Puget Sound, in the Unincorporated Areas of Pierce County, extending from approximately 6,540 feet northeast from Morris Boulevard | 13.3 | 16 |
| 26 | At the shoreline of Puget Sound, in the Fort Lewis Military Reservation, extending from approximately 1,454 feet northwest from Solo Point Road | 14.8 | 17 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 27 | At the shoreline of Puget Sound, in the Town of DuPont, extending from approximately 1,050 feet northwest from Abandoned Railroad | 14.9 | 17 |
| 28 | At the shoreline of Puget Sound, in the Town of Steilacoom, extending from approximately 6,291 feet northwest from Brown Loop | 14.1 | 16 |
| 29 | At the shoreline of Puget Sound, in the Unincorporated Areas of Pierce County, extending from approximately 5,871 feet northwest from Nisqually Road Southwest | No Waves | 12 |
| 30 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 1,187 feet southwest from 134th Avenue | 15.8 | 18 |
| 31 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 1,281 feet southwest from Claussen Road | 14.7 | 17 |
| 32 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 3,922 feet northwest from Sandberg Road | 15.9 | 18 |
| 33 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 4,147 feet northwest from Eckenstam Johnson Road | 12.8 | 14 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 34 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 5,176 feet northwest from 94th Street | 15.5 | 18 |
| 35 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 4,694 feet northeast from Otso Point Road | 15.5 | 18 |
| 36 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 5,660 feet northeast from Admiralty Way | 14.6 | 17 |
| 37 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 5,110 feet east from 106th Street | 14.7 | 17 |
| 38 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 6,685 feet southeast from Oro Bay Road | 13.2 | 15 |
| 39 | At the shoreline of Puget Sound, in Anderson Island, extending from approximately 5,578 feet southwest from Eckenstam Johnson Road | 14.1 | 16 |
| 40 | At the shoreline of Puget Sound, in the Key Peninsula, extending from approximately 3,709 feet south from State Route 302 | 13.9 | 17 |
| 41 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 6,709 feet southwest from 114th Street | 14.2 | 17 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 42 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 4,569 feet southwest from 186th Avenue | 11.2 | 14 |
| 43 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 4,479 feet southwest from 186th Avenue | 13.8 | 17 |
| 44 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 6,690 feet northwest from 92nd Street | 10.9 | 14 |
| 45 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 4,169 feet southwest from Crescent Beach Road | 14.9 | 18 |
| 46 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 7,880 feet southwest from Madrona Beach Lane | 15.1 | 18 |
| 47 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 6,444 feet northwest from 201st Avenue | 15.7 | 18 |
| 48 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,189 feet southeast from East Madrona Boulevard | 14.9 | 17 |
| 49 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 3,781 feet northeast from East Madrona Boulevard | 15.8 | 18 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM

[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| **Transects** | **Location** | **Stillwater 1-percent-annual-chance** | **Maximum Wave Crest 1-percent-annual-chance[1]** |
| 50 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 1,802 feet northwest from Maple Drive | 15.5 | 18 |
| 51 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 3,058 feet southwest from East Madrona Boulevard | 15.4 | 18 |
| 52 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,131 feet northwest from South Herron Road | 14.9 | 17 |
| 53 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,089 feet southwest from 215th Avenue | 12.3 | 15 |
| 54 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 1,550 feet northwest from Whiteman Cove Road | 13.3 | 16 |
| 55 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 1,988 feet southwest from 64th Street Court | 15.2 | 18 |
| 56 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,109 feet west from 176th Avenue Court | 15.6 | 18 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 57 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,819 feet southeast from 163rd Avenue Court | 12.4 | 15 |
| 58 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,473 feet southeast from Yeazell Road | 14.7 | 17 |
| 59 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,670 feet southeast from 156th Avenue | 15.5 | 18 |
| 60 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 5,282 feet east from Yeazell Road | 15.6 | 18 |
| 61 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 1,985 feet southeast from 163rd Avenue | 15.6 | 18 |
| 62 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,778 feet west from 51st Street Court | 13.6 | 16 |
| 63 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 2,285 feet southeast from 45th Street | 14.6 | 17 |
| 64 | At the shoreline of Puget Sound, extending from approximately 2,293 feet northwest from McNeil Island | 15.6 | 18 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 65 | At the shoreline of Puget Sound, in McNeil Island, extending from approximately 4,787 feet southeast from Park Road | 15.4 | 18 |
| 66 | At the shoreline of Puget Sound, in McNeil Island, extending from approximately 7,561 feet northeast from North 16th Avenue | 14.4 | 16 |
| 67 | At the shoreline of Puget Sound, extending from approximately 4,063 feet north from McNeil Island | 13.1 | 15 |
| 68 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 3,963 feet northeast from 150th Avenue | 14.8 | 17 |
| 69 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 7,315 feet northeast from Delano Court | 14.3 | 16 |
| 70 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 9,705 feet northeast from 174th Avenue | 15.0 | 17 |
| 71 | At the shoreline of Puget Sound, in Key Peninsula, extending from approximately 4,915 feet southeast from B Street | 14.4 | 16 |
| 72 | At the shoreline of Carr Inlet, in Key Peninsula, extending from approximately 2,602 feet southeast from Van Beek Road | 12.8 | 15 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 73 | At the shoreline of Carr Inlet, in Key Peninsula, extending from approximately 2,816 feet southeast from 149th Avenue | 14.1 | 17 |
| 74 | At the shoreline of Carr Inlet, in Key Peninsula, extending from approximately 4,235 feet southeast from 146th Avenue | 16.0 | 19 |
| 75 | At the shoreline of Carr Inlet, in Key Peninsula, extending from approximately 4,055 feet southeast from 96th Avenue | 14.9 | 18 |
| 76 | At the shoreline of Carr Inlet, in Key Peninsula, extending from approximately 4,038 feet southeast from 122nd Street | 14.6 | 18 |
| 77 | At the shoreline of Carr Inlet, in Key Peninsula, extending from approximately 3,009 feet southeast from 82nd Street | 13.7 | 17 |
| 78 | At the shoreline of Carr Inlet, in Key Peninsula, extending from approximately 1,763 feet southeast from Goodrich Drive Northwest | 14.7 | 18 |
| 79 | At the shoreline of Carr Inlet, in the Unincorporated Areas of Pierce County, extending from approximately 5,944 feet southwest from State Route 302 | 16.7 | 20 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| **Transects** | **Location** | **Stillwater 1-percent-annual-chance** | **Maximum Wave Crest 1-percent-annual-chance[1]** |
| 80 | At the shoreline of Carr Inlet, in the Unincorporated Areas of Pierce County, extending from approximately 3,999 feet northwest from 71st Avenue Northwest | 16.7 | 20 |
| 81 | At the shoreline of Carr Inlet, in the Unincorporated Areas of Pierce County, extending from approximately 4,608 feet northwest from 104th Street Northwest | 14.6 | 18 |
| 82 | At the shoreline of Carr Inlet in the Unincorporated Areas of Pierce County, extending from approximately 3,612 feet southwest from 92nd Street | 12.2 | 15 |
| 83 | At the shoreline of Carr Inlet, in the Unincorporated Areas of Pierce County, extending from approximately 4,898 feet southeast from 76th Avenue Court | 14.2 | 17 |
| 84 | At the shoreline of Carr Inlet in the Unincorporated Areas of Pierce County, extending from approximately 2,682 feet northwest from Park Avenue Northwest | 15.2 | 18 |
| 85 | At the shoreline of Carr Inlet, in the Unincorporated Areas of Pierce County, extending from approximately 6,965 feet southwest from Raft Island Drive | 14.9 | 18 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 86 | At the shoreline of Carr Inlet, in the Unincorporated Areas of Pierce County, extending from approximately 5,753 feet Northwest from Kopachuck Drive | 15.9 | 19 |
| 87 | At the shoreline of Carr Inlet in the Unincorporated Areas of Pierce County, extending from approximately 2,404 feet northwest from Horsehead Bay Drive | 15.5 | 18 |
| 88 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 1,389 feet south from 115th Avenue Court Northwest | 15.1 | 18 |
| 89 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 3,068 feet southeast from Horsehead Bay Drive Northwest | 18.2 | 21 |
| 90 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 6,087 feet southwest from 36th Street | 15.1 | 18 |
| 91 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 2,785 feet southwest from Shawnee Court Northwest | 14.5 | 16 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 92 | At the shoreline of Puget Sound in Fox Island, extending from approximately 2,987 feet northeast from Bella Vista Drive | 14.5 | 16 |
| 93 | At the shoreline of Puget Sound in Fox Island, extending from approximately 1,724 feet southwest from Island Boulevard | 14.0 | 17 |
| 94 | At the shoreline of Puget Sound in Fox Island, extending from approximately 1,814 feet southwest from Kamus Drive | 11.1 | 14 |
| 95 | At the shoreline of Puget Sound in Fox Island, extending from approximately 2,683 feet south from Queets Place | 13.8 | 17 |
| 96 | At the shoreline of Puget Sound in Fox Island, extending from approximately 3,846 feet northeast from Mowitsh Drive | 14.5 | 16 |
| 97 | At the shoreline of Puget Sound in Fox Island, extending from approximately 3,199 feet northeast from 9th Avenue | 13.6 | 15 |
| 98 | At the shoreline of Puget Sound in Fox Island, extending from approximately 3,353 feet northwest from 7th Lane | 14.4 | 16 |
| 99 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 3,385 feet south from 72nd Avenue Court | 14.4 | 16 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 100 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 2,075 feet southeast from 18th Street Northwest | 14.0 | 16 |
| 101 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 3,084 feet southwest from 38th Avenue Northwest | 14.3 | 16 |
| 102 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 2,342 feet northwest from 42nd Avenue Northwest | 13.0 | 15 |
| 103 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 2,237 feet southeast from A Street Northwest | 15.7 | 18 |
| 104 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 1,464 feet southeast from 23rd Avenue Northwest | 16.0 | 19 |
| 105 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 1,597 feet Northeast from 9th Avenue Court Northwest | 14.7 | 17 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

Table 5b. Transect Descriptions (continued)

| | | Elevation (FEET NAVD88)[2] | |
|---|---|---|---|
| Transects | Location | Stillwater 1-percent-annual-chance | Maximum Wave Crest 1-percent-annual-chance[1] |
| 106 | At the shoreline of Puget Sound in the Town of Gig Harbor, extending from approximately 2,393 feet northeast from Holly Bluff Court | 14.2 | 16 |
| 107 | At the shoreline of Puget Sound in the Town of Gig Harbor, extending from approximately 2,103 feet northeast from Rosedale Street | 14.3 | 16 |
| 108 | At the shoreline of Puget Sound in the Town of Gig Harbor, extending from approximately 1,889 feet northwest from Prentice Avenue | 11.3 | 13 |
| 109 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 1,634 feet southeast from 24th Avenue Northwest | 14.7 | 17 |
| 110 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 1,716 feet southeast from Sea Cliff Drive Northwest | 15.4 | 18 |
| 111 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 904 feet northeast from Point Richmond Drive Northwest | 14.0 | 16 |
| 112 | At the shoreline of Puget Sound in the Unincorporated Areas of Pierce County, extending from approximately 1,165 feet east from 14th Avenue Northwest | 15.0 | 17 |

[1] Because of map scale limitations, maximum wave elevations may not be shown on the FIRM
[2] North American Vertical Datum of 1988

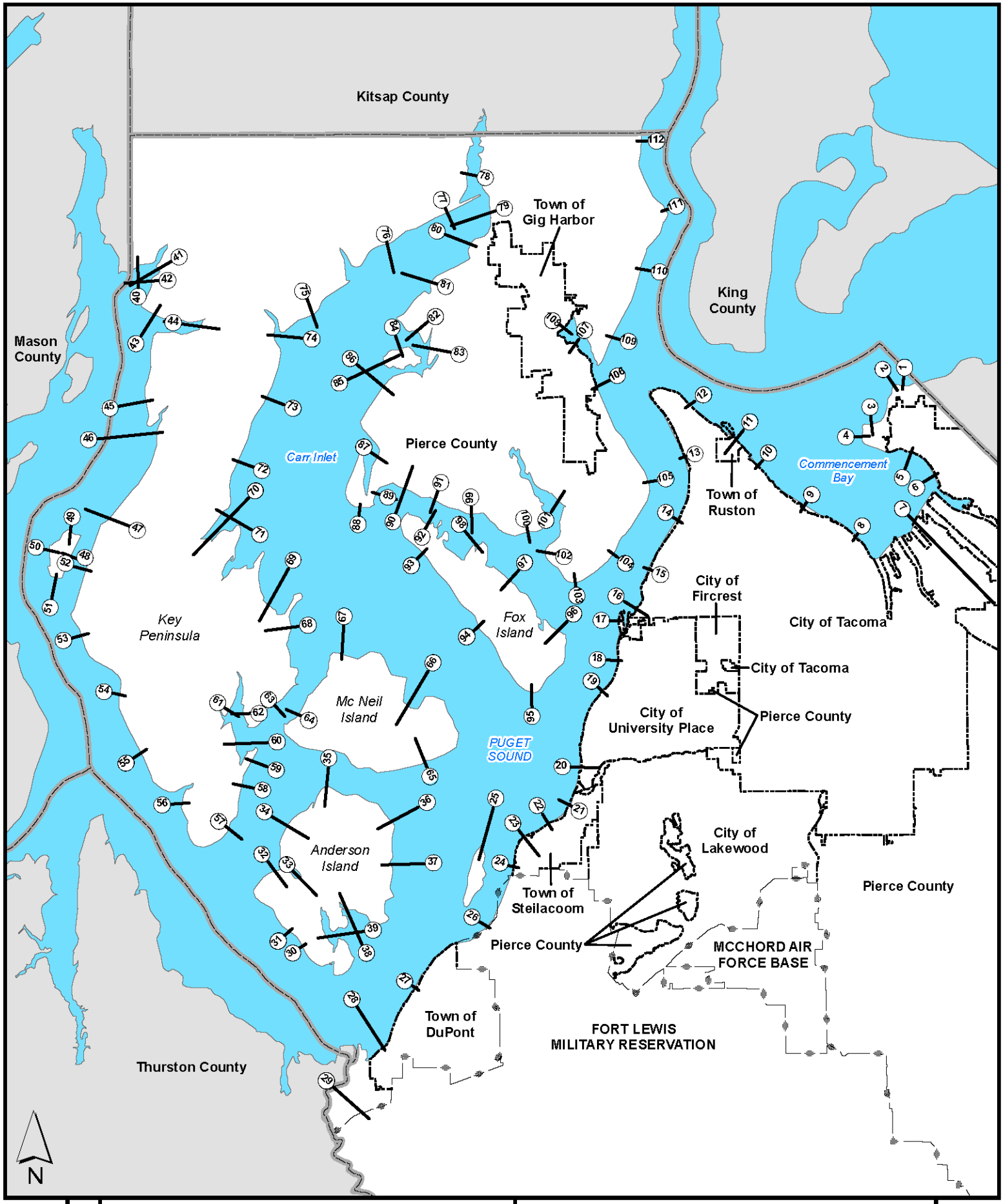


FIGURE 2

FEDERAL EMERGENCY MANAGEMENT AGENCY

# PIERCE COUNTY, WA

AND INCORPORATED AREAS

APPROXIMATE SCALE

0 1 2 4 6 8 Miles

TRANSECT LOCATION MAP

## 3.4 Vertical Datum

All FIS reports and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum used for newly created or revised FIS reports and FIRMs was NGVD29. With the completion of NAVD88, many FIS reports and FIRMs are now prepared using NAVD88 as the referenced vertical datum.

To accurately convert flood elevations from the current NGVD29 datum to the newer NAVD88 datum, the following procedure was implemented. Locations at the upstream and downstream end of the stream, as well as a point to represent the intermediate point between the two end points, were evaluated using the USACE's CORPSCON (Reference 87) datum conversion software. The resulting values for each of the three points were the computed difference between the NGVD29 and NAVD88 elevations. Individual conversion factors at the upstream end, the downstream end, and at an intermediate point, were averaged to develop an average conversion. The final NAVD88 elevations were computed by adding the calculated value to the existing NGVD29 data. Table 6, "Datum Conversion Factors" shows the conversion factors used for the streams and lakes for Pierce County.

**Table 6. Datum Conversion Factors**

| Stream | Vertical Adjustment (feet) |
|---|---|
| Artondale Creek | 3.5 |
| Bonney Lake Outflow | 3.5 |
| Canyon Creek | 3.5 |
| Carbon River | 3.5 |
| Chambers Creek d/s of Leach Creek | 3.4 |
| Chambers Creek u/s of Leach Creek | 3.4 |
| Clarks Creek | 3.5 |
| Clear Creek | 3.5 |
| Clear Creek East Fork | 3.5 |
| Clover Creek | 3.4 |
| Crescent Creek | 3.5 |
| Debra Jane Creek | 3.5 |
| Diru Creek | 3.5 |
| Fennel Creek | 3.5 |
| Greenwater River | 3.7 |
| Horn Creek | 3.4 |
| Hylebos Creek | 3.5 |
| Lacamas Creek | 3.4 |

**Table 6. Datum Conversion Factors**

| Stream | Vertical Adjustment (feet) |
|---|---|
| Leach Creek | 3.4 |
| Little Mashel River | 3.5 |
| Mashel River | 3.5 |
| Meeker Ditch | 3.5 |
| Morey Creek | 3.5 |
| North Creek | 3.5 |
| North Fork Clover Creek | 3.4 |
| Ohop Creek | 3.4 |
| Puyallup River | 3.5 |
| Rody Creek | 3.5 |
| Salmon Creek | 3.5 |
| South Creek Tributary 1 | 3.5 |
| South Creek Tributary 2 | 3.5 |
| South Creek Tributary 3 | 3.4 |
| South Creek Tributary 4 | 3.4 |
| South Prairie Creek | 3.5 |
| Spanaway Creek | 3.4 |
| Squally Creek | 3.5 |
| Swan Creek d/s of cross section I | 3.5 |
| Swan Creek u/s of cross section I | 3.5 |
| Tanwax Creek | 3.4 |
| Unnamed Creek | 3.4 |
| Wapato Creek I | 3.5 |
| Wapato Creek II | 3.5 |
| White River at Buckley | 3.5 |
| White River d/s of cross section AN | 3.9 |
| White River u/s of cross section AN | 3.7 |
| Wilkeson Creek | 3.5 |
| Woodland Creek | 3.5 |
| American Lake | 3.4 |
| Bonney Lake | 3.5 |
| Chambers Bay | 3.4 |
| Debra Jane Lake | 3.5 |
| Gravelly Lake | 3.4 |
| Harts Lake | 3.4 |
| Kreger Lake | 3.5 |
| Lake Kapowsin | 3.5 |

**Table 6. Datum Conversion Factors**

| Stream | Vertical Adjustment (feet) |
|---|---|
| Lake Louise | 3.4 |
| Little Lake | 3.4 |
| Ohop Lake | 3.5 |
| Puget Sound | |
| At Commencement Bay and Salmon Beach | 3.5 |
| At Day Island and Titlow Lagoon | 3.5 |
| At Nisqually Reach | 3.4 |
| At Dupont | 3.5 |
| At Steilacoom | 3.5 |
| Rapjohn Lake | 3.4 |
| Silver Lake | 3.4 |
| Snake Lake | 3.5 |
| Spanaway Lake | 3.4 |
| Steilacoom Lake | 3.4 |
| Tanwax Lake | 3.5 |
| Wapato Lake | 3.4 |

Flood elevations shown in this FIS report and on the FIRM are referenced to NAVD. These flood elevations must be compared to structures and ground elevations referenced to the same vertical datum. For information regarding conversion between the NGVD and NAVD, visit the National Geodetic Survey website at www.ngs.noaa.gov, or contact the National Geodetic Survey at the following address:

*Vertical Network Branch, N/CG13*
*National Geodetic Survey, NOAA*
*Silver Spring Metro Center 3*
*1315 East-West Highway*
*Silver Spring, Maryland 20910*
*(301) 713-3191*

Temporary vertical monuments are often established during the preparation of a flood hazard analysis for the purpose of establishing local vertical control. Although these monuments are not shown on the FIRM, they may be found in the Technical Support Data Notebook associated with the FIS report and FIRM for this community. Interested individuals may contact FEMA to access these data.

To obtain current elevation, description, and/or location information for benchmarks on this map, please contact the Information Services Branch of the NGS at (301) 713-3242, or visit their website at www.ngs.noaa.gov.

## 4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages State and local government to adopt sound floodplain management programs. To assist in this endeavor, each FIS report provides 1-percent-annual-chance floodplain data, which may include a combination of the following: 10-, 2-, 1-, and 0.2-percent annual chance floodplains; and a 1-percent- annual-chance floodway. This information is presented on the FIRM and in many components of the FIS report, including Flood Profiles, Floodway Data tables, and Summary of Stillwater Elevation tables. Users should reference the data presented in the FIS report as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

### 4.1 Floodplain Boundaries

Within this jurisdiction there are one or more levees that have not been demonstrated by the community or levee owner(s) to meet the requirements of 44CFR Part 65.10 of the NFIP regulations as it relates to the levee's capacity to provide 1%-annual-chance flood protection. As such, the floodplain boundaries in this area were taken directly from the previously effective FIRM and are subject to change. Please refer to the Notice to Flood Insurance Study Users page at the front of this FIS report for more information on how this may affect the floodplain boundaries shown on the FIRM.

Please note that the following floodplain boundary delineation descriptions were taken from the August 1987 Pierce County, Unincorporated Areas FIS.

To provide a national standard without regional discrimination, the 1-percent-annual-chance flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent-annual-chance flood is employed to indicate additional areas of flood risk in the community. For each stream studied by detailed methods, the 1- and 0.2-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at a scale of 1:9,600, with contour intervals of 2, 3, and 4 feet (References 48) and at a scale of 1:4,800, with a contour interval of 5 feet (Reference 49). Some boundaries were interpolated using topographic maps at a scale of 1:2,400, with a contour interval of 5 feet (References 50 and 51).

Detailed floodplain boundaries for tidal, lake, and shallow flooding areas were delineated using topographic maps as referenced above.

The boundaries of the 1- and 0.2-percent-annual-chance floods for Debra Jane Lake and Bonney Lake were interpolated using a topographic map at a scale of 1:4,800, with a contour interval of 1 foot, developed from aerial photographs (Reference 47). The flood boundary for the normal full pool elevations of 543 feet on Lake Tapps was delineated by aerial photogrammetry (Reference 47).

The 1- and 0.2-percent-annual-chance flood boundaries for Snake and Wapato Lakes, and along the Puget Sound shoreline were delineated using computed elevations topographic maps (References 52 and 53).

The 1- and 0.2-percent-annual-chance floodplain boundaries are shown on the FIRM. On this map, the 1-percent-annual-chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones A, AE, AH, AO, and VE), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1- and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain has been shown. Small areas within the floodplain boundaries may lie above the flood elevations, but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For the streams studied by approximate methods, only the 1-percent-annual-chance floodplain boundary is shown on the FIRM.

Small areas within the flood boundaries may lie above the flood elevations and, therefore, not be subject to flooding; owing to limitations of the map scale and/or lack of detailed topographic data, such areas are not shown.

For streams studied by approximate methods, the boundary of the 1-percent-annual-chance flood was delineated using topographic maps (Reference 54) and aerial photographs (Reference 43).

For the coastal zones, boundaries were interpolated using LiDAR based topographic data from the Puget Sound LiDAR Consortium.

Approximate 1-percent-annual-chance floodplain boundaries in some portions of the study area were taken directly from the Flood Hazard Boundary Map (Reference 55).

Additional approximate flood boundaries were provided by the Pierce County Public Works Department (Reference 56). These Pierce boundaries were derived from a variety of sources including soils and topographic maps, improvement and basin drainage plans, and aerial photographs.

## 4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local

communities in this aspect of floodplain management. Under this concept, the area of the 1-percent-annual-chance floodplains divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the base flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1 foot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as minimum standards that can be adopted directly or that can be used as a basis for additional floodway studies.

The floodways presented in this study were computed for certain stream segments on the basis of equal-conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated for selected cross sections (see Table 7, Floodway Data.) In cases where the floodway and 1-percent- annual-chance floodplain boundaries are either close together or collinear only the floodway boundary is shown.

The area between the floodway and 1-percent-annual-chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation (WSEL) of the base flood more than 1 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 3.

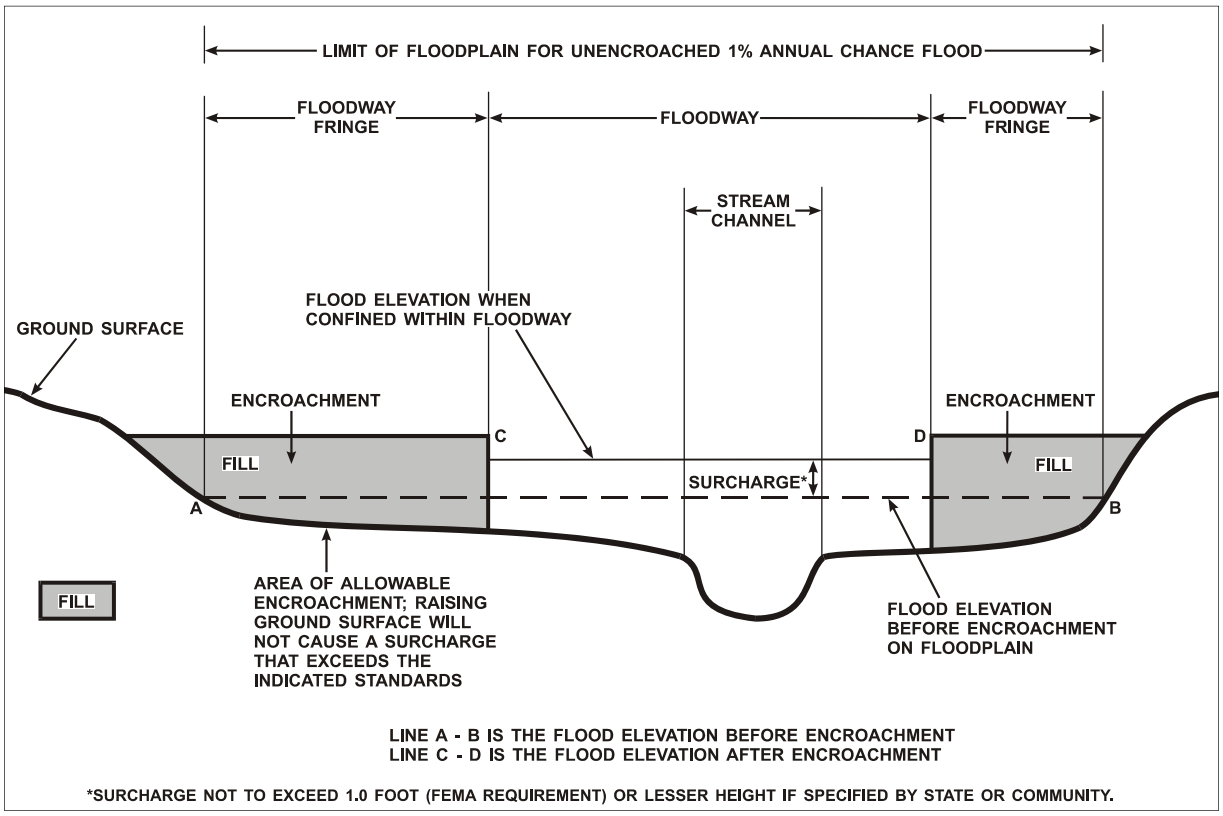


**Figure 3 - Floodway Schematic**

Hydraulic analysis for Wilkeson Creek, Fennel Creek, Debra Jane Creek, and Bonney Lake Outflow revealed that encroachment up to the channel banks will not increase the flood elevations more than 1.0 foot; therefore, the natural channel banks serve as the floodway.

Because the floodways on the above-mentioned streams are defined in this way, no floodway data or delineations are presented.

Floodways were not computed on Swan, Squally, and Clear Creeks downstream of the Burlington Northern Railroad because the flooding in these areas is a result of backwater from the Puyallup River.

Floodway computations are carried out independent of the influence of any other stream. Independent of the influence of the Puyallup River, the 1-percent-annual-chance flood on Clarks Creek used normal depth as a starting water-surface elevation and was determined to be contained in the channel. Consequently, a floodway computation is not necessary for Clarks Creek since the channel banks would act as natural limits of encroachment.

A floodway analysis for North Creek upstream of Harborview Drive would be inappropriate because the 1-percent-annual-chance flood remains within a well-defined channel area. Since flooding of North Creek between Harborview Drive and North Harborview Drive is the result of the

insufficient capacity of an outlet culvert rather than inadequate channel capacity, floodway boundaries were not delineated for this reach either. The study contractor recommends that the 1-percent- annual-chance flood boundaries should, therefore, be considered the limit of allowable encroachment.

Hydraulic analysis revealed that encroachment up to the channel banks will not increase the flood elevations more than 1.0 foot. Therefore, the natural channel serves as the floodway in Wilkeson. Because the floodway is defined in this way, no floodway data or delineations are presented in this study.

Where the channel velocity is high (at or near critical) and/or the flow is confined to the high-water channel, the floodway is shown coincident with the 1-percent-annual-chance flood boundary. This occurs on the following streams: the entire detailed reaches for the Mashel and Little Mashel Rivers, South Creek Tributary No. 2, and Squally Creek; and on portions of Leach, Clover, North Fork Clover, Horn, Unnamed, Lacamas, Swan, Clear, Hylebos, and Salmon Creeks, and South Creek Tributary No. 1. In addition, Pierce County has adopted the 1-percent-annual-chance flood boundary as the floodway on the entire detailed-study reach of Ohop, Tanwax, South Creek Tributary No. 3 and the Greenwater River; and on portions of Horn, Unnamed, Lacamas, Swan, and South Prairie Creeks, and the White River.

On most of the above-mentioned streams where the 1-percent-annual-chance flood boundary and floodway are coincident, no floodway data has been presented on the floodway data table.

The concept of a floodway is not applicable to lake, tidal, and shallow flooding areas.