Final Type A Hydraulic Report

SR 167:
Stage 4 Project (8th St E to S 277th St Southbound HOT Lane)

Prepared for:

Washington State Department of Transportation
Urban Planning Office
King County, Washington

May 2009

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Final Type A

HYDRAULIC REPORT

SR 167/8th St E to S 277th St SB HOT Lane (Stage 4)

MP 10.20 to MP 18.24

PIN 816701C, XL2571, WIN# U16701C

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION
Urban Planning Office
King County, Washington

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May 2009

CERTIFICATION PAGE

Final Type A Hydraulic Report
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WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

May 2009

The engineering material and data contained in this report were prepared under the supervision and direction of the undersigned, whose seal as a registered professional engineer is affixed below.

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Final Type A Hydraulic Report

SR 167: Stage 4 Project (8th St E to S 277th St Southbound HOT Lane)

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Section 1
PROJECT OVERVIEW

The proposed Stage 4 project will add an additional lane to State Route 167 (SR 167) for high occupancy toll (HOT) and is an extension of previous SR 167 HOT lane projects. Stages 1 and 2 consisted of constructing HOT lanes from the north end of SR 167 in Renton to 37th Street NW in Auburn. Stage 3 extended the northbound HOT lanes from 15th Street SW to 15th Street NW.

The Stage 4 project will add a southbound HOT lane from about S 277th Street in Auburn to approximately 8th Street E in Pacific. A vicinity map showing the location of the proposed Stage 4 project is presented in Figure 1-1.

Typically, the Washington State Department of Transportation (WSDOT) will add the HOT lanes by widening the roadway into the median, where there is sufficient room. This will avoid impacting adjacent wetlands and streams. However, in a section of the corridor from about 6th Avenue N to about 5th Avenue SE, the median is not wide enough to accommodate new HOT lanes. Along this section, the roadway will be widened at the outside.

Stage 4 was scheduled to be constructed by 2010 but has been delayed by at least one year. This Type A Hydraulic Report was prepared using the standard outline presented in the WSDOT Hydraulics Manual.

1.1 Site Location

The SR 167 Stage 4 HOT lane project is located in King and Pierce counties. The project begins at S 277th Street at approximately milepost (MP) 18.24 and extends south in the southbound lane through the cities of Auburn, Algona, and Pacific to 8th Street E (MP 10.2).

Table 1-1 presents the sections in which the Stage 4 project is located.

Table 1-1
Project Location

Township	Range	Section
20 N	4 E	2
21 N	4 E	1, 12, 13, 14, 23, 26, 35
22N	4 E	36

The project is located partly in the Green River drainage basin and partly in the White River drainage basin. The runoff from the valley floor is collected in ditches that lead to slow-moving streams (Mill Creek in the Green River basin and an unnamed tributary in the White River basin) before it ultimately discharges into the rivers.

1.2 Scope of Work

This Type A Hydraulic Report provides preliminary stormwater plans for the proposed SR 167 Stage 4 project in accordance with the requirements set forth in the WSDOT Highway Runoff Manual (HRM) and the WSDOT Hydraulics Manual (HM). The report provides preliminary-level design information to establish a basis for cost estimating, permitting, and land acquisition. This report also provides design basis information containing engineering justifications, assumptions, and decisions for preliminary siting and sizing of major drainage treatment and flow control features.

Hydraulic facilities have been proposed for this project in order to manage the resulting changes in stormwater discharge to protect water quality, beneficial uses of the state's waters, and the aquatic environment in the area. The hydraulic features of the SR 167 Stage 4 HOT lane project include detention ponds, floodplain storage, piped collection systems to convey water to flow control facilities, wetland water quality treatment cell, compost-amended vegetated filter strips (CAVFS), and media filter drains.

Note that design figures included in the appendices of this report represent the 100 percent drainage design, which was based on the 60 percent roadway design. The advancement of the roadway design was deferred until a later date. It is acknowledged that some minor adjustments to the drainage design may be required once the roadway design is completed. This will be addressed as part of a future project. It should also be noted that Section 4 of this report documents work that was performed in the spring of 2007 and presented to Ecology. This work included some preliminary analysis of the floodplain storage approach to flow control in the Mill Creek basin. This work was based on the project footprint as of April 1, 2007. Note that the floodplain storage site was developed to accommodate the new impervious area from the future Stage 5 project in addition to the Stage 4 project.

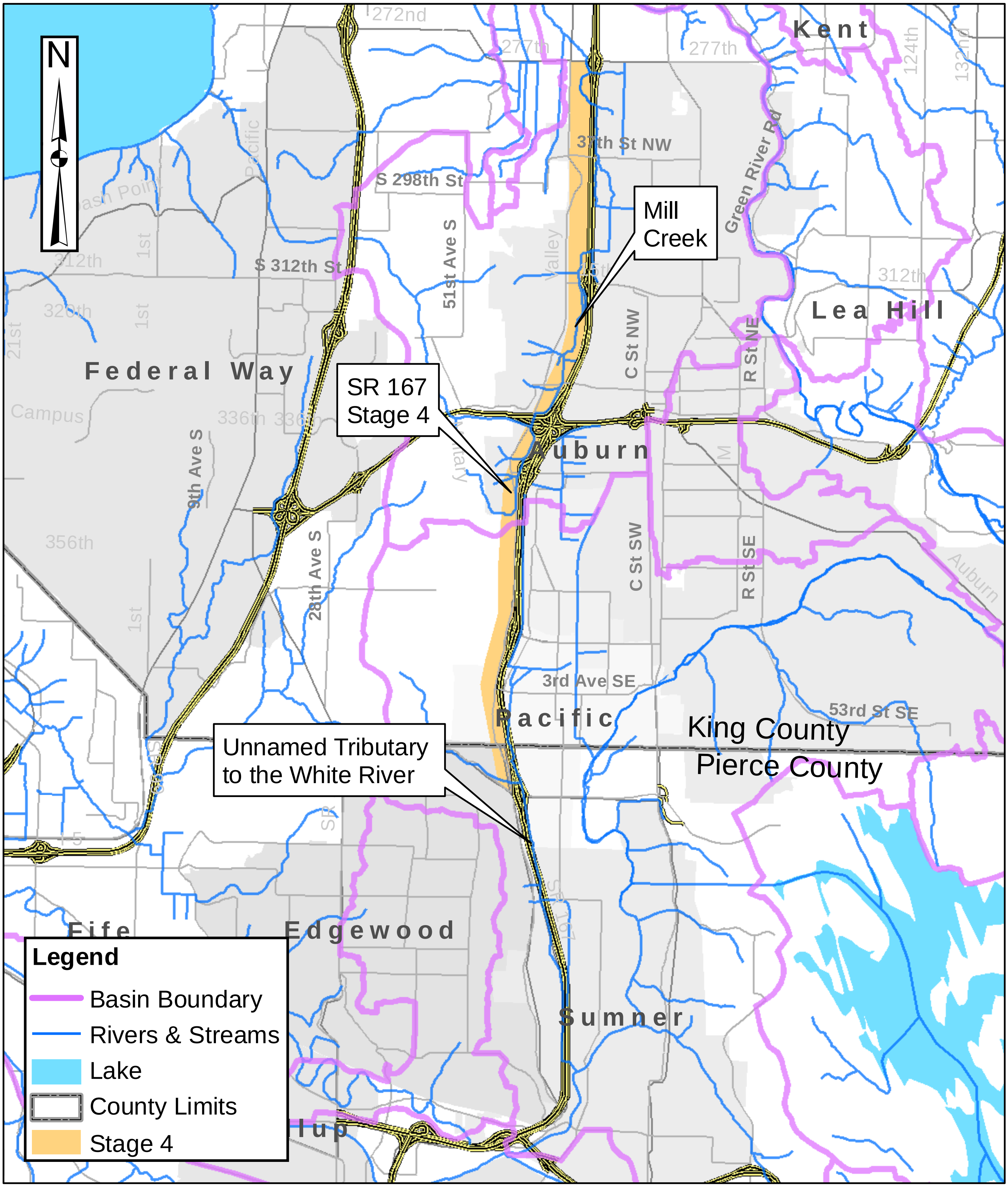

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Figure 1-1
Vicinity Map

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1.3 Areas Impacted

The stormwater impacts from the proposed projects are the result of the impervious surface created by the addition of the HOT lanes. The Stage 4 project increases the amount of impervious area in the SR 167 corridor from about 101.0 acres to about 111.9 acres (an 11 percent increase). The total percent increase in impervious surfaces in the Green River/Mill Creek basin and the basin for the Unnamed Tributary to the White River are shown in Table 1-2.

Table 1-2
Increase in Impervious Surfaces

Basin	Impervious Area of Tributary Basin[1] (acres)	Added Impervious Area from Project[2] (acres)	Increase in Impervious Surface Area (percent)
Mill Creek Basin Tributary to the Green River	1741	7.71	0.4%
Unnamed Creek Basin Tributary to White River	1963	3.26	0.2%
Totals	3704	10.87	0.3%

1. Impervious area estimated from aerial photo (U.S. Geological Survey, 6/13/2002, from http://terraserver.microsoft.com).
2. Increase in impervious surface represents net change in impervious surface for the SR 167 roadway surface based on the project footprint as of the 60 percent design.

Section 2
SITE CONDITIONS

2.1 Existing Conditions

The proposed projects will be located in the Green River and White River valleys (Water Resource Inventory Areas or WRIAs 9 and 10, respectively). The figures in Appendix A show the existing conditions throughout the project corridor. These figures show the existing road alignment, drainage ditches, streams, culvert crossings and detention ponds, as well as the threshold discharge area (TDA) delineation. In addition, the figures show wetlands and existing stream crossings.

Approximately 70 percent of the area covered by the proposed project is located in the Green River basin and drains to Mill Creek, a tributary to the Green River (see Figures 2-1a and 2-1b). Mill Creek runs parallel to SR 167 through a series of ditches and culverts from about 11th Avenue N in Algona to the northern end of the project. The headwaters of Mill Creek are located along the eastern edge of the Federal Way uplands.

From about 11th Avenue N in Auburn, Mill Creek flows north on the east side of SR 167 and enters two large wetlands in the vicinity of the Auburn Supermall. The wetlands are connected by a large culvert under 15th Street SW. From the wetlands, the creek crosses from east to west under SR 167. On the west side of the roadway, Mill Creek is joined by flow from Hill Creek in Peasley Canyon. From there the creek passes under SR 18 and continues north to the limits of the Stage 4 project near S 277th Street. Numerous wetlands line the SR 167 corridor north of SR 18. Some of the wetlands are mitigation sites for projects in the area. Beyond the project limits, Mill Creek flows north and ultimately discharges to the Green River just south of the interchange of SR 516 and SR 167.

Flooding frequently occurs in the Mill Creek subbasin due to high runoff rates from heavily developed basins, high water tables, vegetation-choked drainage ditches, and backwater from the Green River during flood events. Beaver dams have also been an ongoing impediment to conveyance in Mill Creek throughout the project area. WSDOT has removed some of these dams located in the SR 167 right-of-way when they became problems, but on several occasions the beavers have returned and built new dams.

About 30 percent of the project area is within the White River basin and drains to an Unnamed Tributary to the White River (UTWR) (see Figures 2-1a and 2-1b). This tributary appears to have several names, including Milwaukee Ditch, Government Canal, and Soaton Creek. This tributary flows south and runs parallel to SR 167 through a series of ditches and culverts from Algona to the southern end of the project area. From 8th Avenue N, the creek generally runs along the east side of the roadway to just north of 24th Street E in Sumner where the flow is conveyed under SR 167 to the west side. North of 3rd Avenue SW, UTWR joins with flow from a large tributary. At 8th Street E, flow from Jovita Creek joins the UTWR. The creek continues south on the east side of the roadway to the south end of the project. Upstream of the 24th Street E overpass it crosses under SR 167 and flows south to the west of SR 167. As with Mill Creek, beaver dams are a problem along the tributary and have been an ongoing impediment to conveyance throughout the project area.

The project areas within both the Green and White River valleys have extremely flat terrain, high groundwater tables, and adjacent wetlands, which limit the stormwater management options. These constraints make it difficult to find locations to site flow control and water quality treatment facilities to detain and treat the additional surface water runoff from the new impervious surface created by the proposed project. This is particularly an issue in the Green River basin area, where nearly all the roadway is surrounded by wide, valley-bottom wetlands.

The current Federal Emergency Management Agency (FEMA) floodplain delineation for the Green River assumes all the levees along the Green River are effective and that the river does not overtop SR 167. As a result, the projects do not impact the existing floodplain. A new Green River floodplain delineation is pending. The new proposed delineation assumes that the levees are not effective because most of the levees along the Green River have not been certified. It was determined by WSDOT that this proposed floodplain delineation surrounds, but does not overtop, SR 167 within the project corridor.

The White River floodplain does not impact the SR 167 project corridor.

2.2 Existing Hydraulic Features

Site runoff within the current project typically limits flows from the highway to grassy roadside shoulders and then to either roadside ditches or wetlands that abut the highway embankments. Runoff within the existing grassy median either infiltrates into the roadway fill or is carried to the outside shoulder via median drains.

The SR 167 Stage 2 and Stage 3 road projects included the construction of drainage features within the Stage 4 project limits (see Table 2-2 for a complete listing). For example, the Stage 2 and Stage 3 projects installed media filter drains, linear detention ditches, and detention ponds to treat runoff from SR 167 within the Mill Creek basin project corridor (from approximately station 450 to the northern Stage 4 project limit). These existing media filter drains are shown in the figures in Appendix A. Detention ditches constructed as part of the Stage 2 project that also fall within the project corridor collect runoff from the following approximate roadway segments.

- Station 604+00 to 614+00 (SB & NB lanes)
- Station 619+00 to 638+00 (NB lanes)
- Station 626+00 to 638+00 (SB lanes)
- Station 650+00 to 661+00 (SB & NB lanes)
- Station 665+00 to 690+00 (SB lanes)
- Station 670+00 to 690+00 (NB lanes)

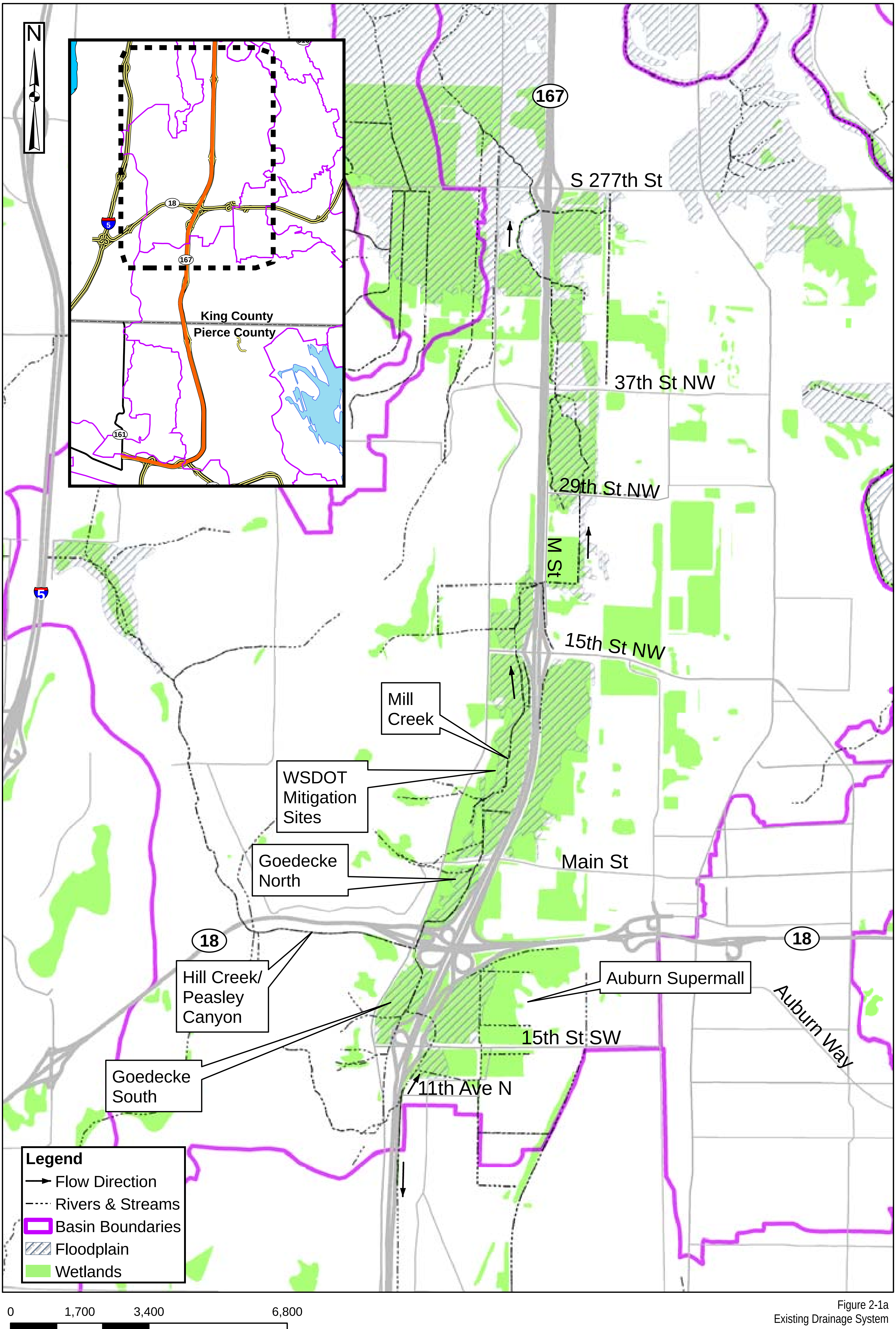

Figure 2-1a
Existing Drainage System

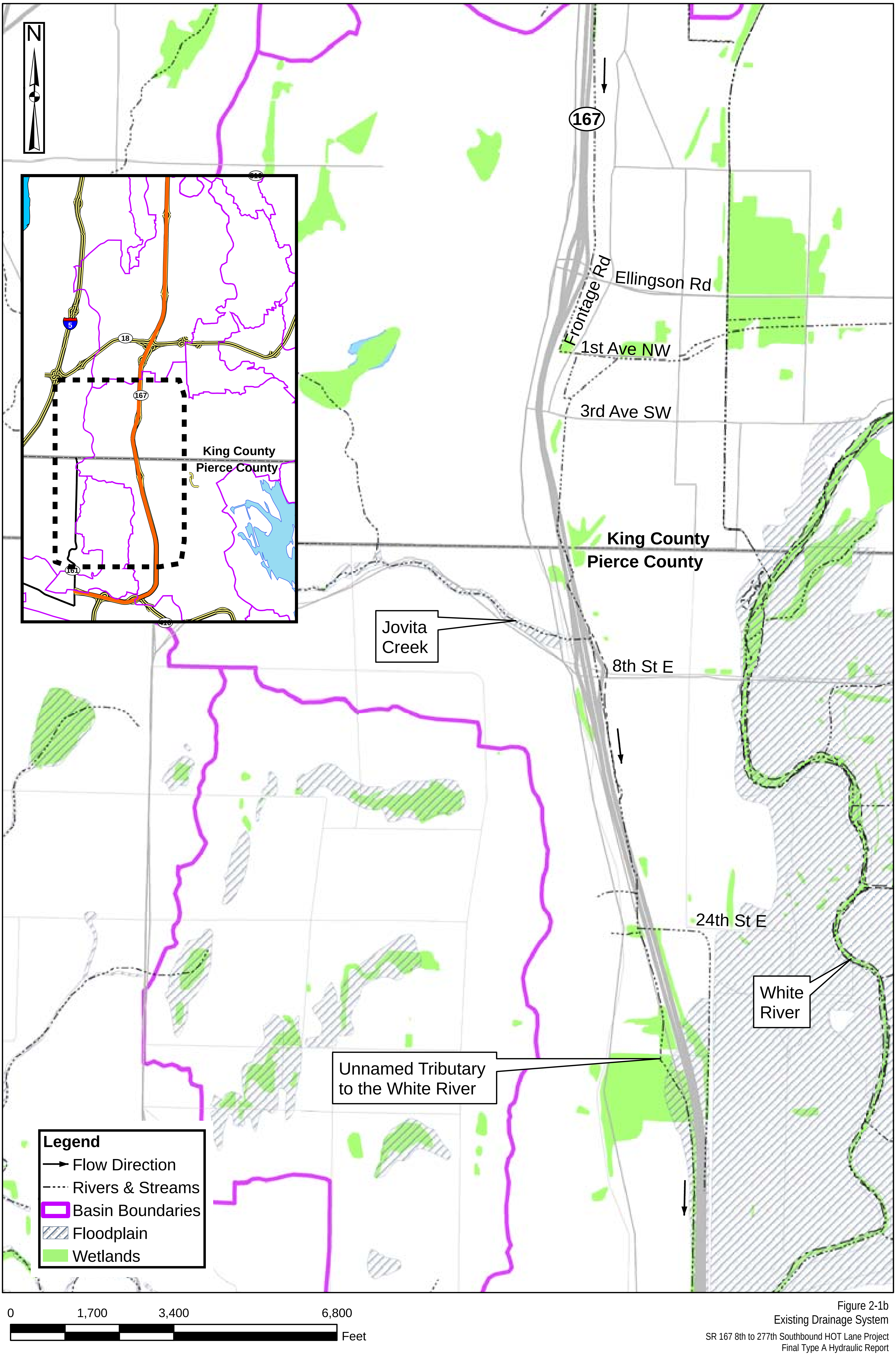

Figure 2-1b
Existing Drainage System

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Two detention ponds were built as part of the Stage 3 project. One pond is at the 15th Street NW interchange and the other is within the existing SR 18 interchange. The existing detention ditches and detention ponds are shown in Appendix A.

Most of the highway runoff from the mainline, ramps, and bridges within the White River basin does not enter any formal stormwater management system. The exception is in the area of 24th Street E in Sumner (south of the current project limits). The SR 167 North Sumner Interchange project built a split diamond interchange that connected SR 167 with an improved 24th Street E. As part of this project, three bioswales and a combination wet/detention pond were constructed. One swale treats runoff along the outside shoulder of the northbound SR 167 off-ramp to 24th Street E. Two other swales are located on the south side of 24th Street E to the east of SR 167 and treat runoff from 24th Street E. The stormwater pond, located just north of the southbound SR 167 off-ramp, treats and detains runoff from an enclosed system along West Valley Highway

There are no mainline or ramp bridges that span waterways within the project limits. The recently constructed overpass for 24th Street E crosses over the UTWR and SR 167. In addition, the UTWR passes under both the on- and off-ramps at 24th Street E via 24-foot-wide, three-sided box culverts.

There are several existing culvert crossings throughout the project limits. The enclosed conveyance structures are denoted on Figures 3-1a and 3-1b by their structure ID (SI). The major stream crossings for Mill Creek occur at:

- Station 504+50 (SI 34) north of 15th Street SW (creek flows westward)
- Station 615+50 (SI 27) north of 15th Street NW (creek flows eastward)
- Station 691+00 (SI 3) south of S 277th Street (creek flows westward)

The major stream crossing for the UTWR occurs beyond the current project limits, at station 290+50 (SI 59), where it flows in a westerly direction under the highway. Other significant channel crossings include:

- The Jovita Creek crossing of SR 167 in the vicinity of 8th Street/Stewart Road near station 334+50 (twin culverts SI 65 and SI 95)
- A small unnamed stream which crosses under SR 167 between West Valley Highway and SR 167 at station 364+00 (SI 73) and discharges immediately into the UTWR

WSDOT and Washington Department of Fisheries have identified both of the above UTWR crossings (SI 73 and twin culverts SI 65 and SI 95) as needing improvements to facilitate fish passage. This is discussed further in Sections 3.4.3 and 4.6.

Several wetland mitigation sites are adjacent to the highway within the project limits. These include:

- The Goedecke North Site on the west side of SR 167 between SR 18 and W Main Street. This site includes the King County Mitigation Tract, the 8th Street NW mitigation site, the Airport Mitigation site, and the 277th Street Mitigation site.

- The Goedecke South Site on the west side of SR 167 between SR 18 and 15th Street SW.
- WSDOT Mitigation Site on the west side of SR 167 between W Main Street and 15th Street NW.

2.3 Existing Threshold Discharge Areas

The threshold discharge areas (TDAs) for the pre-project condition are shown in Appendix A. Each TDA for the projects is defined as the on-site drainage basin meeting all of the following conditions:

- Roadway or drainage improvements are proposed within the on-site basin;
- The on-site basin flows to a WSDOT outfall discharging to a natural drainage system that leaves state right-of-way; and
- The flow distance to the next WSDOT outfall exceeds 1300 feet.

Ramps and crossing roads or highways are only included in TDA area calculations if these areas include one of the following:

- Proposed roadway or drainage improvements associated with the Stage 4 project; or
- Current stormwater management facilities associated with SR 167.

The existing impervious area for each TDA is shown in Table 2-1. The following paragraphs contain detailed descriptions of the TDAs, presented in order from north to south.

TDA M4

TDA M4 comprises the western half of SR 167 from approximately station 663+60 (under the 37th Street overcrossing) to station 707+00. Within this TDA Mill Creek crosses under SR 167 in a large box culvert (SI 3 near station 691+00) and continues north toward the Green River to the west of the highway. All highway runoff from the southbound lanes sheet-flows into a ditch system at the base of the outside shoulder. This ditch was constructed with flow control structures as part of the Stage 2 project such that all existing roadway runoff in TDA M4 is detained (with the exception of the pavement draining to the ditch between stations 684+50 and 686+00). This system outfalls to a wetland at station 684+50 (125 feet LT) that drains to Mill Creek, and directly to Mill Creek at station 691+00 (100 feet LT). Median drainage is conveyed to the outside ditches via storm drains approximately every 600 feet.

Table 2-1
Threshold Discharge Areas - Update

TDA	Existing Impervious Surface (acres)	New Impervious Surface (acres)	Replaced Impervious Surface (acres)	Total New and Replaced Impervious Surface (acres)	Total Impervious Surface (acres)
M1	37.87	3.89	1.45	5.34	41.76
M2	32.45	3.56	1.83	5.39	36.01
M3	11.95	0.26	0.30	0.56	12.21
M4	22.38	0.00	0.00	0.00	22.38
Green River Basin Total	104.65	7.71	3.58	11.29	112.36
W1	33.33	3.26	4.46	7.72	36.59
W2	6.09	0.00	0.00	0.00	6.09
White River Basin Total	39.42	3.26	4.46	7.72	42.68
Totals	144.07	10.97	8.04	19.00	155.04

TDA M3

TDA M3 is comprised of SR 167 between station 620+00 and 663+60 (the eastern half of the TDA appears to start at station 616+50). Runoff from southbound lanes between station 620+00 and 638+15 enters a ditch system that has a flow control structure at station 638+00 (109 feet LT). This discharges to a 30-inch-diameter culvert that conveys runoff eastward to a large ditch on the eastern edge of SR 167 (near 29th Street NW). Runoff from the southbound lanes between stations 620+00 and 638+00 is treated in an media filter drain prior to entering the ditch system.

Between stations 638+15 and 661+00, all southbound runoff is treated by media filter drains. The media filter drains between stations 638+15 and 650+50 discharge to a 24-inch-diameter cross-culvert (SI 18) at station 645+50. (According to as-built drawings, runoff bypassing the media filter drains could enter another cross-culvert at station 647+60 [SI 17]) The media filter drains between stations 650+50 and 661+00 discharge to a ditch with a flow control structure at the outlet at station 661+50. This ditch discharges to a 24-inch-diameter cross-culvert (SI 10) at station 661+75. This culvert discharges to a small ditch that enters Mill Creek just upstream of the 37th Street NW crossing. Off-site runoff and project runoff between station 661+75 and 663+60 enter another 24-inch-diameter cross-culvert (SI 11) at station 661+95.

Storm drains (12-inch-diameter) from the median flow to the outside of the roadway embankment and discharge into existing roadside ditches.

No work is planned along the eastern half of this TDA. Highway runoff leaves the roadway and is treated by media filter drains located along the top of the outside shoulder. With the exception of the highway between stations 638+00 and 649+00, all of the northbound runoff enters detention ditches built under the Stage 2 project.

Existing flow control structures are located at station 638+00 (120 feet RT) and station 661+50 (95 feet RT). These ditches enter the north-flowing Mill Creek system between M Street NW and 37th Street NW.

TDA M2

TDA M2 comprises the southbound lanes and median between stations 529+00 and 620+00 and the northbound lanes from station 536+00 to 617+00. This is roughly the area between SR 18 and the Mill Creek crossing north of 15th Street NW. Runoff from this area is treated extensively by existing media filter drains constructed for Stage 2 and Stage 3 (see Table 2-2 for stationing). Runoff from the southbound lanes discharges to wetlands that fringe Mill Creek. The southbound on- and off-ramps at the 15th Street NW interchange and the off-ramp to westbound SR 18 also discharge directly to Mill Creek or the adjoining wetlands. A detention ditch constructed for Stage 2 detains runoff from station DL2 14+00 to station SR 167 614+00. A pond built for Stage 3 detains runoff between stations 592+00 and 602+50 for the southbound lanes and from the western portion of the 15th Street NW overpass. North-flowing runoff in the median between southbound SR 167 and the NW Line (SR 167 SB off-ramp to SR 18 WB) flows toward Mill Creek via an east-to-west, 12-inch-diameter storm drain near station 540+50 LT.

Runoff from the northbound lanes from station 536+00 to station 553+00 sheet-flows down the outside embankment and into a wetland area. According to as-builts, a 36-inch-diameter cross-culvert at station 541+00 drains the area south of the Main Street overpass to Mill Creek. There is another 24-inch-diameter cross-culvert near station 534+00 that drains to the west (SI 93). From station 553+00 to station 582+00, the roadway is super-elevated and runoff from the northbound lanes flows toward the median. Most of this runoff is treated by existing media filter drains in the median (see Table 2-2 for stationing). Storm drains spaced approximately every 800 feet convey runoff in the median to the large wetland complexes to the east and west of the highway. Two 24-inch-diameter cross-culverts at stations 556+50 and 585+40 (SI 30) convey water from these wetlands on the east side of the highway to Mill Creek on the west. North of station 599 (at the 15th Street NW overpass), runoff from northbound SR 167 is conveyed in a ditch located between the highway and M Street NW that discharges into Mill Creek at the outlet of an 8.5-foot-by-6-foot box culvert (SI 27) near station 615+50. After passing under M Street NW, Mill Creek continues eastward and then northward through agricultural land.

Table 2-2
Existing Flow Control and Runoff Treatment BMPs Within the Limits of the Stage 4 Project

Project	Plan Date	Station 1	Station 2	RT/LT	Drainage Area	Feature
SR 167 15th Street S.W to South Grady Way (covered Stages 1, 1A, 2, & 3)	Hydraulic Report (1993)	Note: Information superceded by later Hydraulic Reports				
SR 167 15th Street SW to S. Grady Way Stage 2	Hydraulic Report Supplement (1995)	AL1 8+30		LT		Pond FCS
		DL1 14+10		RT		Pond FCS
		DL2 14+50	LM 614+00	LT		Bioswale/Detention
		LM 599+00	LM 606+00	LT	SB Mainline	Ecology Ditch
		LM 600+00	LM 627+50	Median		Ecology Ditch
		LM 599+25	LM 602+40	RT	NB Mainline	Ecology Ditch
		LM 607+00	LM 615+00	RT		Ecology Ditch
		LM 615+00		RT		FCS on Existing Ditch
		LM 616+50	LM 649+00	RT		Ecology Ditch
		LM 616+00	LM 627+50	LT		Ecology Ditch
		LM 626+00	LM 638+00	LT		Bioswale/Detention
		LM 638+00		RT		FCS on Existing Ditch
		LM 638+00	LM 661+50	LT		Ecology Ditch
		LM 637+00	LM 644+00	Median		Ecology Ditch
		LM 657+00	LM 663+30	Median		Ecology Ditch
		LM 649+00	LM 661+50	RT		Bioswale/Detention
		LM 661+50		LT		FCS on Existing Ditch
		LM 664+75	LM 684+25	LT		Bioswale/Detention
		LM 671+75	LM 684+25	RT		Bioswale/Detention
		LM 684+65	LM 690+00	LT		Bioswale/Detention
		LM 684+65	LM 689+00	RT		Bioswale/Detention
		LM 692+00	LM 703+50	RT		Bioswale/Detention
SR 167 15th Street SW to S. Grady Way Stage 3	Hydraulic Report Supplement (2005)	LM 467+89	LM 471+95	NB Inside		Ecology Embankment
		LM 467+90	LM 479+72	NB Outside		Ecology Embankment
		LM 475+20	LM 483+49	SB Inside		Ecology Embankment
		LM 495+93	LM 501+77	SB Inside		Ecology Embankment
		LM 501+50	LM 507+46	SB Inside		Ecology Embankment
		LM 505+87	LM 511+70	SB Outside		Ecology Embankment
		LM 504+65	LM 518+50	NB Outside		Ecology Embankment
		SR18 NW 26+37	SR18 NW 38+06	Outside Ramp Shoulder		Ecology Embankment
		LM 531+90	LM 538+21	SB Inside		Ecology Embankment
		LM 546+84	LM 554+60	NB Outside		Ecology Embankment
		LM 546+40	LM 560+15	SB Outside		Ecology Embankment
		LM 554+87	LM 573+50	NB Inside		Ecology Embankment
		LM 561+00	LM 564+00	SB Outside		Ecology Embankment
		LM 566+90	LM 575+59	SB Outside		Ecology Embankment
		LM 573+50	LM 576+10	NB Inside		Ecology Embankment
		LM 577+90	LM 583+15	NB Inside		Ecology Embankment
		LM 578+79	LM 583+66	SB Outside		Ecology Embankment
		15th NW AR2 8+91	15th NW AR2 14+09	Outside Ramp Shoulder		Ecology Embankment
		SR 18 ENS Ramp				Detention Pond
		15th NW I/C				Detention Pond (NW quadrant)
KC 277th I/C Project		Pond in 277th I/C				

TDA M1

TDA M1 extends from station 450+00 to just north of the SR-18 underpass for the southbound lanes. Southbound lanes drain to a ditch along the outside shoulder from station 450+00 to station 475+00 and from station 503+00 to the northern limit of TDA M1 at station 529+00. The southbound lanes are super-elevated toward the median between stations 475+00 and 503+00. Between stations 503+00 and 517+00, the southbound runoff sheet-flows down a large embankment into an existing wetland mitigation site adjoining Mill Creek. The northbound lanes drain to the outside shoulder throughout the entirety of TDA M1.

From station 450+00 until station 479+50, the ditches on either side of the highway are small tributaries to the Mill Creek system. These two channels merge at the outlet from the cross-culvert (SI 37) at 479+50 on the east side of the highway. From there the channel drains northward to the east of SR 167, passes under 15th Street SW, and crosses the highway in a westerly direction near station 504+50 (SI 34). A small tributary channel flowing south, which drains the eastern half of the SR 18 interchange, joins Mill Creek at the inlet to the culvert crossing at station 504+50.

Flows from the western half of the SR 18 interchange enter Mill Creek via an 18-inch-diameter storm drain (not located during field visit) that drains the depression within the southbound SR 167 to eastbound SR 18 loop ramp.

Just outside of the TDAs defined for the Stage 4 HOT lane project, the northbound lanes between stations 521+50 and 536+00 form part of the basin that drains to the newly constructed M1-3 detention pond. The pond discharges to the small south-flowing tributary channel to Mill Creek mentioned above.

TDA W1

TDA W1 comprises a very long highway segment that parallels the receiving body (UTWR) and that typically remains within state right-of-way. The south limits of this TDA are station 302+21. To the south of the current project limits, down to station 290+00, the UTWR flows southward along the west side of the highway. Runoff from the southbound lanes sheet-flows down the roadway embankment. Surface runoff passes through natural vegetation area prior to entering the creek. The creek passes under the 24th Street E overpass and through two large box culverts associated with the southbound SR 167 on- and off-ramps. Northbound lanes, from the southern limits of the project to about station 283+00, drain via sheet flow to a roadside ditch. Flows eventually enter the UTWR via a cross-culvert that drains to the west at station 258+25.

A large culvert (SI 59) at station 290+00 brings the UTWR from the east side of the highway to the west side. From this point until station 370+00, northbound runoff travels down the roadway embankment, through a vegetated buffer, and into the stream. The only exception is that within the 8th Street E diamond interchange, mainline runoff is conveyed via storm drains under the northbound on- and off-ramps and then discharged into the UTWR. Southbound runoff enters a ditch and wetland system between stations 290+00 and 326+00. This area consists of wetlands and depressions that are possibly connected to UTWR via cross-culverts at stations

317+50 (SI 61), 314+00 (SI 106), and 297+00 (SI 108). If these areas were to overflow it is assumed runoff would enter the UTWR near station 291+00. From station 337+00 to station 364+00, southbound runoff enters a ditch that outlets to a 36-inch-diameter corrugated metal pipe (CMP) cross-culvert (SI 69) near station 345+00. North of 364+00 southbound runoff (except in certain super-elevated areas) enters a tributary to the UTWR that crosses the roadway at station 365+00, draining from west to east. Between stations 373+00 and 393+00 the roadway is super-elevated. It appears that the entire roadway drains to a poorly defined ditch on the east side of the highway, which drains to a large depressional area adjacent to the UTWR centered near station 373+00. From station 393+00 to station 410+00, the roadway is on a tangent and it drains to the outside. From station 408+00 to station 428+00 the roadway is super-elevated and drains via sheet flow to the west. Northbound lanes drain to the median in the super-elevated section and likely drain to the west side of the roadway (although this has not been confirmed). The divide between the White River and Green River is not very distinct. It occurs somewhere near station 450+00. Between stations 427+00 and 450+00, southbound lanes drain to the west and into a poorly defined ditch that flows to the south.

TDA W2

There is a small portion of the project in the White River basin that is distinct from TDA W1. TDA W2 comprises the northbound lanes between stations 395+00 and 408+00 and between stations 425+00 to 450+00, which drain to the UTWR within WSDOT right-of-way along the east side of SR 167. The UTWR leaves the right-of-way near station 395+00 and joins with a large tributary just north of 3rd Avenue SW in the vicinity of the Interurban Trail.

2.4 Soils

Much of the project is proposed to be located on previously placed fill. Outside of areas that have been filled, the valley floor is typically comprised of Alderwood soil, which has very low permeability. In addition, some areas contain Seattle Muck and Norma soils, both of which have seasonal high groundwater and poor drainage.

Three types of geotechnical fieldwork have been conducted for this project by WSDOT staff from the Northwest Region Materials Lab:

- Newly installed piezometers are monitored monthly at the proposed detention pond and floodplain storage sites.
- Soil cores were collected at the piezometer locations.
- Shallow soil cores were collected at representative sites along the edge of shoulder where certain runoff treatment BMPs are proposed.

Piezometer information and shallow soil cores have been collected. Piezometers were installed and shallow soil core samples were taken to provide information for the preliminary design of the drainage features. The locations of the piezometers and soil core samplings are shown on the figures in Appendix B. The piezometer readings were used to determine the groundwater elevation at pond locations. Piezometer readings can be found in Appendix B. Soil information associated with the piezometer installations will provide information regarding design infiltration rates through the pond bottom during final design.

The shallow soil core samples were used to determine the infiltration rate for the design of compost-amended vegetated filter strips. The results of the soil core sampling are documented in Appendix B.

2.5 Outfalls and Enclosed Drainage Characterization

The enclosed drainage discharging to a location off WSDOT right-of-way or into ditches and water bodies within WSDOT right-of-way were inventoried by Jones and Stokes during their field work. This data was reviewed by R. W. Beck (based on information in the existing base map and collected during downstream analysis fieldwork) and is summarized in Table 2-3. Table 2-3 assigns a structure ID (SI) to each enclosed drainage structure, and these SIs are used throughout this report. The discharge locations are shown on the figures in Appendix A. In addition, the existing enclosed conveyances within the project limits were classified in terms of the function of the conveyance. However, to best evaluate the type and functionality of each these conveyances, we also relied upon the following additional sources of information:

- WSDOT’s existing drainage base map
- Culvert survey data collected by Perteet, Inc., in 2007 (Jones & Stokes 2008)
- Field data collected by Perteet, Inc., during wetland survey work in 2007
- Downstream analyses and the limited field inspection done by R. W. Beck during preparation of this report.

Table 2-3
Summary of Enclosed Drainage Information for the Stage 4 Project

Structure ID (SI)	GPS Lat	GPS Long	Mile Post	Size, material, shape (if known)	Flow direction	Discharges directly into a stream	Category	Notes
1	47.35244	122.24545	c. 18	18" Dia	W	Y	S	Outlets to Mill Creek; drains interchange & median
2	47.35223	122.24539	c. 18	Probably a 4'x4' box constructed for Stg 3	W	Y	W	Connects Wetland to Mill Creek; submerged
3	47.34773	122.24516	17.8	15' wide by 7' tall Concrete Box Culvert	W	Y	C1	Conveys Mill Creek under SR167
4	47.34741	122.24516	c. 17.6	12" Dia CPP	N/W	?	P	Inlet and outlet from detention ditch control structure
5	47.34597	122.24515	c. 17.5	12" Dia CPP (Same Pipe as No. 7)	N/W	N	P	Outlet from a detention ditch
6	47.34593	122.24527	c. 17.5	12" Dia Concrete	W	N	S	Drains from median
7	47.34581	122.24525	c. 17.5	12" Dia CPP (Same Pipe as No. 5)	N/W	N	P	Outlet from detention ditch control structure
8	47.34307	122.24519	c. 17.3	12" CMP	W	N	S	Drains from median into detention ditch
9	47.3414	122.24522	c. 17.2	12" Dia CMP	W	N	S	Drains from median into detention ditch
10	47.33978	122.2453	16.9	24" Dia Concrete (Same as culvert 100)	E	N	D (W?)	Conveys ditch (or floodplain area) flow to east side of roadway; 10 & 11 could be reversed
11	47.33971	122.24532	16.9	24" Dia DI or Steel (Same as culvert 101)	E	N	D (W?)	Conveys ditch (or floodplain area) flow to east side of roadway; 10 & 11 could be reversed
12	47.33964	122.24529	16.9	12" Plastic Pipe	N	N	P	Outlet from detention ditch control structure
13	47.33957	122.24532	16.9	12" Plastic Pipe	N	N	P	Inlet to detention ditch control structure (or could be outlet from ecology embankment)
14	47.33894	122.24529	c. 17					No Culvert Found
15	47.33809	122.24529	c. 17	12" Plastic Pipe	W	N	S (P?)	Likely from median although different side from Stg 3 plans (possibly ecology embankment underdrain into detention ditch)
16	47.33659	122.24525	c. 16.9					No Culvert Found
17	47.33583	122.24538	c. 16.8	18" Dia CMP (24" on design dwg.)	E	N	D (W?)	Conveys ditch (or floodplain area) flow to east side of roadway
18	47.3352	122.24538	16.8	24" Dia CMP with asphalt lining	E	N	D (W?)	Conveys ditch (or floodplain area) flow to east side of roadway; 18 & 19 could be reversed
19	47.33326	122.24537	c. 16.6	12" Dia CMP (30" on Stg 2 dwg.)	W	N	D (W?)	Conveys ditch (or floodplain area) flow to east side of roadway; 18 & 19 could be reversed
20	47.33322	122.24538	c. 16.6	18" Dia CPP	N	N	P	Outlet from detention ditch control structure
21	47.33313	122.24538	c. 16.6	18" Dia CPP	N	N	P	Inlet to detention ditch control structure
22	47.33205	122.2453	c. 16.6	18" Dia CPP (12" on Stg 2 dwg.)	W	N	S	Drains from median into detention ditch
23	47.33018	122.24536	c. 16.4	12"	W	N	S	Drains from median into detention ditch; completely submerged
24	47.33017	122.24533	c. 16.4	12" Dia CPP	N	N	P	Conveys flow from Ecology Embankment into detention ditch
25	47.32826	122.24537	c. 16.3	12" Dia CMP	W	N	S	Drains from median
26	47.32718	122.24538	c. 16.2	12" Dia CPP	W	N	P	Conveys flow from Ecology Embankment into roadside ditch
27	47.32691	122.24538	c. 16.2	8.6' Wide by 6' Tall Concrete Box Culvert	E	Y	C1	Conveys Mill Creek under SR167
28	47.32682	122.24549	c. 16.2	24" Dia CPP	N	Y	P	Outlet from detention ditch control structure into Mill Creek
29	47.3266	122.24535	c. 16.2	24" Dia CPP	N	N	P	Inlet to detention ditch control structure
30	47.3188	122.24551	15.6	24" Dia CMP	W	?	W	
31	47.30386	122.25268	c. 14.5	12" Dia CMP	S	N	S	Conveys flow from inside SR167 ramp area to inside SR18 ramp area
32	47.3005	122.25471	c. 14.2	12" Dia CMP	W	N	S	Drains from median
33	47.29898	122.25565	c. 14.1	12" Dia CMP	W	N	S	Drains from median
34	47.29819	122.2562	c.14	Twin 10'w by 5'h with 8" wall between; Concrete Box Culvert	W	Y	C1	Conveys Mill Creek under SR167
35	47.29459	122.258	c. 13.8	10" Dia CPP	W	N	S	Conveys inside ramp to ditch/wetland.
36	47.29328	122.25848	13.7	12" CMP	W	N	S	Could not find on Basemap; possibly from median or off-site

Table 2-3 (continued)

Structure ID (SI)	GPS Lat	GPS Long	Mile Post	Size, material, shape (if known)	Flow direction	Discharges directly into a stream	Category	Notes
37	47.29144	122.25868	13.6	6'w by 4'h Concrete Box Culvert	E	Y	C1	WDFW ID991223. WIDENING TO MEDIAN AND OUTSIDE AT THIS LOCATION
38	47.29073	122.25873	13.5	12" CMP	E?	N	D	Not found on Basemap. Same as culvert No. 87. Probably a cross-drain
39	47.2903	122.25855	13.5					Not found on Basemap. Not found by J&S. Same as No. 86
40	47.28942	122.25859	13.4					Not found on Basemap. Same as culvert No. 85. Not found by J&S.
41	47.28765	122.25857	13.3					Not found on Basemap. Same as culvert No. 84. Not found by J&S.
42	47.28634	122.25871	13.2	12" CMP	W	N	S	Not found on Basemap. Found by J&S. Probably a median drain.
43	47.28513	122.25871	13.2	12" CMP	W	?	C2	Not found on Basemap. Found by J&S. CMP - enters at road, buried 17 feet, exits near stream. Could connect roadside ditch with stream or other ditch
44	47.27545	122.25878	c. 12.6	8" CPP	W	N	S	Drains into inside of ramp
45	47.27437	122.25909	c. 12.5	8" CPP	W	N	B	Drains into inside of ramp
46	47.27373	122.25941	c. 12.4	18" CMP	W	N	C2	Not found on Basemap. Found by J&S. Runs under off-ramp from SB-167
47	47.27332	122.25943	12.4	12" CMP	W	N	B	Drains into inside of ramp
48	47.2729	122.25957	c. 12.4	12" CMP	W	N	S	Drains into inside of ramp
49	47.2727	122.25981	c. 12.4	18" CMP	W	?	S	Drains inside of ramp to tributary or off-site ditch
50	47.27082	122.26084	c. 12.2	12" CMP	W	?	D	Connects roadside ditch to tributary or off-site ditch (or unlikely goes across to Milwaukie Ditch)
51	47.25943	122.26059	c. 11.4					Not found on Basemap. Not found by J&S.
52	47.25023	122.25819	c. 10.6	18" CMP	W	N	C2	Not found on Basemap. Found by J&S. Drains under 8th Street on-ramp to SR-167 SB
53	47.23212	122.24982	9.32	36" Diameter CMP	W	?	W	Conveys wetland/ditch to Milwaukee Ditch under SR167
54	47.23464	122.25101	9.5	48" CPP	E	N	D	Not found on Basemap. Found by J&S. Under SR-167 NB off-ramp to 24th Street
55	47.23533	122.25136	9.6	36" CPP	S	N	C2	Conveys flow from north of overpass to south
56	47.23597	122.25156	9.8	18" CMP	SW	N	P	Pond outlet
57	47.23694	122.25197	9.8	36" Steel Pipe	E?	N	S	Conveys flow from inside median to ramp area?
58	47.23685	122.25168	9.8	24" CPP	W	N	C2	Conveys flow from wetland\roadside ditch to inside of ramp
59	47.2406	122.25364	10	Twin 14' by 9' CMP Arches	W	Y	C1	Conveys Milwaukee Ditch under SR167. WDFW ID 991211
60	47.24525	122.25536	10.3	24" dia CMP	E	?	W	Not found on Basemap. Found by J&S.
61	47.24778	122.25597	c. 10.4	24" CPP	E	N	S	Probably drains median and GORE on west side to stream on east side.
62	47.249	122.25658	c. 10.5	12" CMP	E	N	S	Not found on Basemap. Found by J&S. Probably drains median
63	47.2501	122.25668	c. 10.6	12" CMP	E	N	S	Not found on Basemap. Found by J&S. Probably drains interchange area to stream
64	47.25093	122.25666	c. 10.7	18" CMP	E	N	S	Not found on Basemap. Found by J&S. Probably drains interchange area to stream
65	47.25237	122.25726	10.8	84" Diameter CMP	E	Y	C1	Conveys Jovita Creek to Milwaukee under SR167. WDFW ID 105 R050320a. Just south of culvert 95
66	47.25156	122.25726	c. 10.7	12" CMP	E?	N	S	Conveys flow between SR167 ramp areas

Table 2-3 (continued)

Structure ID (SI)	GPS Lat	GPS Long	Mile Post	Size, material, shape (if known)	Flow direction	Discharges directly into a stream	Category	Notes
67	47.25221	122.25768	c. 10.7	12" CMP	E	N	S	Not found on Basemap. Found by J&S. Conveys flow in or out of interchange area
68	47.25433	122.25809	10.9					
69	47.25517	122.2584	11	48" CMP	E	?	D	Not found on Basemap. Found by J&S. Conveys drains ditch on west side of SR-167
70	47.2561	122.25855	11	12" CMP	E	?	S	Not found on Basemap. Found by J&S. Probably median drains but could be small cross-culverts
71	47.25734	122.25893	11.1	12" CMP	E	?	S	Not found on Basemap. Found by J&S. Probably median drains but could be small cross-culverts
72	47.25903	122.2598	11.3					Not found on Basemap. Not found by J&S.
73	47.26033	122.26001	11.3	6' by 4' Concrete Box Culvert	E	Y	C1	Conveys trib to Milwaukee under SR167. WDFW ID 996290
74	47.26188	122.26035	11.4	12" CMP	E	?	S	Not found on Basemap. Found by J&S. Probably a median drain.
75	47.2647	122.26148	11.6	8" Concrete	E	N	B	Drains roadway into roadside ditch
76	47.26486	122.26139	11.6	12" CMP	E	N	B	Drains roadway into roadside ditch
77	47.2653	122.26146	11.7	12" CMP	E	N	B	Drains roadway into roadside ditch
78	47.26534	122.26309	11.8	18" Concrete	?	?	?	Not found on Basemap. Found by J&S. Appears to be outside of r/w
79	47.27571	122.25762	12.4	12" CMP	E	?	C2	Not found on Basemap. Found by J&S. Probably connects roadside ditch to stream/large ditch
80	47.27939	122.258	12.6	12" CMP	E	N	B	Not found on Basemap. Found by J&S. Drains bridge approach
81	47.27942	122.25773	12.7	18" CMP	E	?	C2	Not found on Basemap. Found by J&S. Connects roadside ditch to stream/large ditch under access road.
82	47.28693	122.25797	13.2	Probably a 12"	E	N	S	Drains from median. Culvert not found by J&S
83	47.28751	122.25794	13.2					Not found on Basemap. Not found by J&S.
84	47.28764	122.25793	13.2					Not found on Basemap. Same as culvert No. 41. Not found by J&S.
85	47.28938	122.25797	13.4					Not found on Basemap. Same as culvert No. 40. Not found by J&S.
86	47.2903	122.2579	13.4					Not found on Basemap. Not found by J&S. Same as No. 39
87	47.29086	122.25764	13.5	12" CMP	E?	N	D	Not found on Basemap. Same as culvert No. 38. Probably a cross-drain
88	47.29115	122.25787	13.5	Probably a 12"	NE	N	P	Median drain. Not found by J&S.
89	47.29149	122.25777	13.5	6'w by 4'h Concrete Box Culvert	E	Y	C1	Same Culvert as No. 37
90	47.29247	122.25759	13.6	Probably a 12"	E	N	S	Drains from median. Not found by J&S
91	47.29399	122.25736	13.7	Inlet Grate	E	?	S	Drains from median and inside ramp
92	47.31519	122.24557	15.3	24" CPP	W	N	P	Conveys flow from Ecology Embankment into roadside ditch or wetland
93	47.30561	122.25179	c. 14.6	24"	W	Y	W	Conveys flows from wetland under SR167 to Mill Creek
94	47.25003	122.25582	on off ramp @ 8th st	21' by 10' Double Concrete Box Culvert	S	Y	C1	Conveys Milwaukee Ditch under 8th St E. WDFW ID 105 R050320b
95	47.25232	122.25729	10.8	84" Diameter CMP	E	?	C1	High flow conveyance of Jovita Creek to Milwaukee under SR167. WDFW ID 105 R050320a. Just north of 95, could be clogged with debris
96	47.27295	122.25801	on Ellingson Rd. east of 167	3' by 2' Twin CMP	S	Y	C1	Conveys Milwaukee Ditch under Ellingson Rd on east side of SR-167

Table 2-3 (continued)

Structure ID (SI)	GPS Lat	GPS Long	Mile Post	Size, material, shape (if known)	Flow direction	Discharges directly into a stream	Category	Notes
97	47.27306	122.26028	on Ellingson Rd. west of 167	4.5 feet CST	S	?	C1	Conveys tributary or off-site ditch flows under Ellingson Rd on west side of SR-167
98	47.33977	122.24438	16.9					
99	47.33976	122.24435	16.9					
100	47.33975	122.2444	16.9					
101	47.33971	122.24437	16.9					
102	47.34594	122.24433	17.5					
103	47.3474	122.24456	17.6					
104	47.32716	122.24448	16.2					
105	47.31517	122.24582	15.3					
106	47.24733	122.25552		54" dia CMP	E	?	W	WDFW ID 996288. Not found on Basemap. Found by J&S.
107	47.31511	122.24467	15.3					
108	47.24227	122.25441	10.1	30" dia CMP	E	?	W	WDFWID 992999 - non fish-bearing. Not found on Basemap. Found by J&S.

Note: This table is based on best available information. For certain conveyances conflicting information is available so professional judgement was used to characterize the conveyance.

Enclosed Drainage Category Key	
C1	culvert that conveys stream or large off-site ditch flows
C2	small cross-culvert under a ramp or access road
W	enclosed conveyance between significatn wetland or floodplain areas; sometimes the inlet/outlet are beyond r/w limits; may outlet directly to a stream
D	enclosed conveyance between parallel highway ditches (no or minor off-site contribution)
P	storm drain directly connected to a stormwater mgmt facility such as a detention pond or ecology embankment.
B	bridge drain or outlet from an overpass area
S	on-site storm drain outlet (includes drains from median or ramps)
t	no evidence of a conveyance from any of the available sources (may have been based solely on pavement markings)
t	not found on the existing drainage basemap (as of 10/07)

2.5.1 Classification

Based on the surrounding land areas, drainage pathways, and types of conveyances found within the project limits, the following functional classes of enclosed conveyances were identified:

Type C1: Culverts that convey *off-site channel flows* through roadway fill.

Type C2: Culverts under an on- or off-ramp. These small culverts drain an enclosed area within an interchange. This designation is also used for small culverts connecting two ditches on the same side of the highway.

Type W: Culvert (or, rarely, a storm sewer) that connects large wetland or floodplain areas to one another or directly to a stream. Sometimes the inlet/outlet is beyond the state right of way limits. This designation is also used to denote culverts connected to a large ditch to a stream.

Type D: Culvert or storm sewer connection between ditches on either side of highway. No or minor off-site contribution to these flows. Where noted, these culverts connect a ditch directly to a stream.

Type P: Storm sewer directly connected to a stormwater management facility such as a detention pond or media filter drain. Usually the storm pipes in and out of the control structure are the only visible conveyance.

Type B: Bridge drain or drain from a bridge approach. These are usually small-diameter pipes which drop about 20+/- feet at a relatively steep grade to the discharge point.

Type S: On-site storm sewer outlet other than Type P or B (includes drains from medians or ramps).

For the purpose of this report, a culvert is defined as closed conduit that is open on both ends (i.e., has no structure attached to either the inlet or outlet) and conveys flow through an artificial embankment. Structures greater than 20 feet in width are generally referred to as bridges. Culverts are seldom connected to the enclosed storm drainage system associated with the roadway.

2.5.2 Additional Considerations

A wide variety of storm drain and culvert configurations can be found within the project limits. At one extreme is the rather unusual (and relatively complex) storm drain system associated with SR 167 north of the Main Street crossing. This system contains storm drains for existing detention ditches and water quality BMPs as well as a parallel system of ditches and culverts for off-site flows. The generally low gradient of the entire area adds to the difficulty of determining flow paths and basin areas based on field inspection alone.

The figures in Appendix A illustrate the location of conveyances based on the sources of information listed in Section 2.5. Table 2-3 represents our best characterization of the enclosed drainages within the project limits. Some additional considerations for the user of this information include:

- Conveyance types C1, D, and W allow water to move freely from one side of the highway embankment to the other without any sort of manhole, grate inlet, drop inlet, or catch basin. The only exception might be on the inlet end of Type D crossings where a drop inlet could be present.
- Where possible, conveyances discharging *directly* to Mill Creek or the UTWR have been identified. However:
  - Culverts or drains discharging to a ditch that connect to the stream channel are not listed as directly discharging to a stream. However, in certain areas these ditches may be subject to backwater from the stream.
  - It can be difficult to determine from the base map alone if a drain discharges below the ordinary high water mark of a stream. If this determination is critical then a field inspection is recommended.

2.6 Existing Utilities

Existing utilities that conflict with proposed storm drain improvements proposed by this project are shown on the proposed conditions figures in Appendix C. The conflicting utilities include a sewer line that crosses the floodplain storage site. The sewer will be relocated to within the West Valley Highway Right-of-Way as part of this project.

In addition, there is a sewer line in the vicinity of proposed Pond W1-4. It appears that the sewer is under the northern pond embankment and will not be subject to traffic loading and therefore does not need to be encased.

Section 3
DEVELOPED CONDITIONS

This section summarizes the proposed developed conditions within the project subbasins as well as the stormwater improvements proposed for the project. This section describes the design standards, downstream analysis, and impervious area for each threshold discharge area (TDA). The hydraulic and hydrologic analyses and detailed sizing analysis are described in Section 4.

3.1 Proposed Drainage Basins

The TDAs for the proposed project conditions are shown in Appendix C. Table 2-1 shows the existing impervious area, new impervious area, and replaced impervious surface in each TDA. Sections 2.2 and 2.3 include a written description of the pre-project conditions.

The following paragraphs describe the post-project conditions. While the impervious areas change due to the proposed road widening, the flow paths and conveyance types within each TDA remain unchanged from pre-project conditions, except as noted below.

3.1.1 Mill Creek Basin (TDAs M1, M2, M3, and M4)

No on-site drainage modifications are proposed within the Mill Creek basin except for providing runoff treatment best management practices (BMPs) within the existing roadside embankments. Flow control is proposed at a location outside the roadway right-of-way.

Storm drainage and outfalls: Median drains will need to be adjusted due to impacts associated with roadway fill placement. Survey information was not available to check the capacity of the median drains that receive additional runoff from the new impervious surface. This will need to be checked once survey becomes available. No modifications to outfalls are proposed within the Mill Creek TDAs.

Flow control: Floodplain storage is proposed to offset the project's hydrologic impacts within the Mill Creek basin (see Section 4.1). Minor modification to the control structure at one of the existing Stage 2 detention ditches within the project limits is proposed to ensure that it will continue to operate as it was designed. Since the floodplain storage provides all the flow control required for the Mill Creek basin portion of this project, this existing pond is not used to mitigate Stage 4 runoff. However, some new impervious area will be routed through it. Therefore, to route the new additional runoff through the existing pond without controlling it, the control structure is proposed to be adjusted so that this change will not affect its original design. This is further discussed in Section 4.1.1.6.

Table 3-1
Existing and Proposed Water Quality Treatment Facilities for Stage 4 - Developed Condition

Unique Identifier[1]	Existing or Proposed	Starting Station	Ending Station	Facility Type	Stage[2]	TDA	Length (ft)	Depth (if applicable) (ft)	Width (if applicable) (ft)	Contributing Area (ac)
DR14-3	Proposed	ES1 97+17 (19.41 LT)	ES1 104+22 (19.93 LT)	CAVFS	Stage 4	M1	705	1.0	8.0	0.47
DR18-1	Proposed	ES2 6+42 (18.89 RT)	ES2 3+95 (17.43 RT)	CAVFS	Stage 4	M1	247	1.0	6.0	0.16
DR18-2	Proposed	ES2 7+05 (18.47 RT)	ES2 6+42 (18.87 RT)	CAVFS	Stage 4	M1	63	1.0	6.0	0.04
DR11-6	Proposed	LM' 456+43 (71.61 LT)	LM' 464+98 (83.33 LT)	CAVFS	Stage 4	M1	825	1.0	10.0	1.16
WQ-M2-3	Existing	LM' 467+90 (38.42 RT)	LM' 471+90 (37.70 RT)	Media Filter Drain	Stage 3	M1	400			0.04
WQ-M1-3	Existing	LM' 467+97 (82.05 RT)	LM' 479+51 (81.13 RT)	Media Filter Drain	Stage 3	M1	1154			0.95
DR12-1	Proposed	LM' 475+30 (10.29 LT)	LM' 483+49 (8.93 LT)	Media Filter Drain	Stage 4	M1	819	~2.5 (at trench base)		1.39
WQ-M3-3	Existing	LM' 478+29 (39.47 LT)	LM' 483+27 (37.79 LT)	Media Filter Drain	Stage 3	M1	798			Removed due to Widening. Replaced by DR12-1
WQ-M7-3	Existing	LM' 496+00 (35.44 LT)	LM' 507+35 (37.43 LT)	Media Filter Drain	Stage 3	M1	1135			Removed due to widening. Partially replaced by DR14-5.
DR14-5	Proposed	LM' 496+01 (9.44 LT)	LM' 503+69 (.02 LT)	Media Filter Drain	Stage 4	M1	768	~2.5 (at trench base)		0.98
WQ-M11-3	Existing	LM' 504+69 (81.77 RT)	LM' 518+50 (81.82 RT)	Media Filter Drain	Stage 3	M1	1381			1.18
WQ-M10-3	Existing	LM' 506+03 (110.66 LT)	LM' 511+72 (92.15 LT)	Media Filter Drain	Stage 3	M1	569			0.96
DR16-2	Proposed	LM' 511+72 (91.98 LT)	LM' 515+00 (92.16 LT)	CAVFS	Stage 4	M1	328	1.0	12.0	0.55
DR18-4	Proposed	LM' 525+87 (80.28 LT)	LM' 529+94 (80.80 LT)	CAVFS	Stage 4	M1	407	1.0	10.0	0.55
DR16-3	Proposed	NE 10+13 (18.22 RT)	NE 19+45 (20.77 RT)	CAVFS	Stage 4	M1	932	1.0	9.0	0.62
DR16-5	Proposed	NE 19+45 (20.78 RT)	CD 9+13 (39.45 RT)	CAVFS	Stage 4	M1	322	1.0	10.0	0.13
DR14-1	Proposed	NEW 13+94 (22.58 LT)	NEW 16+00 (23.73 LT)	Media Filter Drain	Stage 4	M1	206	~2.5 (at trench base)		0.34
DR14-2	Proposed	NEW 9+46 (17.48 LT)	NEW 13+94 (16.58 LT)	CAVFS	Stage 4	M1	448	1.0	10.0	0.24
DR13-2	Proposed	WS' 89+14 (28.00 LT)	WS' 94+19 (30.49 LT)	CAVFS	Stage 4	M1	505	1.0	6.0	0.33
DR24-1	Proposed	AR2 9+01 (17.85 RT)	AR2 14+00 (20.41 RT)	CAVFS	Stage 4	M2	499	1.0	11.0	0.36
WQ-M29-3	Existing	AR2 9+01 (22.68 RT)	AR2 14+00 (22.39 RT)	Media Filter Drain	Stage 3	M2	499			Replaced by DR24-1.

Unique Identifier[1]	Existing or Proposed	Starting Station	Ending Station	Facility Type	Stage[2]	TDA	Length (ft)	Depth (if applicable) (ft)	Width (if applicable) (ft)	Contributing Area (ac)
DR25-2	Proposed	DL2 13+92 (40.23 LT)	LM' 613+84 (87.13 LT)	Media Filter Drain	Stage 4	M2	989	~2.5 (at trench base)		1.47
DR18-6	Proposed	LM' 529+94 (80.80 LT)	LM' 538+00 (85.61 LT)	CAVFS	Stage 4	M2	806	1.0	10.0	1.12
WQ-M16-3	Existing	LM' 532+00 (80.64 LT)	LM' 538+00 (86.20 LT)	Media Filter Drain	Stage 3	M2	600			Replaced by DR18-6.
DR20-3	Proposed	LM' 546+52 (104.73 LT)	LM' 560+00 (93.61 LT)	CAVFS	Stage 4	M2	1348	1.0	12.0	2.27
WQ-M20-3	Existing	LM' 546+52 (108.62 LT)	LM' 560+00 (97.99 LT)	Media Filter Drain	Stage 3	M2	1348			Replaced by DR20-3
WQ-M21-3	Existing	LM' 546+98 (110.82 RT)	LM' 554+49 (97.78 RT)	Media Filter Drain	Stage 3	M2	751			1.13
WQ-M22-3	Existing	LM' 554+95 (14.10 RT)	LM' 558+51 (14.21 RT)	Media Filter Drain	Stage 3	M2	356			0.53
WQ-M23-3	Existing	LM' 560+00 (14.03 RT)	LM' 576+06 (14.26 RT)	Media Filter Drain	Stage 3	M2	1606			2.21
DR22-2	Proposed	LM' 560+20 (99.72 LT)	LM' 562+36 (98.81 LT)	Media Filter Drain	Stage 4	M2	216	~2.5 (at trench base)		0.36
WQ-M24-3	Existing	LM' 562+81 (98.52 LT)	LM' 565+79 (96.83 LT)	Media Filter Drain	Stage 3	M2	298			0.49
DR23-1	Proposed	LM' 566+01 (93.14 LT)	LM' 566+87 (92.93 LT)	CAVFS	Stage 4	M2	86	1.0	9.0	0.14
WQ-M25-3	Existing	LM' 567+03 (97.86 LT)	LM' 575+00 (96.59 LT)	Media Filter Drain	Stage 3	M2	797			1.39
DR23-2	Proposed	LM' 575+15 (92.54 LT)	LM' 575+72 (92.27 LT)	CAVFS	Stage 4	M2	57	1.0	14.0	0.11
DR23-4	Proposed	LM' 577+50 (92.42 LT)	LM' 578+84 (92.67 LT)	CAVFS	Stage 4	M2	134	1.0	14.0	0.26
WQ-M26-3	Existing	LM' 578+00 (14.04 RT)	LM' 583+04 (14.10 RT)	Media Filter Drain	Stage 3	M2	504			0.60
WQ-M27-3	Existing	LM' 579+00 (96.81 LT)	LM' 583+49 (98.55 LT)	Media Filter Drain	Stage 3	M2	449			0.85
DR25-1	Proposed	LM' 593+27 (80.99 LT)	LM' 597+93 (80.94 LT)	CAVFS	Stage 4	M2	466	1.0	10.0	0.39
WQ-M32-2	Existing	LM' 599+02 (82.74 RT)	LM' 602+38 (82.10 RT)	Media Filter Drain	Stage 2	M2	336			0.43
WQ-M31-2	Existing	LM' 599+27 (83.71 LT)	LM' 605+90 (84.75 LT)	Media Filter Drain	Stage 2	M2	663			1.08
WQ-M34-2	Existing	LM' 616+12 (88.15 LT)	LM' 619+89 (85.49 LT)	Media Filter Drain	Stage 2	M2	377			0.63
DR18-3	Proposed	NW 16+43 (25.03 LT)	NW 26+53 (24.74 LT)	Media Filter Drain	Stage 4	M2	1010	~2.5 (at trench base)		0.60
DR18-5	Proposed	NW 26+51 (18.77 LT)	NW 35+59 (19.45 LT)	CAVFS	Stage 4	M2	908	1.0	6.0	0.49
WQ-M18-3	Existing	NW 26+51 (24.57 LT)	NW 38+09 (24.59 LT)	Media Filter Drain	Stage 3	M2	1158			Replaced by DR18-5 and DR20-5

Unique Identifier[1]	Existing or Proposed	Starting Station	Ending Station	Facility Type	Stage[2]	TDA	Length (ft)	Depth (if applicable) (ft)	Width (if applicable) (ft)	Contributing Area (ac)
DR20-5	Proposed	NW 35+59 (19.45 LT)	LM' 543+55 (116.34 LT)	CAVFS	Stage 4	M2	254	1.0	15.0	0.53
WQ-M34-2	Existing	LM' 620+04 (82.80 LT)	LM' 627+32 (84.21 LT)	Media Filter Drain	Stage 2	M3	728			1.15
WQ-M36-2	Existing	LM' 625+17 (80.12 RT	LM' 630+67 (82.62 RT)	Media Filter Drain	Stage 2	M3	550			0.72
DR27-1	Proposed	LM' 627+59 (80.30 LT)	LM' 637+86 (80.49 LT)	CAVFS	Stage 4	M3	1027	1.0	9.0	1.27
WQ-M36-2	Existing	LM' 631+14 (80.80 RT)	LM' 637+89 (83.83 RT)	Media Filter Drain	Stage 2	M3	675			0.88
WQ-M38-2	Existing	LM' 638+19 (82.85 LT)	LM' 645+26 (82.41 LT)	Media Filter Drain	Stage 2	M3	707			0.91
WQ-M37-2	Existing	LM' 638+22 (83.60 RT)	LM' 648+86 (84.36 RT)	Media Filter Drain	Stage 2	M3	1064			1.39
WQ-M39-2	Existing	LM' 645+60 (83.60 LT)	LM' 661+19 (82.98 LT)	Media Filter Drain	Stage 2	M3	1559			2.03
DR28-1	Proposed	LM' 649+13 (86.81 RT)	LM' 661+89 (86.79 RT)	Media Filter Drain	Stage 4	M3	1276	~2.5 (at trench base)		1.59
DR29-1	Proposed	LM' 664+78 (85.11 LT)	LM' 689+80 (64.01 LT)	Media Filter Drain	Stage 4	M4	2502	~2.5 (at trench base)		3.21
DR31-1	Proposed	LM' 691+54 (59.37 LT)	LM' 697+84 (71.48 LT)	CAVFS	Stage 4	M4	630	1.0	11.0	0.92
DR32-1	Proposed	LM' 697+84 (77.51 LT)	LM' 707+00 (106.66 LT)	Media Filter Drain	Stage 4	M4	916	~2.5 (at trench base)		1.61
DR8-4	Proposed	ERS' 19+95 (27.36 LT)	ERS' 22+88 (28.62 LT)	CAVFS	Stage 4	W1	293	1.0	6.0	0.18
DR2-1	Proposed	LM' 328+52 (146.83 RT)	LM' 332+27 (126.56 RT)	CAVFS	Stage 4	W1	375	1.0	8.0	0.37
DR3-1	Proposed	LM' 343+02 (83.35 LT)	LM' 354+57 (79.74 LT)	CAVFS	Stage 4	W1	1155	1.0	8.0	1.54
DR4-2	Proposed	LM' 354+57 (87.62 LT)	LM' 364+70 (89.74 LT)	CAVFS	Stage 4	W1	1013	1.0	13.0	1.34
DR5-3	Proposed	LM' 360+00 (74.74 RT)	LM' 363+99 (69.29 RT)	CAVFS	Stage 4	W1	399	1.0	8.0	0.43
DR5-4	Proposed	LM' 364+72 (68.26 RT)	LM' 369+93 (61.42 RT)	CAVFS	Stage 4	W1	521	1.0	8.0	0.57
DR5-2	Proposed	LM' 364+96 (90.01 LT)	LM' 366+80 (88.99 LT)	CAVFS	Stage 4	W1	304	1.0	13.0	0.24
DR9-2	Proposed	LM' 430+50 (112.52 LT)	LM' 433+90 (101.68 LT)	CAVFS	Stage 4	W1	340	1.0	10.0	0.33
DR9-1	Proposed	NER' 36+26 (54.44 LT)	NER' 41+24 (52.59 LT)	CAVFS	Stage 4	W1	498	1.0	9.0	0.57
DR7-6	Proposed	W3 13+00 Vicinity		Constructed Stormwater Wetland	Stage 4	W1				2.96
DR7-1	Proposed	LM' 394+92 (135.90 RT)	LM' 397+14 (137.91 RT)	CAVFS	Stage 4	W2	290	1.0	16.0	0.23

Unique Identifier[1]	Existing or Proposed	Starting Station	Ending Station	Facility Type	Stage[2]	TDA	Length (ft)	Depth (if applicable) (ft)	Width (if applicable) (ft)	Contributing Area (ac)
DR7-16	Proposed	LM' 397+34 (138.18 RT)	LM' 402+12 (169.79 RT)	CAVFS	Stage 4	W2	478	1.0	16.0	0.65
DR8-5	Proposed	SER 7+25 (33.28 RT)	SER 9+51 (33.22 RT)	Media Filter Drain	Stage 4	W2	226	~2.5 (at trench base)		0.12
DR8-6	Proposed	SER 9+51 (27.22 RT)	SER 11+61 (29.60 RT)	CAVFS	Stage 4	W2	210	1.0	11.0	0.11

Notes:

1. See locations in the figures in Appendix A (existing facilities) and C (proposed facilities).
2. Stage 2 facilities are existing, Stage 3 facilities are under construction, and Stage 4 proposed.
3. Structure Note numbers (DR) are given for the Unique Identifier for proposed facilities. Existing facilities identifiers start with "WQ".

Runoff treatment: New and existing media filter drains (MFD) and compost-amended vegetated filter strips (CAVFS) are proposed to provide enhanced runoff treatment within the roadside embankment. Table 3-1 summarizes the specific information for each proposed BMP. The BMPs are shown in Appendix C.

Culverts: No culvert work is proposed within the Mill Creek basin.

Ditches: No new or modified ditches are proposed. The capacity of the median ditch, where it is proposed to receive additional runoff due to the added impervious surface, was checked and found to be adequate so no modification to the ditch is proposed. However, there were two locations where the proposed roadway design, which was prepared using a pervious existing conditions base map, does not correspond with the current base map. The ends of the proposed roadway cross section template do not intersect the existing surface. For this reason, it was not possible to check the ditch capacity at these locations. This is discussed further in Section 4. Once the roadway design is updated based on the current base map, the median ditch capacity should be re-evaluated.

3.1.2 White River Basin (TDAs W1 and W2)

Two detention ponds and one constructed stormwater wetland are proposed for White River TDAs. In addition, new storm drain collection systems are proposed in order to convey road runoff to these facilities.

Storm drainage: New enclosed drainage systems are proposed at the following locations to convey water to detention pond Pond-W3-4 and constructed stormwater wetland Wet-W1-4. In most instances, the collection systems will require that a curb system be installed.

- Southbound outside shoulder from station SR 167 395+73 to ERS 14+45 (Pipe-W5-4 and Pipe-W6-4)
- Southbound outside shoulder from station SR 167 404+61 to SR 167 411+34 (Pipe-W4-4)

The storm drains listed above will cross existing roadways at the following location:

- Station ERS 13+50 (ramp)

In addition to the storm drains listed here, median drains will need to be adjusted to extend through the new impervious surface proposed by the project. Where the project widens the road into the median, the existing inlet will be raised or a new inlet will be added to drain the remaining portion of the median and convey flow to the existing drain under the roadway. Where the road is widened to the outside, the median drain will be extended as needed.

Also, short lengths of pipe are proposed to carry runoff out of the proposed detention ponds and constructed stormwater wetland. The locations of the outfalls are shown in Appendix C. Pond outflow will be discharged into a level spreader where flow will discharge into an existing ditch or wetland.

The storm drains carrying flows away from the proposed detention pond will discharge into the natural receiving water body within the state right-of-way and outside of the ordinary high water; therefore, these drains are not considered new outfalls as defined in the 2008 *Highway Runoff Manual* (HRM).

Median drains will need to be adjusted due to impacts associated with roadway fill placement.

Outfalls: No new outfalls are proposed for the Stage 4 project.

Flow control: Two new detention ponds are proposed for the Stage 4 project. The locations of the new ponds are shown in Appendix C and Appendix M.

Runoff treatment: New media filter drains and CAVFSs are proposed to provide enhanced runoff treatment within the roadside embankment. A constructed stormwater wetland cell is proposed to be combined with detention in Pond W3-4. Table 3-1 summarizes the specific information for each proposed BMP.

Culverts: See Section 4.6 for discussion of proposed culvert modifications. Modifications are proposed for SI 73 and SI 65 (Jovita Creek) in order to provide fish passage.

Ditches: The capacity of the median ditch where the ditch is proposed to receive additional runoff due to the added impervious surface was checked. The median ditch was found to be adequate and no modifications are proposed. However, there were two locations where the proposed roadway design, which was prepared using a pervious existing conditions base map, does not correspond with the current base map. The ends of the proposed roadway cross section template do not intersection the existing surface. For this reason, it was not possible to check the ditch capacity at these locations. This is discussed further in Section 4. Once the roadway design is updated based on the current base map, the median ditch capacity should be re-evaluated.

In addition, where the roadway is to be widened to the outside in the vicinity of Ellingson, the existing drainage ditches are proposed to be modified.

3.2 Design Standards

The preliminary stormwater flow control and runoff treatment BMPs were sized primarily using guidelines from the 2008 HRM. Appendix H contains the WSDOT Stormwater Design Documentation spreadsheet that was used to determine the minimum requirements for this project.

The stormwater conveyance systems are described in the following paragraphs and shown in Appendix C.

3.2.1 Flow Control

3.2.1.1 Mill Creek

An alternative, "non-conventional" stormwater management approach, floodplain storage, is proposed along Mill Creek to provide flow control for the portion of the project in the Green River basin. The HRM allows an alternative flow control standard if watershed-scale hydrologic modeling and field observations show the alternative approach would be effective. The analysis that supports the use of this alternative approach is described in more detail in Section 4.

As discussed in Section 2, the Green River valley has extremely flat terrain, backwater flow conditions, high groundwater, and adjacent wetlands. In the Mill Creek basin nearly all of the roadway is surrounded by wide, valley-bottom wetlands and frequently flooded areas. These challenges make it difficult to locate suitable sites for flow control and water quality treatment facilities. As a result, a non-conventional stormwater management approach was taken involving floodplain storage.

The intent of the floodplain storage approach is to create additional storage adjacent to the existing creek to offset potential impacts caused by increased runoff from the proposed project. The main difference between floodplain storage and conventional detention is that floodplain storage provides storage during flood stages without any constructed controls to hold water and release at a prescribed rate. Floodplain storage also has the opportunity to provide additional habitat benefits that conventional detention does not.

The floodplain storage approach is also consistent with the recommendations of the *Mill Creek Basin Flood Management Plan* (NHC 1999), which was prepared for King County, the City of Auburn, and the City of Kent.

In order for the Washington Station Department of Ecology (Ecology) to approve an alternative stormwater management approach, WSDOT is required to demonstrate that the approach will not increase the stream channel erosion rates beyond those characteristic of natural or re-established conditions. Supporting data must be provided to show this approach is protective of water quality and satisfies state and federal water quality laws. This supporting data is described in Section 4.1.1.

3.2.1.2 White River

The flow control detention pond facilities in the White River basin were sized using MGS Flood, a continuous simulation model, so that the pond outflow matches the duration of the pre-developed peak flows from 50 percent of the 2-year storm flow up to the 50-year storm flow assuming forested pre-project conditions. The model results are included in Appendix E. Further discussion about the modeling is provided in Section 4.

The "equivalent area option" (2008 HRM, Section 4-3.6.1) was used to ensure runoff from areas equivalent to the total new impervious surface will be detained by the proposed facilities. The equivalent area option allows stormwater detention to be applied to an equivalent area when that area is more feasible than providing the detention for the new impervious area because of site constraints. This is particularly applicable to road widening projects where it is difficult to isolate runoff from the new "widened" portion of the road.

3.2.2 Runoff Treatment Design Criteria

According to the 2008 HRM, enhanced runoff treatment is required for the Stage 4 project due to the high average daily traffic (ADT) count (> 30,000 vehicles) and because the runoff from the projects discharges to surface waters. Treatment is only required for the new impervious surface; however, an attempt was made to retrofit as much of the existing impervious surface as possible. Enhanced treatment options were assessed for all new, replaced, and existing pavement areas within the project limits. Runoff treatment was provided for all pavement areas except when the following constraints existed:

<u>For areas with storm drains discharging to detention ponds:</u>

- No runoff treatment was provided for highway areas draining to proposed detention pond sites that could not accommodate a centralized enhanced treatment BMP (such as a constructed stormwater wetland) within the state right-of-way. Additionally, at this stage of design, centralized water quality BMPs are not proposed if they would impact natural wetlands or require excavation into groundwater.

<u>For areas without a storm drain system:</u>

- No runoff treatment was provided for highway areas draining to an existing or proposed embankment slope steeper than 4:1 (H:V).
- No runoff treatment was provided if the embankment width or conditions (delineated wetland, high groundwater, etc.) do not allow for placement of an enhanced treatment BMP. A centralized enhanced treatment BMP, such as a constructed stormwater wetland, would require a liner, which is difficult to construct below groundwater. Therefore, situations that would require excavation into groundwater were avoided.

Within each TDA, enhanced runoff treatment is proposed for an area of highway that exceeds the area of new pavement surface in that TDA. (Within the White River and Mill Creek basins, highway areas equivalent to 296 percent and 265 percent, respectively, of new impervious area are proposed for enhanced runoff treatment.) In locations where the actual new and replaced pavement is not proposed to receive runoff treatment, the requirements of the "equivalent area option" (2008 HRM, section 4-3.6.1) will be met. The equivalent area option allows runoff treatment to be applied to an equivalent impervious area where that is more feasible than providing treatment to the actual new pavement.

Based on field reviews, general basin characteristics, and the existing geotechnical information for the project, centralized dispersion areas and infiltration ponds were not considered feasible runoff treatment options for the Stage 4 project. The groundwater elevations and soil types were not conducive to relying upon these runoff treatment methods.

Within this context, runoff treatment BMPs were selected in the following order of preference:

- **Compost-amended vegetated filter strip (CAVFS):** CAVFS is the preferred method of runoff treatment where highway runoff leaves the roadway as sheet flow. This BMP relies on amending the existing soils with aged compost to provide: (a) chemical processes for the removal of dissolved heavy metals and (b) filtering for solids retention as runoff percolates through the amended soil layer. The 2008 HRM allows the designer to model infiltration losses into the embankment soils underlying CAVFS. The key design criteria for this runoff treatment BMP are:
  - The embankment slopes are 4:1 or flatter.
  - CAVFS widths are dependent upon infiltration rates and pavement width draining to the CAVFS. On this project, widths of CAVFS vary between 6 and 18 feet.
  - Amended soils lie enough above groundwater level to allow infiltration.
  - Ideally, the CAVFS is protected from damage by errant vehicles and infestation by invasive plant species.
- **Media filter drain:** The media filter drain is used in a similar manner as the CAVFS. However, media filter drains require a special media mix (“Ecology Mix”) to provide water quality treatment and an underdrain system under most soil conditions. At this time, credit for infiltration losses is not addressed in the 2008 HRM. The key design criteria are:
  - The embankment should preferably be on a 4:1 or flatter slope. A slope of up to 3:1 can be used but requires special slope reinforcement and embankment protection measures.
  - Embankments are about 10 feet wide (includes gravel pavement edge and grassy filter strip upslope of Ecology Mix).
  - Ecology Mix and underdrains lie above the saturation zone.
  - Ideally, media filter drains are protected from damage by errant vehicles and infestation by invasive plant species.
- **Constructed stormwater treatment wetlands (CSW):** CSWs are shallow, constructed wetlands that are designed to treat stormwater through settling, filtering, and biological processes associated with emergent and floating aquatic plants. The key design criteria of CSWs are:
  - The wetland must have a minimum of two cells that retain the volume associated with the runoff treatment design storm.
  - Soil amendments and plants are to be installed and maintained to mimic a natural wetland system.

- The groundwater information gathered at the site indicates that the seasonal highwater is below the proposed bottom of the CSW and therefore, a liner is not necessary. However, the need for a liner should be confirmed by WSDOT's geotechnical engineer.
- For CSWs that also provide detention, the difference between the runoff treatment design water surface and the two-year storm event water surface must not exceed 3 feet.

3.2.3 Conveyance Design Criteria

The conveyance systems were designed to contain the runoff from the storm event with the mean recurrence interval as defined in Figure 1-4 of the *Hydraulics Manual*.

3.2.4 Precipitation Values

The conventional detention ponds were sized using an extended precipitation time series developed by MGS Engineering Consultants, Inc. for the MGS Flood (Version 3.12) computer model.

The precipitation used in the HSPF model of the Mill Creek drainage basin to evaluate the floodplain storage consisted of adjusted 15-minute data from the Sea-Tac gage from water years 1949 through 1989 and 1998 through 2005 combined with 15-minute data from the Star Lake rain gage for water years 1990 through 1997. The Sea-Tac gage is located at Seattle-Tacoma International Airport and is maintained by the National Weather Service (Gage #7473). The Star Lake gage is located south of Star Lake, near Federal Way, and is maintained by King County (Gage #41U). The Sea-Tac precipitation had been increased by 15 percent based on the relationship from 1989 through 1991 of Sea-Tac cumulative rainfall to Star Lake cumulative rainfall. (NHC 2001.)

3.2.5 Other Requirements

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3.2.6 Level of Retrofit

The level of retrofit for the combined Stage 4 project is presented in Table 3-2.

Table 3-2
Level of Stormwater Retrofit

Threshold Discharge Area (TDA)	Existing Impervious within Project TDA (ac)	New Impervious within Project TDA (ac)	Impervious Area Proposed for Runoff Treatment (ac)	Percent of New Impervious Proposed for Runoff Treatment	Impervious Area to Flow Control Facilities[2] (ac)	Percent of New Area Proposed for Flow Control
M1	37.87	3.89	5.63	144.7%	N/A (Demonstrative approach utilizing a floodplain storage strategy)	
M2	32.45	3.56	5.97	167.6%		
M3	11.95	0.26	3.06	1178.2%		
M4	22.38	0.00	5.74	N/A[1]		
Mill Creek TDA Totals	104.65	7.71	20.40	264.6%		
W1	33.33	3.26	8.53	261.8%	3.26	100.0%
W2	6.09	0.00	1.11	N/A[1]	0	N/A
White River TDA Totals	39.42	3.26	9.64	295.8%	3.26	100.0%

[1]There is no new impervious surface in this TDA. Division by zero would result in infinity.

[2] Roadway impervious area. For all the impervious and pervious area draining to the detention facilities, refer to Table 4-9.

3.2.6.1 Runoff Treatment

All lanes and ramps within the project limits were assessed for suitability to receive enhanced water quality treatment. This included all new, replaced, and existing impervious areas.

Centralized runoff treatment systems, such as constructed stormwater wetlands, were not proposed within the Mill Creek basin due to a lack of sites that would function properly and would not adversely impact wetlands. Constructed stormwater wetlands were proposed within the White River basin in association with the detention pond sites. These facilities are only proposed where they will not adversely impact existing wetlands, require excavation into groundwater, or require acquisition of additional property.

For areas without a storm drain system, runoff treatment is proposed via CAVFS or media filter drains only in those areas that meet all of the following criteria:

- Roadway embankment side slopes are 4:1 (H:V) or flatter,
- Facility footprint does not impact a wetland, and
- Facility bottom is above the observed groundwater elevation.

3.2.6.2 Flow Control

The proposed flow control strategy for the project areas within the Mill Creek basin is to use floodplain storage, for which a level of retrofit was not determined (see Section 4.1 for a detailed discussion of this approach). This is a demonstrative method of stormwater management that documents that the project's impacts will not have an adverse impact on the water quality in the Mill Creek system.

Within the White River basin, detention ponds are proposed to offset the runoff from 100 percent of the new impervious surface (see Table 3-3).

3.3 Pipe Alternatives

Acceptable pipe alternatives for this project include:

- Corrugated Polyethylene Pipe AASHTO M 294 Type S
- Ductile Iron
- Concrete Pipe
- PVC

However, corrugated metal pipe is not an acceptable alternative because its roughness is about twice that of the above listed pipe materials and therefore would not provide the same hydraulic capacity.

All the pipes listed are acceptable in the project area which is within Corrosion Zone II.

3.4 Downstream Analysis

The purpose of the downstream analysis is to assess the conveyance systems at the downstream ends of the project and at intermediate locations where flows exit WSDOT right-of-way. The resources consulted as part of the analysis included topographic maps; aerial photographs; discussions with WSDOT maintenance staff; discussions with local agencies including cities of Auburn, Sumner, and Pacific and Pierce County Drainage District 24; and drainage plans and studies from local jurisdictions. In addition, field reconnaissance was conducted to verify information and a hydrologic analysis was performed to assess stream flows.

Both Mill Creek and the Unnamed Tributary to the White River (UTWR) were inspected for a quarter mile downstream of the project limits and where the creek left the right-of-way. No significant erosion problems were identified for either creek.

3.4.1 Review of Resources

WSDOT maintenance staff, city staff from Auburn, Pacific and Sumner, and a commissioner from Drainage District 24 were consulted concerning conveyance and erosion issues in the channels along SR 167 in the project area. The Drainage District is responsible for maintaining the UTWR through Pacific and south through Sumner.

WSDOT is responsible for maintaining Mill Creek and the remainder of the UTWR when they flow within WSDOT right-of-way.

The Drainage District reported no known erosion or conveyance issues along the UTWR other than ongoing conveyance impediments resulting from beaver dams. The Drainage District removes (or hires WSDOT to remove) the dams when they become a problem. But after time, the beavers often rebuild the dams. Therefore, this is an ongoing maintenance issue that periodically needs attention.

Discussions with WSDOT maintenance personnel confirmed conveyance problems along Mill Creek due to flow backing up at the confluence with the Green River when the Green River is at flood stage, high local runoff from basin tributaries, a seasonally high water table, vegetation-choked drainage ditches, and inadequately sized culverts. WSDOT personnel indicated that the Mill Creek water levels have risen to within a vertical foot of the roadway pavement in the area between SR 18 and 15th Street NW. Beaver dams have also been an ongoing impediment to conveyance in Mill Creek throughout the project area. Similar to the UTWR, WSDOT removes these dams when they become problems, but after time the beavers return and build new dams.

3.4.2 Inspection of Drainage Systems

A field reconnaissance was performed for a quarter mile downstream of the north and south ends of the project. Field inspections were made on October 27, 2006, November 9, 2006, and February 2, 2007. In addition, field reconnaissance was also performed at a few intermediate locations where the creeks leave the WSDOT right-of-way. Field observations are discussed below. Photos are presented in Appendix D. Refer to Figures 3-1 and 3-2 and Appendix A for the referenced enclosed conveyance locations.

As discussed in Section 2.5, there are several types of enclosed conveyance systems within the project limits. The existing enclosed conveyances were classified in terms of function and given a structure identification (Table 2-3). In the discussion of the downstream systems, these are referred to by their structure identification (SI).

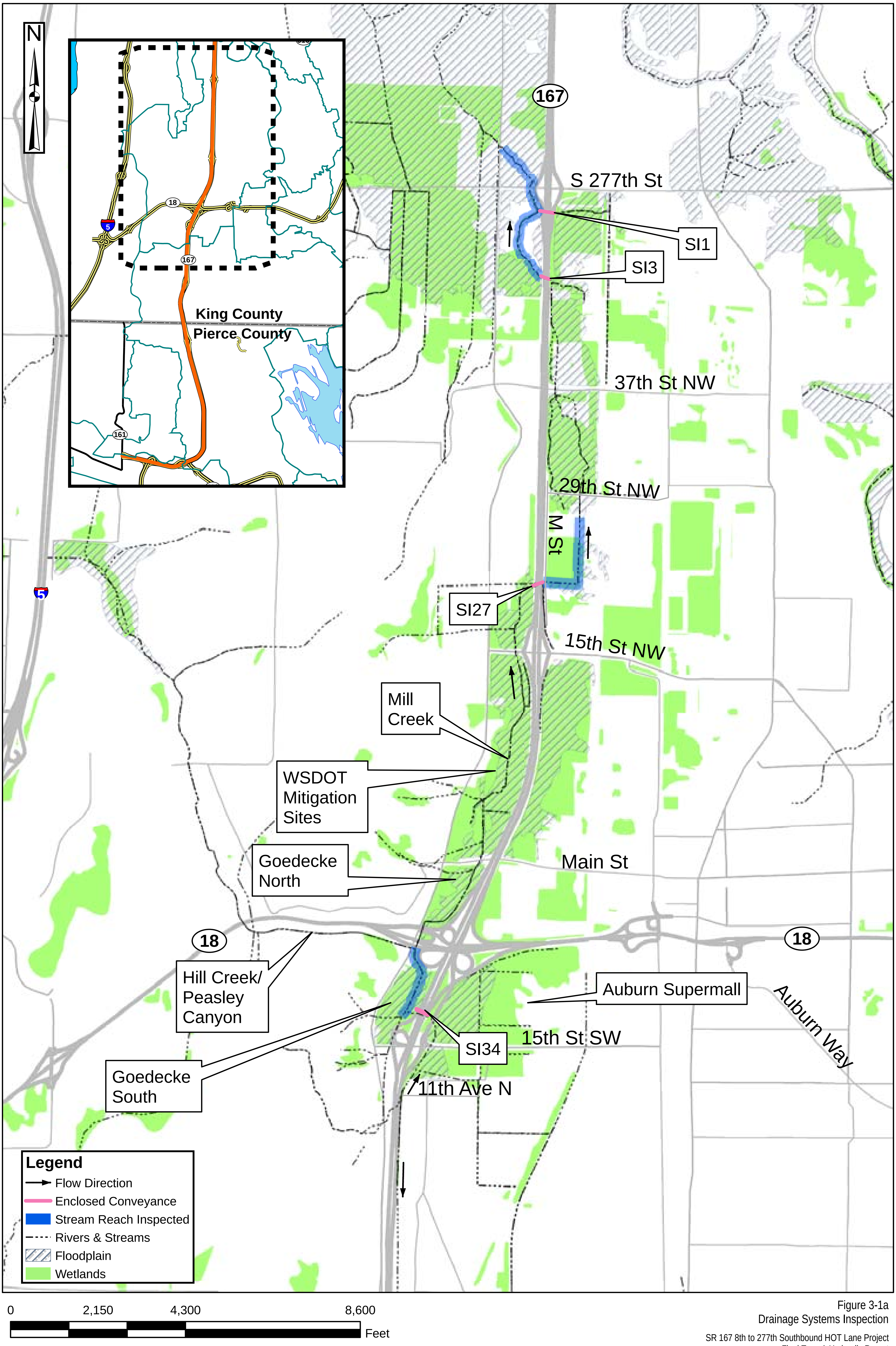

Figure 3-1a
Drainage Systems Inspection

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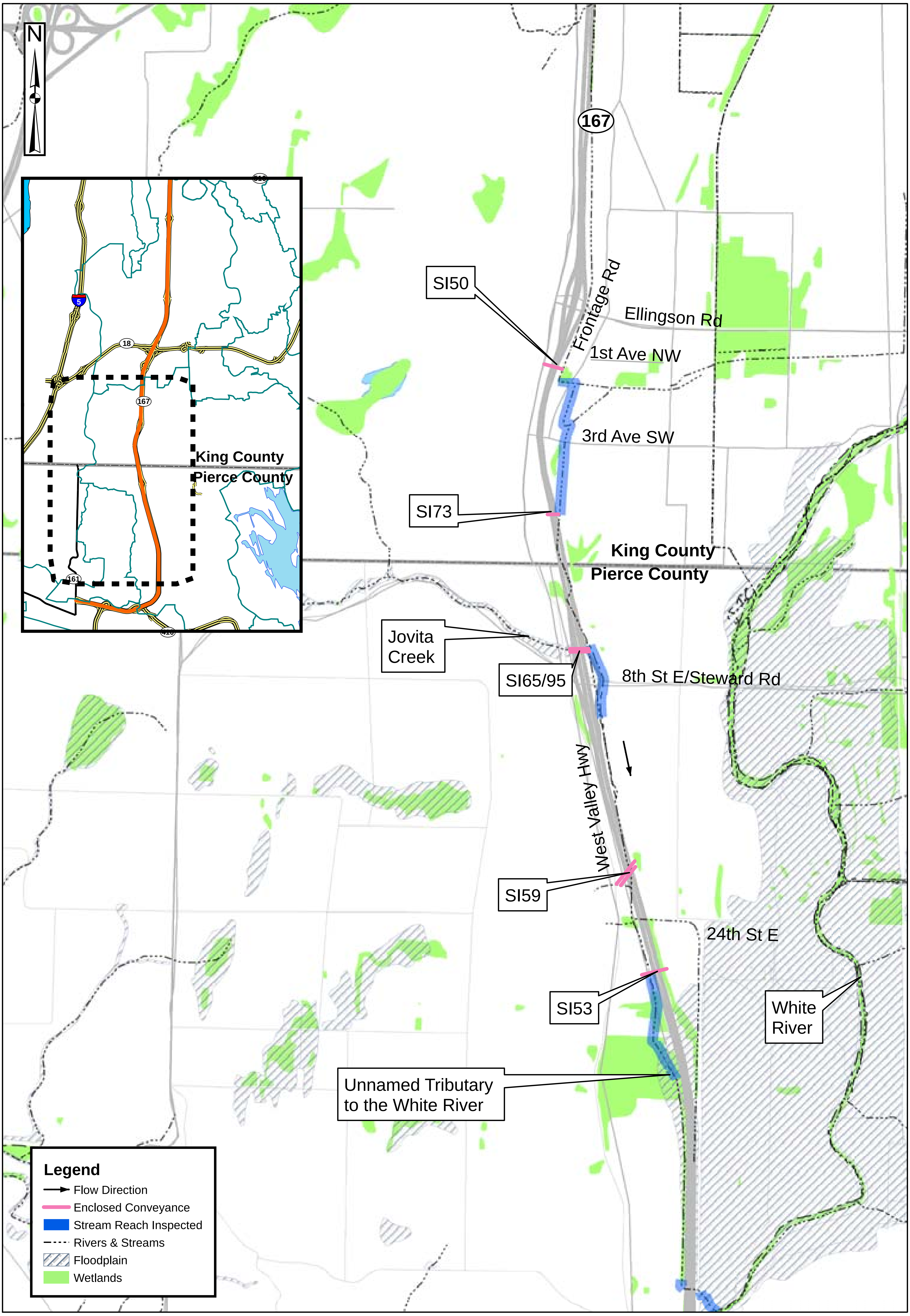

0 2,250 4,500 9,000
Feet

Figure 3-1b
Drainage Systems Inspection

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3.4.2.1 Mill Creek – SI 3 to One-Quarter Mile Downstream of North Project Limits – February 2, 2007

SI 3 conveys Mill Creek under SR 167 from east to west. The culvert is a concrete box culvert consisting of twin 6-foot-wide by about 8-foot-high chambers separated by 1.5-foot-thick concrete wall. The culvert was completely submerged during the field visit but appeared to be in good condition. The water surface elevation on the downstream end was a half inch above the culvert soffit. The creek was flowing bank full on the downstream end of the culvert. The upstream end of SI 3 was completely submerged and the water surface elevation was about 6 inches above the culvert soffit. A large pool was observed on the upstream end; however, the water level had not reached the top of bank. Downstream from SI 3 the creek flows through agricultural land at an estimated velocity of less than 1 foot-per-second (fps), the water was clear, and the bank vegetation was predominately reed canary grass.

SI 1 picks up drainage from the east side roadside ditch and frontage road along SR 167 and conveys the water to Mill Creek on the west side of SR 167. SI 1 is a 12-inch-diameter corrugated metal pipe (CMP) with a crushed downstream end. The upstream end of SI 1 was not found and was most likely submerged by water at the time of observation. Mill Creek downstream of SI 1 is 15 to 20 feet wide. A small channel about 40 feet in length carries the water from the SI 1 outlet to Mill Creek. The small channel is 3 to 5 feet in depth with side slopes at about 1H:1V and is choked by reed canary grass.

Mill Creek flows under S 277th Street through four circular CMP culverts. The two outside culverts have a 9-foot diameter and the two inside culverts have a 6.6-foot diameter. The inlets and outlets to the culverts at the base of the roadway fill are mitered to match the fill slope. The culverts appeared in good condition. The flow from the culvert discharges into a 35-foot-wide area of ponded water. The water in the center of the pool was estimated to be 5 feet in depth.

The channel resumes about 50 feet downstream from the crossing beginning at 20 feet wide and becoming narrower as the channel continues downstream to width near 10 feet.

Mill Creek flows under West Valley Highway just northwest of S 277th Street. An approximately 20-foot-wide bridge with columns spans the creek allowing it to flow under the highway. The channel just upstream and downstream from the crossing flows through agricultural properties that provide no riparian cover. On the downstream side of the crossing the channel has a 2- to 3-foot-high constructed berm along the north bank. The channel bottom width in this area is 5 to 8 feet wide and is choked with reed canary grass. The channel is 1 to 2 feet deep with a 3- to 5-foot bank height. The water was clear in this area and the average velocity was estimated at about 0.8 fps.

3.4.2.2 Mill Creek – Downstream of Culvert SI 27 – November 9, 2006

After Mill Creek exits SI 27, the flow splits, with some of the flow heading north in a roadside ditch and most of the flow heading east out of the WSDOT right-of-way and under M Street NW via a 9-foot-wide by 5-foot-high box culvert. From the M Street NW culvert crossing, the creek continues east for about 940 feet and then turns 90 degrees to head north for about 1300 feet until it reaches property owned by the Muckleshoot Tribe. The channel has a 2- to 3-foot bottom width with side slopes at 3H:1V or 4H:1V. The east-flowing portion of the channel has a berm on the south side that provides approximately 3 vertical feet of depth before flow would overtop and spill into the lower property to the south. There was no evidence of erosion through this portion of the channel.

During the field visit November 9, 2006, which was just after a significant rain event, creek water levels at the M Street NW culvert were observed up to its soffit and flow in the downstream channel was actively spilling over the berm. Further downstream at 29th Street NW, the creek overtopped the road in two places on either side of the creek. The roadway overtopping was the result of backwater downstream of the creek that created a tailwater elevation equal to the top of the road. Because the overtopping is a result of backwater, it is unlikely that a larger culvert at this location would alleviate this problem.

3.4.2.3 Mill Creek – Downstream of Culvert SI 34 – October 27, 2006

According to the *Concept Habitat Mitigation Plan, The SLQ Industrial Site, Auburn Washington* (Raedeke Associates, Inc. 1997), after the creek exits Culvert C9, it turns north and flows along the toe of the SR 167 embankment and then curves back west near the SR 18 on-ramp before it enters a culvert under SR 18. However, during the field visit, much of the channel in this area was undefined beginning about 50 feet after it exits SI 34. Between there and about 400 feet upstream of the culvert under SR 18, it appeared that the flow spreads out through a large marshy area. In the 400-foot-long section where the channel is defined, it is about 8 to 10 feet wide and about 3 feet deep, although there are about 15 vertical feet between the channel bottom and the top of West Valley Highway.

Habitat mitigation sites have been developed between 15th Street SW and SR 18. This area is referred to as Goedecke South. The site to the south was constructed in 2001 while the site to the north is currently under construction.

Flow from Peasley Canyon joins Mill Creek about 60 feet upstream of the SR 18 crossing. There was a bar of cobbles deposited on the downstream side of Peasley Canyon culvert under West Valley Highway.

3.4.2.4 Unnamed Tributary to the White River – Near 3rd Avenue SW – February 2, 2007

The UTWR leaves the WSDOT right-of-way near 1st Avenue NW downstream of SI 50. After the creek exits the right-of-way, it flows to east for about 230 feet through a channel with a 5- to 8-foot bottom width and about 6 feet to the top of bank. The creek then crosses under Frontage Road S through a 48-inch-diameter CMP. The flow continues east in a channel to the Interurban Trail and then turns southwest and continues along the west side of the Interurban Trail for about 1700 feet. The straight channel along the west side of the Interurban Trail is grass-lined and there are no signs of erosion. The creek flows under the Interurban Trail via a 60-inch-diameter CMP culvert and joins a larger tributary just before flowing under 3rd Avenue SW. The exposed crown of the culvert has some minor damage on both sides of the trail but it does not appear to affect the culvert's capacity. The larger tributary conveys local drainage from the cities of Algona and Pacific. The larger tributary has a bottom width of 5 to 6 feet, bank heights of 3 to 4 feet, and velocities comparable to the UTWR at the time of inspection.

The UTWR flows underneath 3rd Avenue SW through a 6-foot-diameter concrete culvert. The channel is fairly uniform from the 3rd Avenue SW crossing until it reaches SI 73. The channel has an approximately 5-foot-wide bottom width and the channel material appears to consist mostly of silt. The water was approximately 1 to 2 feet deep. The banks are 3 to 5 feet high with 1H:1V side slopes. The vegetation is mostly grass but trees are present on the east side of the channel for a portion of the reach. Approximately 500 feet downstream from 3rd Avenue SW, the channel flows under a wooden footbridge. At about 1500 feet downstream from 3rd Avenue the UTWR flows under a small utility crossing via three 36-inch-diameter CMP culverts. The culverts are approximately 15 feet long. Flow is completely blocked in the left and middle culverts due to sediment and debris. The average flow velocity downstream of 3rd Avenue SW was estimated to be 0.4 fps.

3.4.2.5 Unnamed Tributary to the White River – Downstream of 24th Street E Southbound On/Off Ramps – February 2, 2007

The channel downstream of 24th Street E Southbound SR 167 on-ramp has a 5-foot-bottom width and is about 2 feet deep with 3- to 5-foot bank heights. The channel side slopes are about 10H:1V. The channel is choked with reed canary grass. The average velocity in the thalweg of the channel was estimated to be about 1.6 fps. Approximately 2500 feet downstream of 24th Street E, the UTWR flows around the west side of a wetland mitigation area. The velocity of the UTWR flowing around the wetland area decreases to about 0.4 fps. Between 24th Street E and the last southerly SR 167 crossing, the channels flows within an 80-foot-wide corridor between SR 167 and private properties to the west which sit on high fill and are not liable to flood.

3.4.2.6 Unnamed Tributary to the White River – Crossing at 8th Street E/ Stewart Road SE – February 2, 2007

Just downstream of SI 65/95, the UTWR flows under 8th Street E through twin 10-foot-wide by 9-foot-high concrete box culverts. It was unclear if the 8th Street E culvert is a 3-sided or 4-sided box culvert. Before the channel enters the culvert under 8th Street E it is forced to make an abrupt 45 degree turn toward the west in order to enter the culvert. The bottom width of the channel at this crossing is 5 to 10 feet wide and it has a silty, muddy, organic bottom material. The water was 1 to 2 feet deep. The bank side slopes are approximately 3H:1V. The channel vegetation consisted of invasive grass and brush with some shade trees along the banks. The water appeared clear and the average velocity was estimated to be 0.8 fps. No signs of erosion were present upstream or downstream of this crossing.

3.4.3 Analysis of Off-Site Effects

This section discusses the analyses of off-site flows where it was necessary to assess impacts to proposed project modifications. Figure 3-2 shows the off-site subbasins draining to project culverts with proposed modifications, SI 65/95 (Jovita Creek) and SI 73. Hydrologic information for the tributary subbasins is summarized in Table 3-3. The May and January high fish passage flows were determined based on regression equations provided in the *Design of Road Culverts for Fish Passage* (WDFW 2003). The 25- and 100-year peak flow hydrology was determined based on the USGS Regression equations as recommended in the Hydraulics Manual. Low flow for fish passage analysis was assumed to be 0 cfs as recommended by WDFW (2003).

Table 3-3
Culvert Design Flows

Culvert ID	Stream System	Contributing Area (ac)	Culvert Size (openings)	Jan. High Fish Passage Flow (cfs)	May High Fish Passage Flow (cfs)	Fish Passage Low Flow (cfs)	Peak Existing Conditions Flow [1] (cfs)		Field Observations
							25-Year	100-Year	
SI 73	Unnamed Tributary to White River	399	4'x5' Concrete Box	5.6	1.2	0	53.1	70.3	Culvert condition appeared adequate. Filled slightly with sediment
SI 65/95	Jovita Creek	1964	Twin 84" Diameter CMPs	24.7	7	0	210.4	277.1	North culvert invert is 18" higher than south culvert invert. North culvert likely an overflow culvert and is partially blocked by a tree at the upstream end and it appears that a plate has been added at the downstream end that partially blocks the exit. Culverts also have a weir in front of the upstream entrance. The weir has a 4" high notch with an 18" wide crest that is 2' thick.

1. Based on USGS Regression Equations plus one standard deviation

It is noted that in *Magnitude and Frequency of Floods in Washington* (Sumioka, Kresch, and Kasnick 1998), the regression equations were developed using data from unurbanized basins. Both tributaries' basins include some urbanization, so there was some concern that the peak flow estimates may not be conservative. As a result, the peak flows were increased by one standard deviation as a factor of safety. The resulting peak flows were then compared to recorded flows in other Puget Sound area creeks as a validation.

The first validation was to compare the resulting 100-year peak flow estimated for Jovita Creek (277.1 cfs) to recorded flow at Peasley Canyon. Both creeks are in the same general vicinity and have similar land use, terrain, and geology. In addition, both basins include significant lakes: Lake Dolloff in the Peasley Canyon basin and Trout Lake in the Jovita Creek basin. Unfortunately, the period of record for the Peasley Canyon gage (October 1989 to February 1992 and then April 2004 to the present day) is too short for meaningful statistical analysis. However, the January 1990 event was recorded and it is considered a significant event. This event was used as a validation check and the results are presented in Table 3-4. A flow per unit area value was determined for Peasley Canyon for the 1990 event and applied to the Jovita Creek basin area. The resulting January 1990 flow for Jovita Creek (281 cfs) using this method is quite close it the 100-year flow estimate (277.1 cfs) using the USGS equations plus one standard deviation.

Table 3-4
Jovita Creek Flow Validation
(Based on Peasley Canyon Recorded Peak Flow for January 1990)

Recorded Peasley Peak Flow [1]	348	cfs
Peasley Contributing Area	2434	acres
Flow per acre	0.14	cfs/acre
Jovita Creek Contributing Area	1964	acres
Estimated Jovita Creek Flow [2]	281	cfs

1. Based on recorded 15 minute data
2. Based on applying 0.14 cfs/acre to 1964 acres

An additional validation was performed using additional creek data in the Puget Sound area that do not have the same local proximity as Peasley Canyon, but reflected urban land uses and have flow data of a sufficient length for statistical analysis. This validation is summarized in Table 3-5. Again, a flow per unit area was developed based on the recorded data and applied to the Jovita Creek basin. The results show that the 100-year flow based on the maximum flow per unit area (271 cfs) corresponds well with the value developed using the USGS equations plus one standard deviation (277.1 cfs).

As a result of the validation efforts, the USGS equations for the 25- and 100-year events plus one standard deviation were used to determine the peak flow events for Jovita Creek and SI 73.

Table 3-5
Jovita Creek Flow Validation
Based on USGS Data for Puget Sound Area Creeks

SOURCE	BASIN	LAND USE	AREA (ACRE)	PERIOD	--------- PREDICTED FLOWS (cfs) ----------				----- UNIT-AREA DISCHARGE (cfs/ac) -----			
					Q2	Q10	Q25	Q100	Q2	Q10	Q25	Q100
USGS	JUANITA	URBAN	4,282	64 TO 90	216	310	364	454	0.050	0.072	0.085	0.106
USGS	JUANITA	URBAN	4,282	77 TO 90	144	305	408	593	0.034	0.071	0.095	0.138
USGS	MAY CREEK	MIXED	8,000	46 TO 79	214	385	477	620	0.027	0.048	0.060	0.078
USGS	THORTON CREEK	URBAN	7,744									
USGS	MCALEER CREEK	URBAN	4,992	64 TO 74	141	215	251	304	0.028	0.043	0.050	0.061
USGS	LYONS CREEK	URBAN	2,349	64 TO 75	109	147	164	188	0.046	0.063	0.070	0.080
USGS	LEACH CREEK	URBAN	3,027	58 TO 79	48	88	123	203	0.016	0.029	0.041	0.067
	FLETT CREEK	URBAN	4,691	60 TO 79	48	87	110	149	0.010	0.019	0.023	0.032
USGS	COAL CREEK	URBAN	4,352	64 TO 79	157.7	258	332	473	0.036	0.059	0.076	0.109
USGS	VALLEY CREEK	URBAN	1,952	49 TO 77	44	87	127	220	0.023	0.045	0.065	0.113
USGS	MERCER CREEK	URBAN	7,680	56 TO 90	245	361	417	498	0.032	0.047	0.054	0.065
USGS	MERCER CREEK	URBAN	7,680	76 TO 90	294	404	436	469	0.038	0.053	0.057	0.061
								AVG	0.031	0.050	0.062	0.083
								MAX	0.050	0.072	0.095	0.138
								MIN	0.010	0.019	0.023	0.032

Return Period	Jovita Creek Contributing Area (acre)	Discharge per Unit Area (cfs/acre)[1]		Resulting Jovita Creek Flow for Comparison (cfs)	
		Max	Min	Max	Min
25-Year	1964	0.095	0.023	187	45
100-Year	1964	0.138	0.032	271	63

[1]Based on the above analysis

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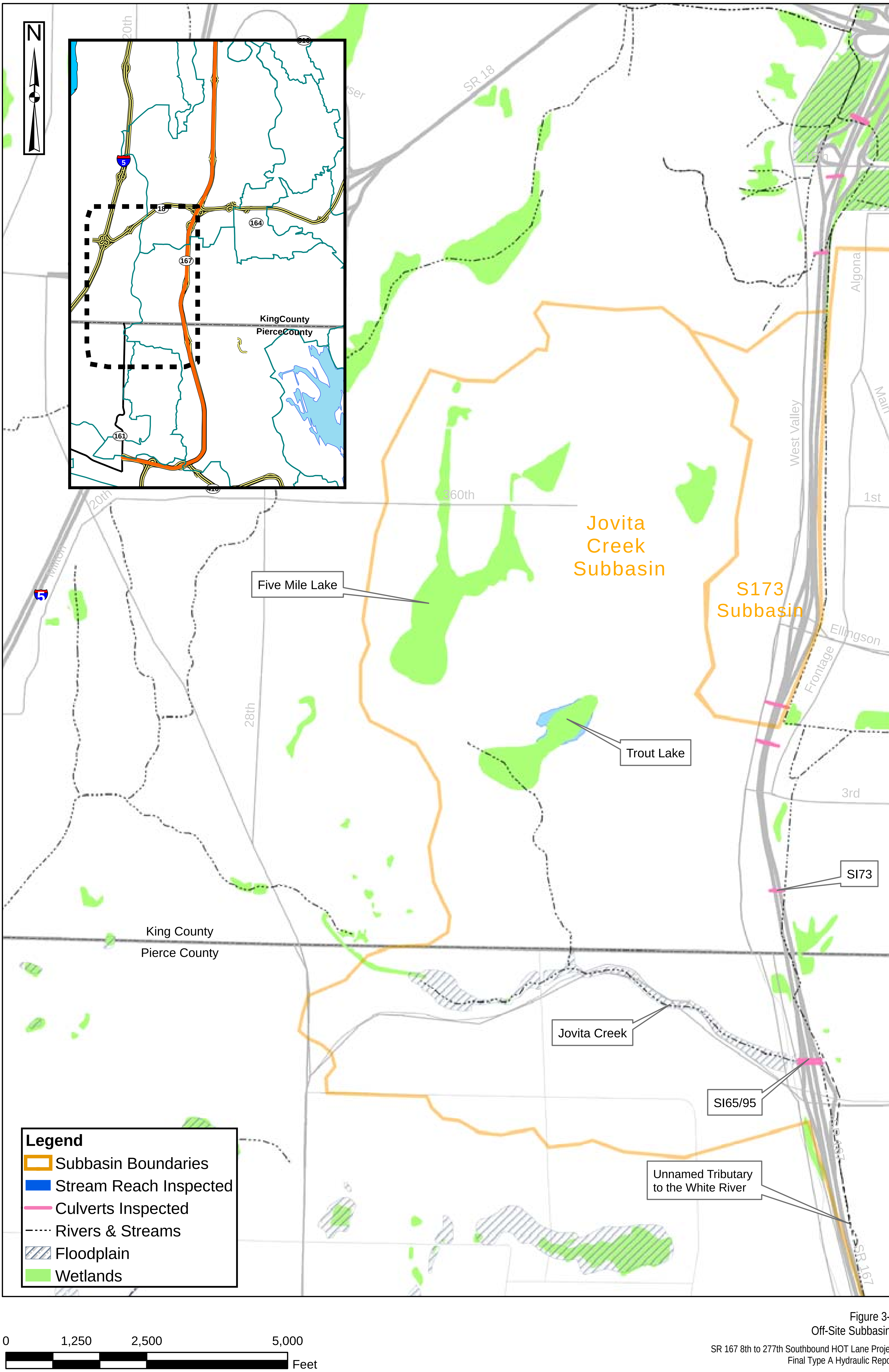

Figure 3-2
Off-Site Subbasins

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Section 4
HYDROLOGIC AND HYDRAULIC DESIGN

This section summarizes the design calculations for the flows and hydraulic features in the proposed project. Locations of the proposed hydraulic features are shown in Appendix C. Calculations, including model input and output, and for the various types of hydraulic features, are included in Appendix E.

To size detention ponds, determine flows for the existing culverts, and evaluate the compost-amended vegetated filter strips (CAVFS), hydrologic and hydraulic modeling was conducted using the MGS Flood Hydrologic Simulation Model for Stormwater Facility Analysis for Western Washington, Version 3.12. MGS Flood is a continuous, rainfall-runoff computer model developed specifically for stormwater facility design in Western Washington. The program uses the Hydrological Simulation Program-Fortran (HSPF) routine to compute runoff from rainfall. Features of the program include: (1) a routing routine that uses a stage-storage-discharge rating table to simulate the performance of stormwater retention/detention facilities or reservoirs, (2) routines to compute streamflow magnitude-frequency and duration statistics, and (3) graphics routines for plotting hydrographs, streamflow frequency, and duration characteristics. The program also generates runoff treatment volumes required for design of water quality facilities.

The hydrology and hydraulics for floodplain storage were evaluated using the HSPF Version 12 and HEC-RAS Version 3.1.3 models developed previously for the *Mill Creek Basin Flood Management Plan* (NHC 1999).

Hydrologic and hydraulic modeling for the conveyance systems was conducted using spreadsheets from WSDOT to determine hydrology, inlet spacing, hydraulic capacity, and cover. In addition, King County Backwater model (KCBW) was used to check the hydraulic capacity of the proposed system.

4.1 Flow Control BMPs

Flow control best management practices (BMPs) are required to reduce the impacts of stormwater runoff from impervious surfaces and land cover conversions. The following paragraphs describe the flow control approach used for these projects.

4.1.1 Floodplain Storage

A floodplain storage approach is proposed for the portion of the project within the Mill Creek (Green River) basin. This approach is preferred over conventional detention because of: (1) the lack of appropriate sites for detention considering the adjacent wetlands, (2) high groundwater, and (3) the environmental benefits of this approach compared to open ponds or underground vaults. While floodplain storage would typically be considered more difficult to permit, there are some factors that make it more desirable in this area, including well-documented downstream flooding in the Mill Creek system and current regional efforts to add more storage to the Mill Creek system (such as the Auburn Environmental Park and regional storage ponds proposed as part of the *Mill Creek Basin Flood Management Plan*). Due to the uncertainties associated with advancing an alternative approach, the design team began a series of meetings with the Department of Ecology (Ecology) to establish what information would be required and what process must be followed in order to determine if the approach is acceptable. This process would require analysis to be performed to show that certain performance criteria could be met. The goals of the meetings with Ecology were to establish how the analysis should be performed and what the appropriate design criteria should be.

Note that the floodplain storage concept is different than providing compensatory floodplain storage to mitigate for filling in the floodplain. The latter issue is discussed in Section 4.9.

4.1.1.1 Approach

Based on initial meetings with Ecology, an approach was developed to demonstrate that floodplain storage will mitigate the increase in peak runoff from the impervious surface added from the project. Ecology allows alternatives to the flow control requirement if it can be shown that the project will not adversely impact water quality and will meet the flood control objectives of the local jurisdictions. For an alternative approach to be approved, appropriate supporting data must be provided that demonstrates the alternative approach is protective of water quality and satisfies state and federal water quality laws. The intent of the flow control requirement is to control the peak flows from the project and their durations such that they do not create erosion in the receiving water body. The approach included four parts:

- Field investigations (Rapid Stream Assessment) of Mill Creek
- Flood storage siting
- Hydrologic and hydraulic analyses
- Coordination and concurrence from agencies and local jurisdictions

Field Investigations – Rapid Stream Assessment

The Rapid Stream Assessment (RSA) technique was developed by the Metropolitan Washington Council of Governments (1992) to evaluate stream health using chemical, biological, and physical indicators. The method is derived from various stream survey protocols, including the U.S. Environmental Protection Agency's Rapid Bioassessment Protocols (Plafkin et. al. 1989), the Izaak Walton League of America and Save Our Streams stream survey techniques (Kellogg 1992), and the U.S. Department of Agriculture's Water Quality Indicators Guide: Surface Waters (Terrell and Perfetti 1989).

For the SR 167 project, the RSA method was tailored to review the channel stability and erosion potential by including observations of average velocity, appearance of water quality, bank- and bed-forming material, evidence of recent or ongoing erosion, and channel shape. The RSA was performed at five locations along Mill Creek. These locations are shown in Figure 4-1.

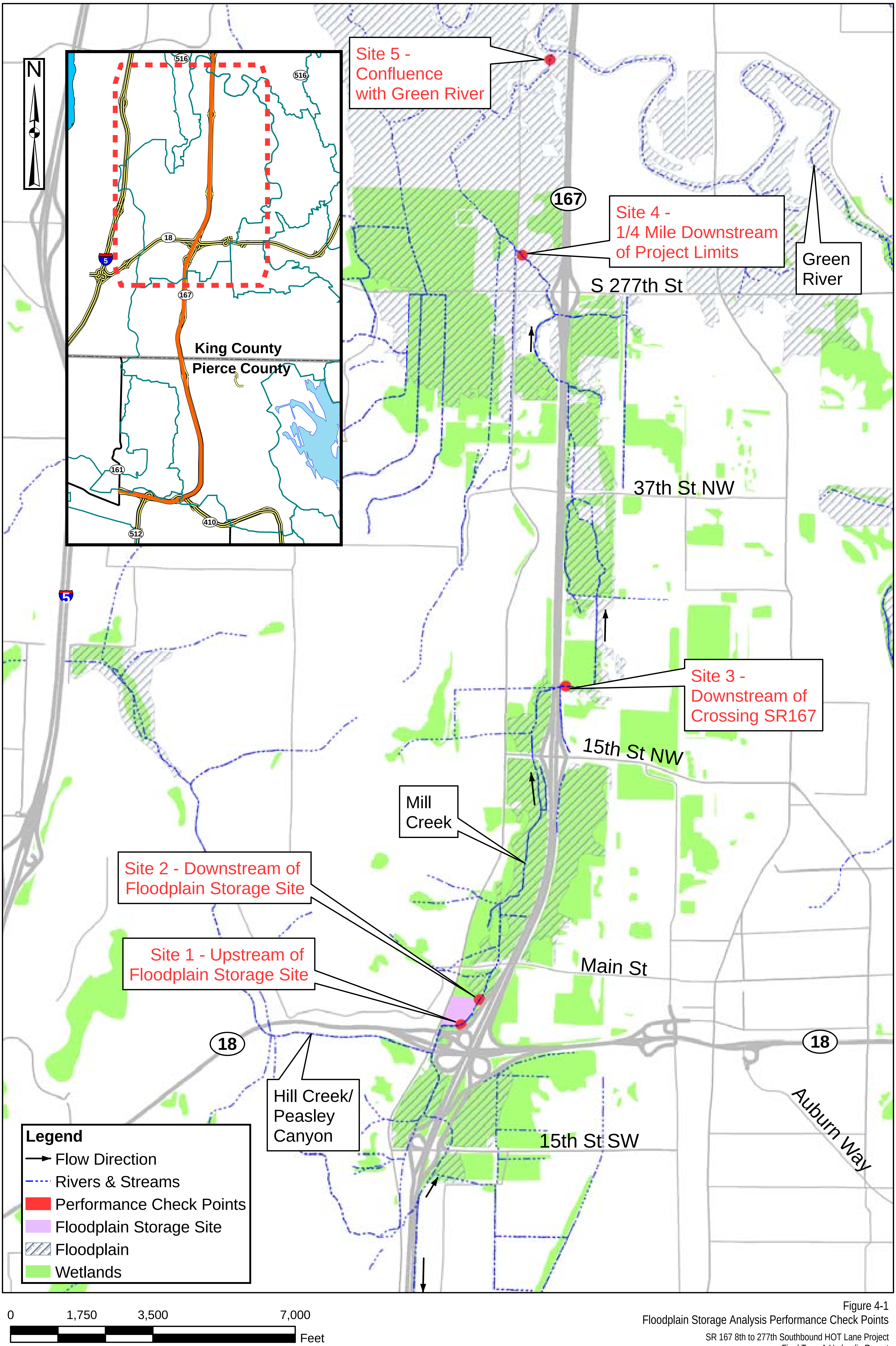

Figure 4-1
Floodplain Storage Analysis Performance Check Points

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The RSA locations shown on Figure 4-1 were selected with input from Ecology and include locations upstream and downstream of the floodplain storage site (Sites 1 and 2), partway between the floodplain storage site and the end of the project (Site 3), a quarter mile downstream of the project (Site 4), and at Mill Creek's confluence with the Green River (Site 5).

Jones & Stokes performed the Rapid Stream Assessment of the five locations on May 1, 2007, and May 31, 2007. A memorandum that discusses the methodology and results of the RSA can be found in Appendix F. This appendix includes detailed worksheets and photographs for each of the five locations. Note that the site numbering in the Jones & Stokes memorandum is the reverse of the numbering presented in this hydraulic report.

Floodplain Storage Siting

Field visits and review of topographic mapping were used to identify potential sites for floodplain storage. The sites identified as having some potential for providing floodplain storage are shown on Figure 4-2. The potential sites were evaluated based on their proximity to Mill Creek, the potential storage volume that could be developed at the site, potential for impacts to existing wetlands and other aquatic resources, and ease of property acquisition. In addition, sites located in the upper portion of the basin were preferred because the farther upstream the facility, the greater portion of the creek that would be mitigated. Based on these criteria, Site C was chosen as the preferred location for the floodplain storage option.

Hydrology and Hydraulic Analyses

Flow Control Criteria

There is no defined methodology or flow control objective for floodplain storage. The flow duration standard used by WSDOT for sizing detention facilities was not developed for low-gradient, silt-bedded stream channels like Mill Creek. Therefore, an alternative standard was proposed for sizing and evaluating floodplain storage design. This proposed criterion consists of comparing the peak annual flow rates and velocity durations within a 57-year period of record for the pre- and post-project conditions at the five sites selected along Mill Creek. Floodplain storage was sized to mitigate for the converted impervious surfaces resulting from both the Stage 4 project and the future Stage 5 project in the Mill Creek basin.

HSPF

The hydrologic impact of floodplain storage was evaluated using an HSPF model developed for the *Mill Creek Basin Flood Management Plan* (NHC 1999). The plan looked at existing and future full buildout land uses for the entire Mill Creek basin. The existing land use model was used as the basis for the HSPF modeling. Some reaches in the model were aggregated such that the length of stream between check point sites is represented as a single reach. In addition, the reach that included the floodplain storage site was split up in order to create one reach that would represent the floodplain storage. This was done to facilitate data extraction at specific check point locations along the creek.

This existing land use model was modified to represent three conditions in the Mill Creek basin:

- Pre-project conditions
- Post-project conditions with no project floodplain storage
- Post-project conditions mitigated with project floodplain storage

In addition, these HSPF models were developed for two conditions in Mill Creek: current restricted conditions and a potential future condition where restrictions have been removed. Future projects along Mill Creek, such as the project proposed by the U.S. Army Corps of Engineers, may remove some of the restrictions by removing invasive vegetation such as reed canary grass, modifying the channel cross section, or possibly replacing some culverts with larger diameter culverts.

Two modeling scenarios, one with current conveyance conditions and one with conveyance restrictions removed, were used in the analysis. If projects along Mill Creek remove restrictions and improve conveyance, the Mill Creek water surface elevation would be reduced at the proposed floodplain storage site, thus reducing the storage contained at the site. In addition, the removal of restrictions would also increase peak flows and velocities. For these reasons, the HSPF models with the conveyance restrictions removed were used for the initial assessment of peak flows and velocities in the creek. This was done because it provides a “worst case” depiction of the long-term erosion potential when using floodplain storage as an alternative mitigation and also reduces water surface elevations which reduce the amount of active floodplain storage. The HSPF models with the more restricted conveyance, which is more representative of the current condition, were used to assess water level changes at the floodplain storage site for design purposes.

The model from NHC includes a calibrated baseflow that enters the system between Sites 2 and 3. This baseflow was not adjusted for either pre- or post-project conditions.

Note also that the model did not include a few small linear detention facilities, constructed for the Stage 2 and 3 projects, that exist along the corridor, nor did it include the CAVFS proposed for the project.

The HSPF model input and output are included in Appendix E.

HEC-RAS

HEC-RAS, a steady-state step backwater model, was used to develop stage-storage-discharge relationships (FTABLEs) for Mill Creek for use in the HSPF model and to determine velocities in the channel. HEC-2 models of Mill Creek developed for the *Mill Creek Basin Flood Management Plan* (NHC 1999) were converted to HEC-RAS. The HEC-RAS models extend from the Green River to the downstream end of the crossing under SR 167 just south of SR 18.

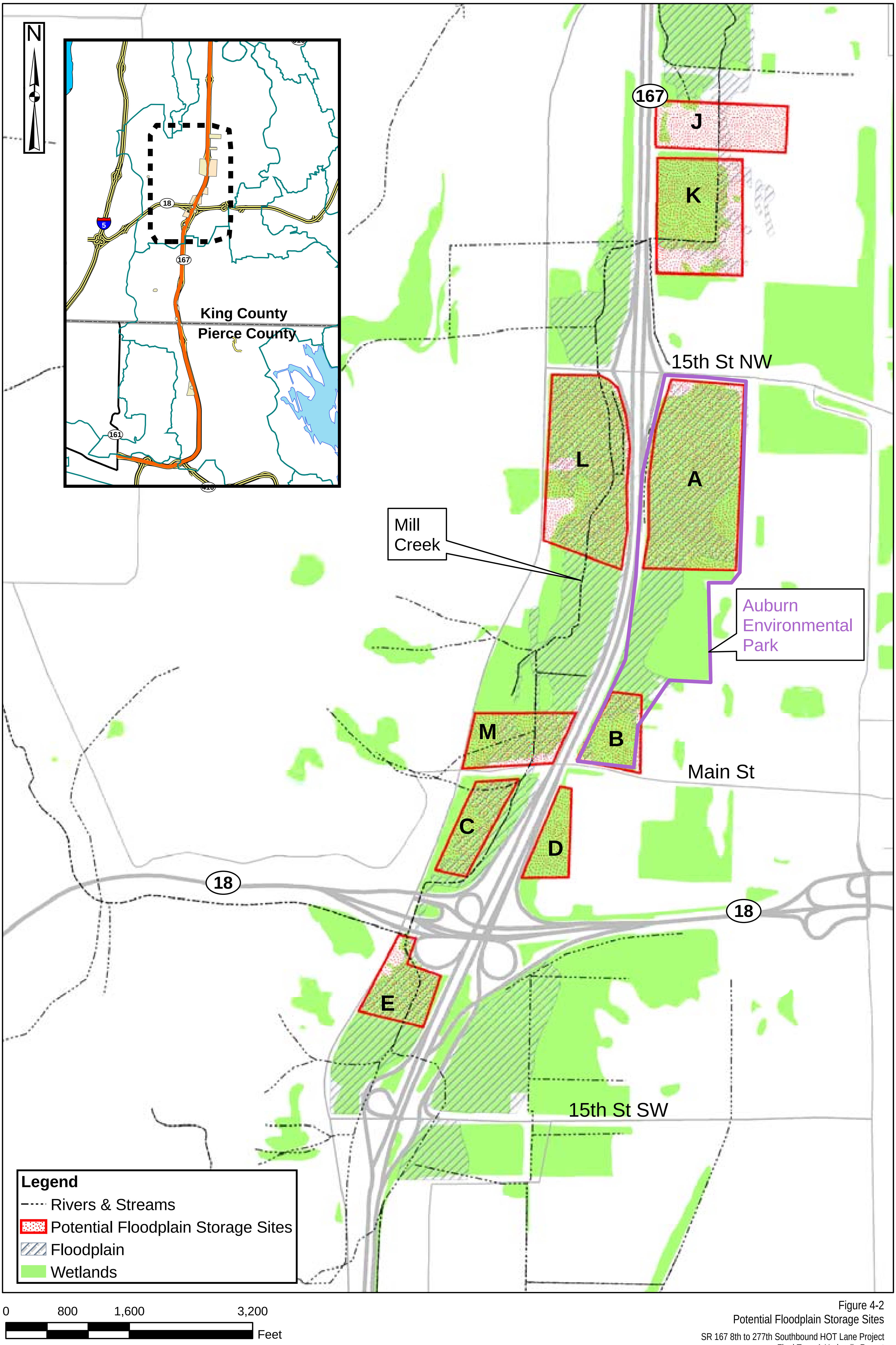

Figure 4-2
Potential Floodplain Storage Sites

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Several different HEC-RAS models were developed to represent different channel conditions. In general, they include:

- Existing channel conditions
- Existing channel conditions with floodplain storage
- Future (restrictions removed) channel conditions with floodplain storage
- Future (restrictions removed) channel without floodplain storage

The latter two channel conditions with restrictions removed were used only in the initial evaluation presented to Ecology because this condition would result in the highest velocities within the channel. The conservative scenario included the assumption that all of the structural restrictions would be removed and the Manning's roughness coefficients would be reduced to represent a maintained channel. Removing the restrictions allows for higher velocities and also reduces the water surface elevations. The lower water surface elevations reduce the amount of active floodplain storage. This results in a conservative analysis of the floodplain storage mitigation because it provides a "worst case" depiction of the long-term erosion potential. Existing channel conditions were used to develop FTABLEs for subsequent final analyses. Existing channel conditions were used in order to provide data based on existing conditions that could be used to move forward with the design.

The HEC-RAS models used for the final analysis are included in Appendix E.

Tailwater Elevation in the Green River

The HEC-RAS analysis was used to assess the velocities as an indicator of the creek's potential for erosion. The water level in the Green River has an influence on the velocities in Mill Creek, particularly in the lower reach. As noted as part of the field investigations, the lower reach showed signs of bank erosion and is of particular concern with respect to velocity.

The *Mill Creek Basin Flood Management Plan* (NHC 1999) noted that Green River stages during extreme storm events could cause backwater all the way to 37th Street NW, which would significantly reduce the velocities in Mill Creek. If a more localized storm occurred that produced high Mill Creek flows but did not produce high Green River flows (i.e., if the Green River tailwater elevation was low) the Mill Creek velocities would be higher. Therefore, a sensitivity analysis was performed to assess the impacts of a range of Green River tailwater elevations on velocity in Mill Creek. Table 4-1 shows the average Mill Creek velocities for a 2-year storm recurrence interval at the Green River confluence (between Sites 4 and 5) and a quarter mile downstream from the project limits (between Sites 3 and 4) for various Green River tailwater elevations. In addition, the table shows the maximum 2-year velocity in the lower section of Mill Creek between Sites 4 and 5 for the same Green River tailwater levels.

Table 4-1
Effects of Green River Levels on Velocities in Mill Creek

Green River Event Frequency	Green River Water Surface Elevations[1] (feet)	Average Mill Creek Velocities Between Sites 3 and 4 (fps)[2,3,4]	Average Mill Creek Velocities Between Sites 4 and 5 (fps)[2,4]	Maximum Velocity Between Sites 4 and 5 (fps)[2,4]
Average Annual	24.53	1.7	1.8	6.2
Average Winter	26.54	1.7	1.8	6.5
Average Summer	20.87	1.7	2.4	6.8
2-Year	37.92	1.4	0.6	1.6
100-Year	41.84	0.1	0.1	0.6

1. NGVD 1929
2. All velocity estimates are based on a 2-year storm flow in Mill Creek.
3. Velocities no longer impacted by tailwater upstream of 37th Street NW.
4. For site locations, see Figure 4-1.

Since there is no significant difference between the average annual, summer, and winter results, and because most of the major local storms occur during the winter months, an average winter water elevation in the Green River was used as the tailwater elevation for the Mill Creek HEC-RAS model. This tailwater would also result in more conservative (i.e., higher) velocity estimates in Mill Creek with respect to its erosion potential because the average winter water elevation in the Green River would typically be lower than would occur during a significant storm event that would most likely impact flows in both Mill Creek and the Green River. The average winter water elevation was determined by entering the average winter flow for the Green River into a HEC-2 model of the Green River obtained from a 1992 King County study. This model had been used as part of an analysis for the Interurban Bridge for the City of Tukwila.

Figure 4-3 shows the 100-year water surface profile (NGVD 1929) for Mill Creek assuming conveyance restrictions were removed in the creek and winter flows in the Green River are at an average level. In addition, the figure shows the elevations of the Green River for the conditions shown in Table 4-1. Note that the main project datum is NAVD 88; however, the models all referred to NGVD 29 datum. The conversion is NGVD 29 + 3.51 feet = NAVD 88.

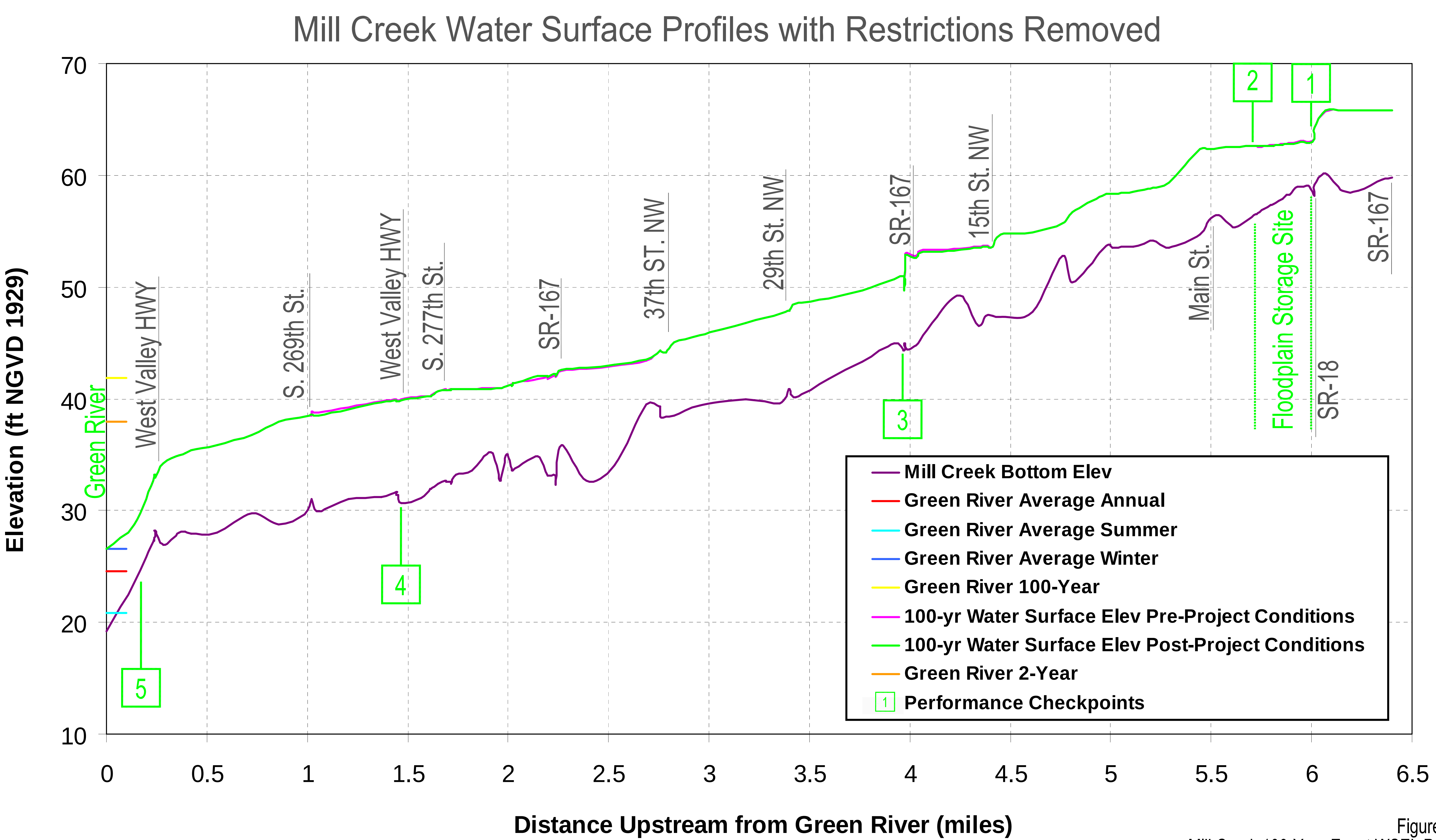

Figure 4-3
Mill Creek 100-Year Event WSEL Profile
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Coordination with Agencies and Local Jurisdictions

Several meetings were held with the agencies and local jurisdictions to identify concerns they may have with the floodplain storage approach and to gain their concurrence that floodplain storage is an acceptable flow control approach for the SR 167 project. In general, all parties were in favor of the approach. Table 4-2 lists the agency and local jurisdiction comments and concerns.

Table 4-2
Agency and Local Jurisdictions Concerns and Comments

Agency/Jurisdiction	Concern/Comments
Department of Ecology	Water quality, wetland impacts, local concurrence
Corps of Engineers	Compatibility with proposed Mill Creek habitat enhancement project
City of Kent	Compatibility with planned habitat enhancement in lower reach
City of Auburn	Consistency with nearby mitigation projects and nearby land use objectives
National Marine Fisheries Service/ U.S. Fish and Wildlife Service	Water quality and temperature (shade)

4.1.1.2 Floodplain Storage Conceptual Design

Key considerations for floodplain storage design include preventing fish stranding, providing shading, and avoiding or minimizing adverse impacts to adjacent wetlands. To prevent fish stranding, backwater channels will be included to allow fish a path out of the storage area and back into the creek when high water levels recede. Plantings will be included in the design of the floodplain storage area to provide shade. To minimize adverse impacts to the adjacent wetland mitigation sites to the north, a buffer will be maintained on the north side of the floodplain storage site.

Three configurations were considered for floodplain storage.

- In Configuration 1 (see Figure 4-4), the floodplain storage is a backup channel. As water backs up in Mill Creek, it will back up into the floodplain storage area. Cross sections for this configuration are shown in Figures 4-5 and 4-6.
- In Configuration 2 (see Figure 4-7) the Mill Creek channel would be relocated to run through the floodplain storage area. Under this confguration, the upstream end of the existing channel would be filled to help direct flow into the relocated channel. The remainder of the existing channel would remain unfilled such that Mill Creek flows could still back up into it.
- Configuration 3 (see Figure 4-8) includes a diversion structure at the upstream end of the floodplain storage area. Low flows would continue into the existing Mill Creek channel and high flows would overflow into the floodplain storage area. Flow from Mill Creek could also back up into the floodplain storage area via the connection at the downstream end of the floodplain storage site.

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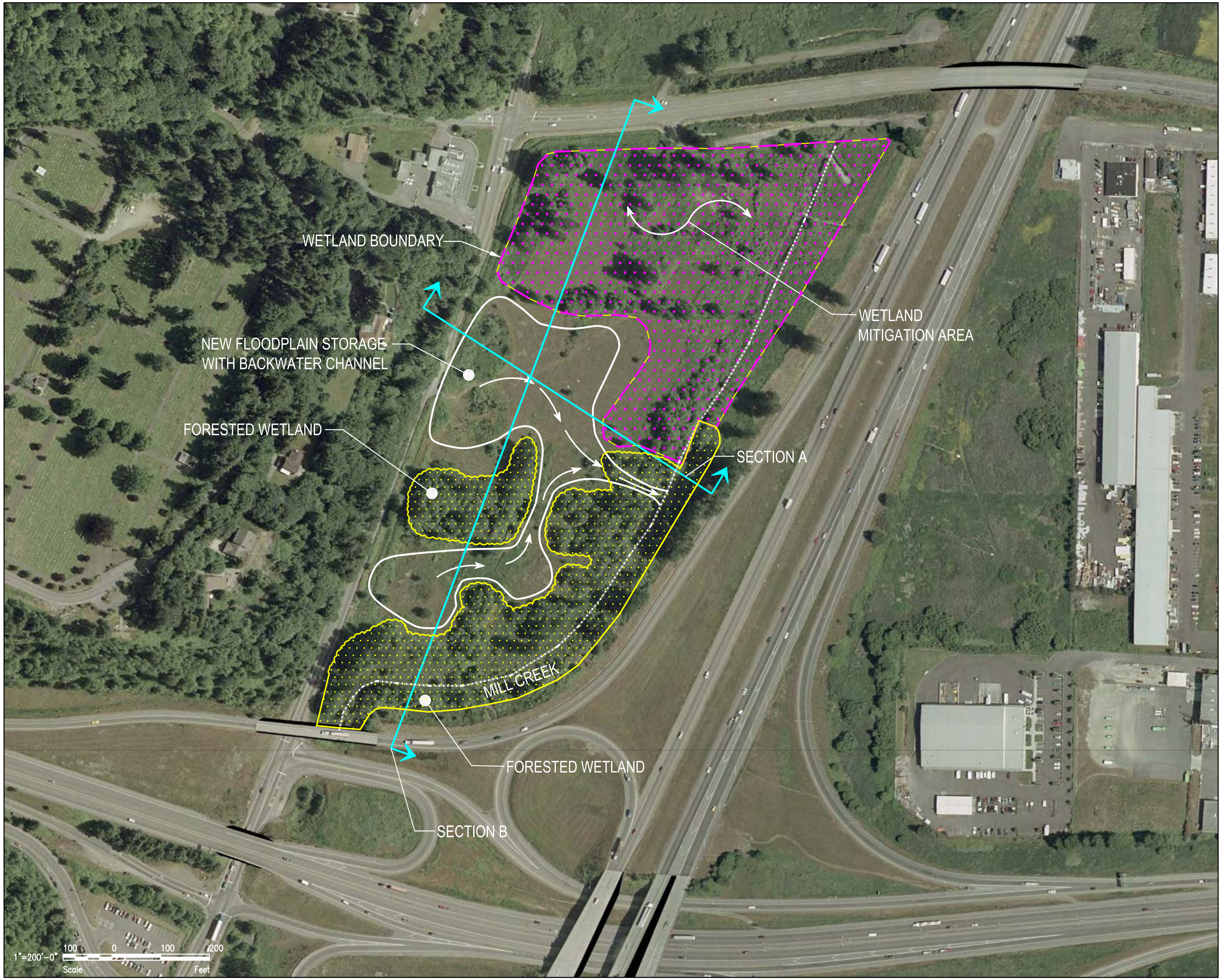

Figure 4-4
Floodplain Storage Configuration 1
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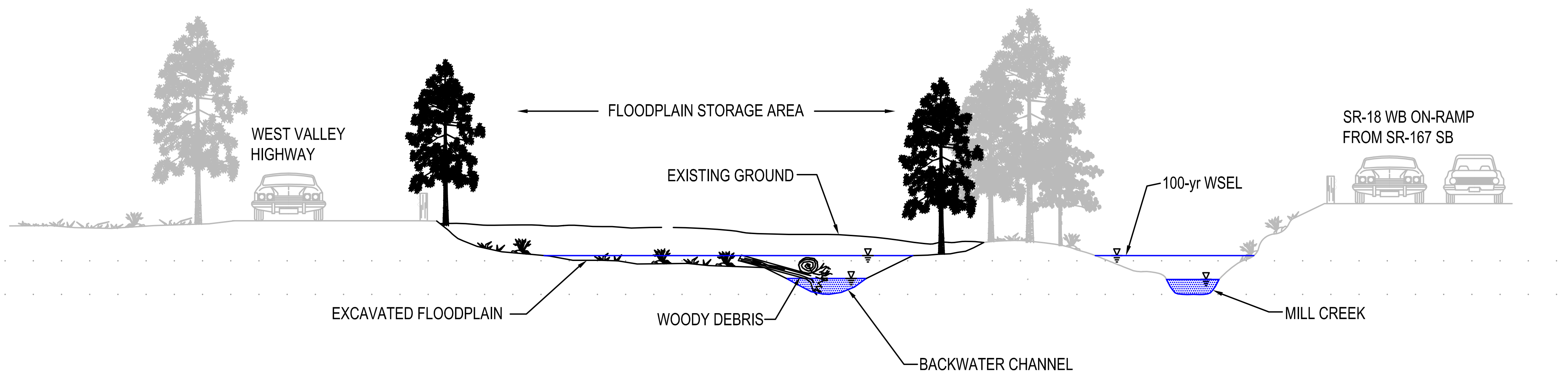
FLOODPLAIN STORAGE SITE SECTION A (SEE FIGURE 4-4)



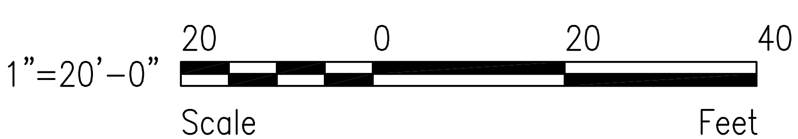


Figure 4-5
Floodplain Storage Section A
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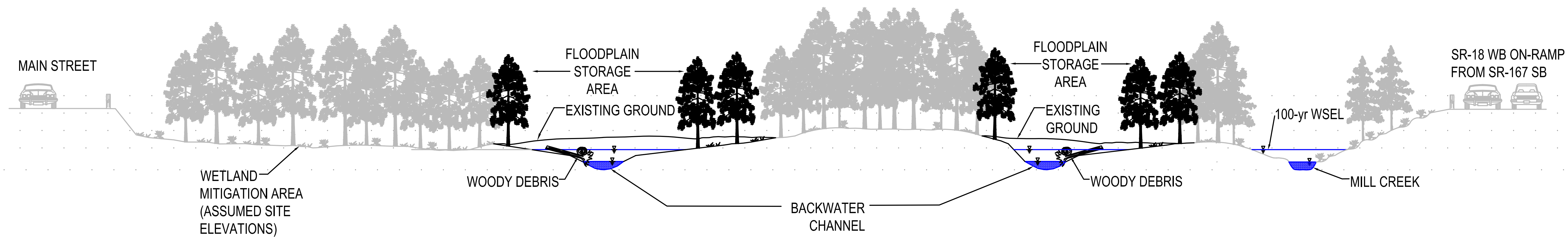
FLOODPLAIN STORAGE SITE
SECTION B (SEE FIGURE 4-4)



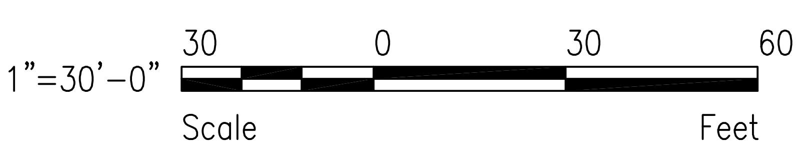


Figure 4-6
Floodplain Storage Section B
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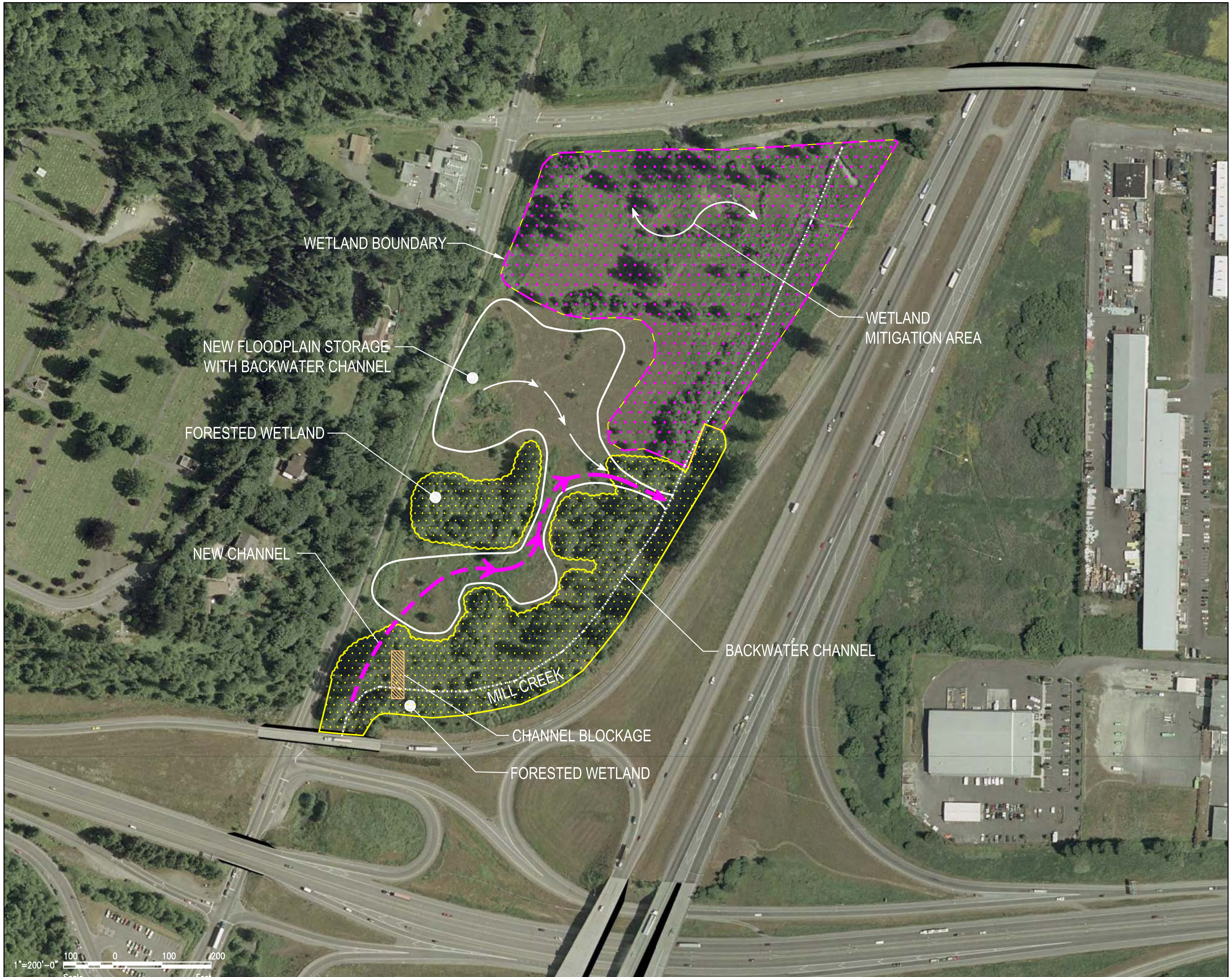

Figure 4-7
Floodplain Storage Configuration 2
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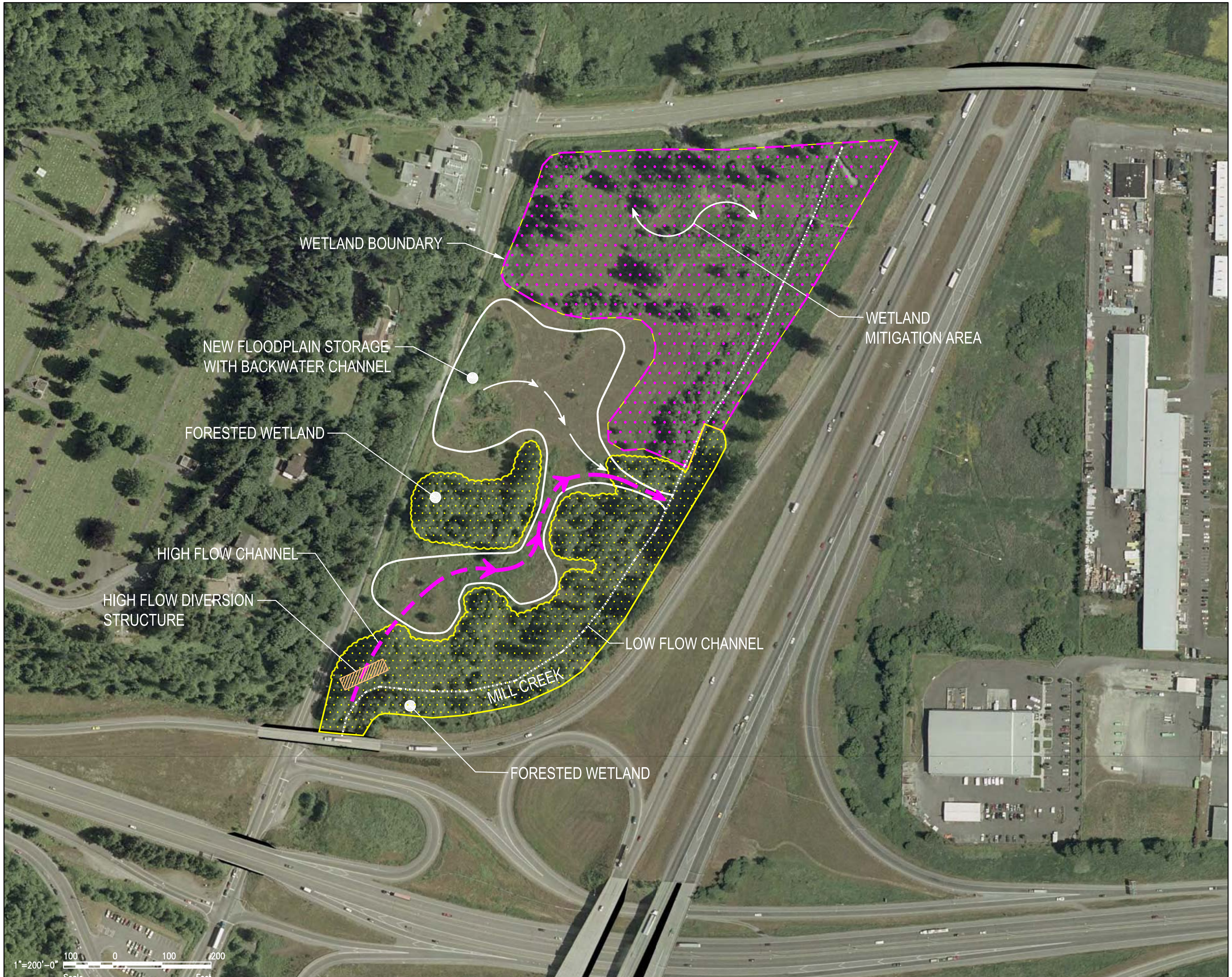

Figure 4-8
Floodplain Storage Configuration 3
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4.1.1.3 Results Presented to Ecology

Based on the approach outlined above, the performance of the floodplain storage was evaluated at the five sites which bracket the project (see Figure 4-1). Note that the hydraulic and hydrologic results for the five sites are based on the HSPF model with FTABLEs representing the conveyance with restrictions removed. Also, the CAVFS were not included in the hydrologic analysis and these may reduce flows, particularly from smaller frequent storm events. Floodplain storage performance was measured based on:

- Site conditions at each location (Rapid Stream Assessment)
- Flow changes determined using HSPF
- Channel velocities measured using HEC-RAS (averaged over the stream length)

It should be noted that the term “average peak velocity” refers to the peak annual velocity averaged over a given reach from the noted site to the next upstream site. The velocity was averaged over the reach length in order provide a velocity that represented the whole reach rather than a specific location. “Maximum peak velocity” refers to the highest average peak velocity that occurs over the entire simulation period.

The results of the analysis indicate that velocities in Mill Creek do not necessarily increase as the flow increases. Velocities tend to increase with flow until the creek water level reaches the top of the channel. However, when flow increases such that the channel overtops and extends into the floodplain, velocity can actually decrease. This is due to the initial increase in hydraulic radius and composite roughness. The relationship between average peak velocity and flow for the reaches upstream of the five sites are shown in Figures 4-9 through 4-13. Figure 4-14 shows the change in durations for four different ranges of velocities that vary from 1 to 2.5 feet per second resulting from the project.

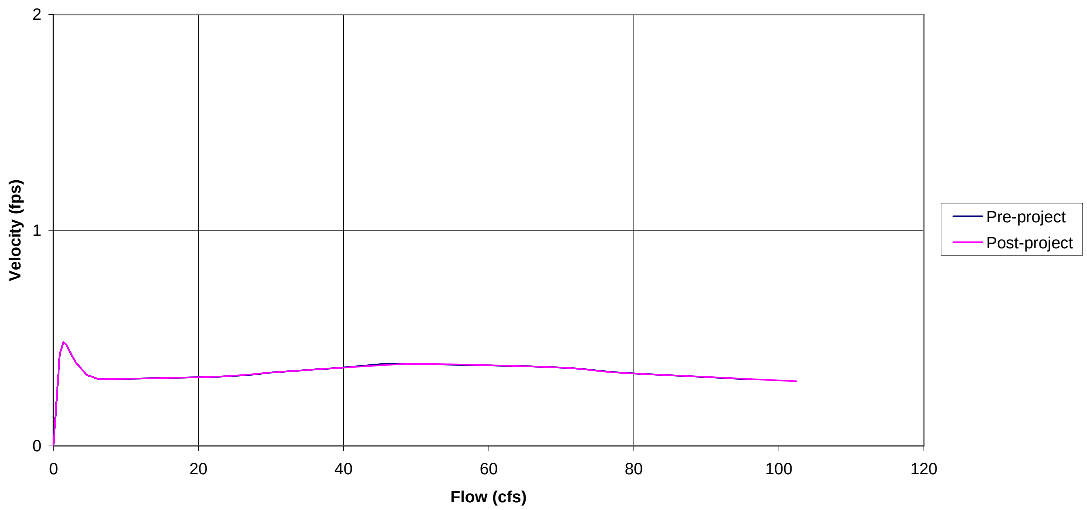


Figure 4-9. Velocity vs. Flow – Upstream of Site 1

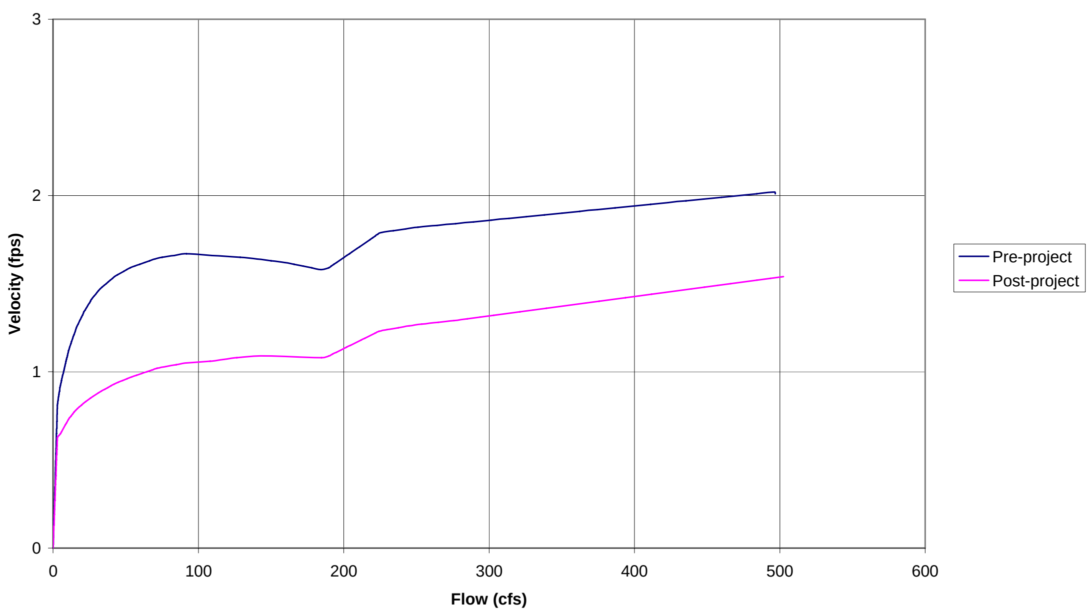


Figure 4-10. Velocity vs. Flow – from Site 1 to Site 2

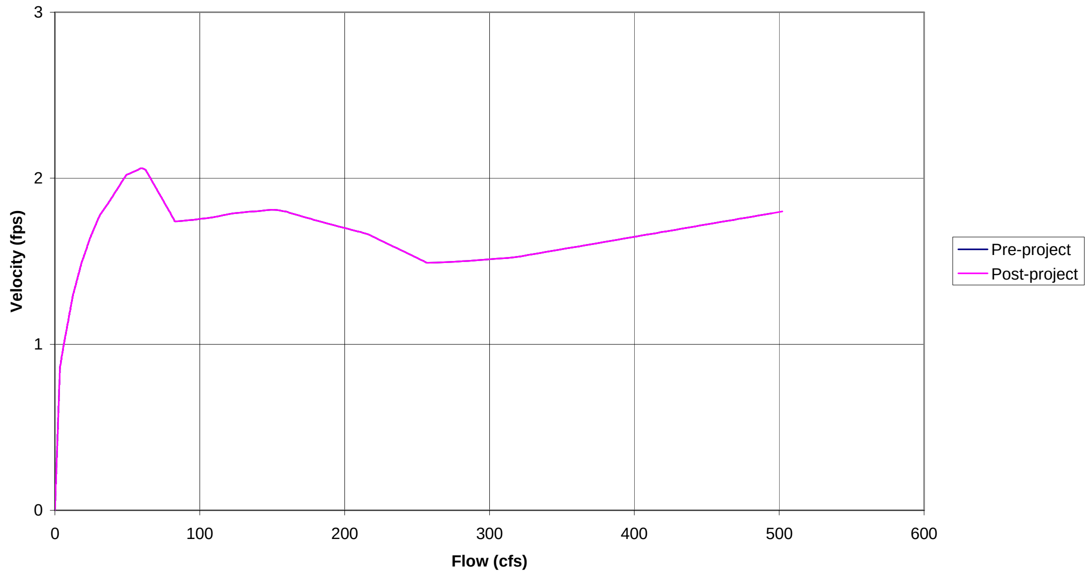


Figure 4-11. Velocity vs. Flow – from Site 2 to Site 3

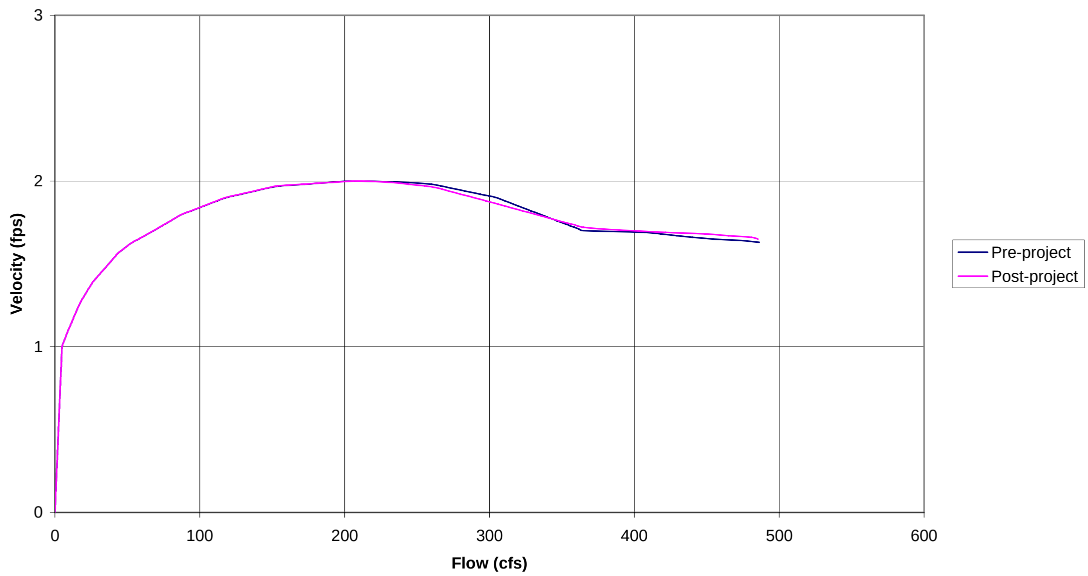


Figure 4-12. Velocity vs. Flow – from Site 3 to Site 4

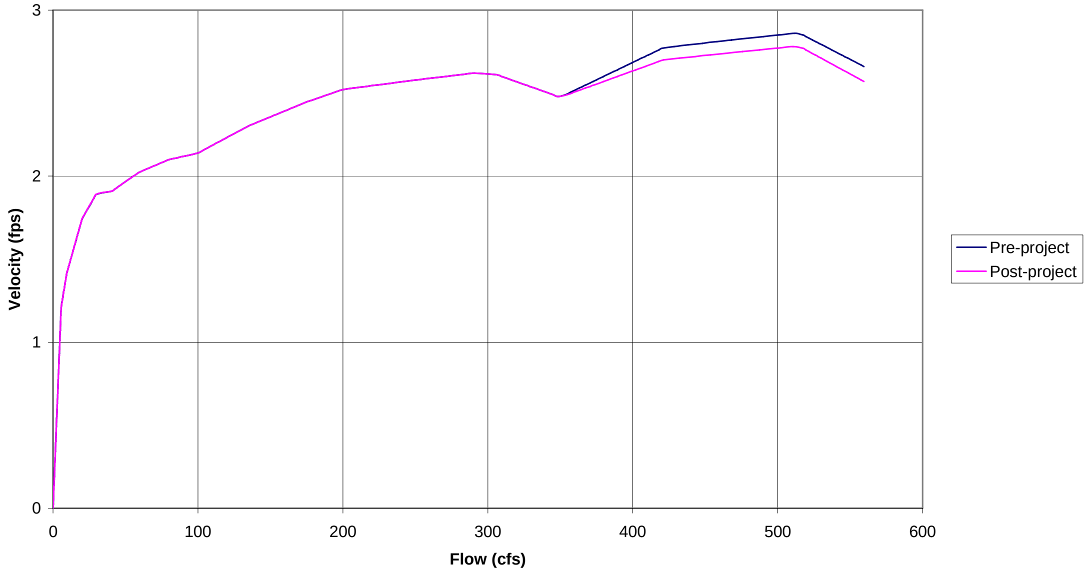
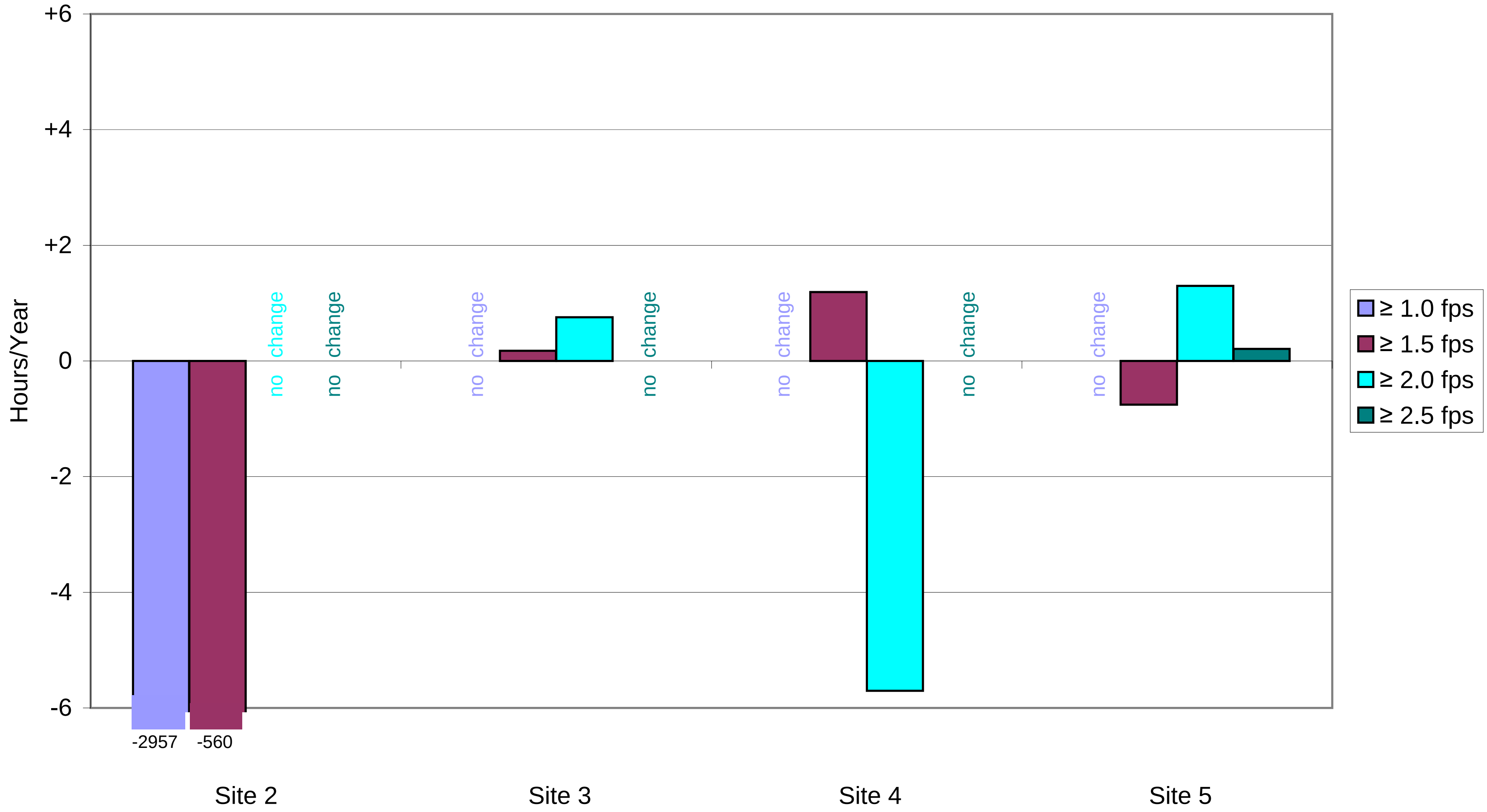


Figure 4-13. Velocity vs. Flow – from Site 4 to Site 5

Figure 4-14
Change in Velocity Durations, Based on Unrestricted Conveyance
(i.e., how much more often a given velocity will be exceeded post-project)



Site 1 Results

Site 1 is just upstream of the proposed floodplain storage site. Refer to Figure 4-1 and Photo 1. The RSA noted the channel-forming material was comprised primarily of silt, and the banks were lined with grass, shrubs, and trees. The velocity was measured at about 0.23 feet per second (fps) at the time of the RSA field visit. In addition, no evidence of erosion or deposition was noted.

The hydrologic and hydraulic analyses indicated that the peak annual flood flow was increased in 57 out of 57 years of record when compared to existing peak annual flood flow. This is because Site 1 is upstream of the floodplain storage site and therefore does not receive the benefits of the floodplain storage mitigation. In the existing condition, the average peak velocity in the reach upstream of Site 1 is 0.5 fps. In the post-project mitigated conditions, the average peak velocity is also 0.5 fps.

Site 2 Results

Site 2 is just downstream of the floodplain storage site. Refer to Figure 4-1 and Photo 2. The RSA noted the channel-forming material was comprised primarily of silt, and the banks were lined with grass, willows, and cattails. The velocity was too slow to be measured accurately at the time of the RSA field visit. In addition, there was no evidence of erosion or deposition.

Photo 1. Performance Check Point Site 1

Photo 2. Performance Check Point Site 2

The hydrologic and hydraulic analyses indicated that the post-project peak annual flood flow was reduced in 38 out of 57 years of record when compared to existing peak annual flood flow. Figure 4-14 shows that duration of velocities less than 2 fps decrease significantly post project. Velocities at this site did not exceed 2 fps. In the existing condition, the average peak velocity in the reach between Site 2 and Site 1 ranges from 1.6 fps to 1.8 fps. In the post-project mitigated conditions, the average peak velocity ranges from 1.0 fps to 1.3 fps.

Site 3 Results

Site 3 is located at the SR 167 crossing just downstream of 15th Street NW. Refer to Figure 4-1 and Photo 3. The RSA noted the channel-forming material was comprised primarily of silt, and the banks were lined with grass, willows, and blackberries. The velocity was measured to be about 0.89 fps at the time of the RSA field visit. In addition, there was no evidence of erosion or deposition.

The hydrologic and hydraulic analyses indicated that the post-project peak annual flood flow was reduced in 57 out of 57 years of record when compared to existing peak annual flood flow. Figure 4-14 shows that there is a small increase in velocity durations (less than 1 hour/year) for velocities greater than or equal to 1.5 fps and greater than or equal to 2.0 fps but no velocities exceeded 2.5 fps. In the existing condition, the average peak velocity in the reach between Site 2 and Site 3 is 2.1 fps. In the post-project mitigated conditions, the average peak velocity is also 2.1 fps.

Site 4 Results

Site 4 is located a quarter mile downstream of the end of the project (downstream of S 277th Street). Refer to Figure 4-1 and Photo 4. The RSA noted the channel-forming material was comprised primarily of silt and the banks were lined with grass and blackberries. The velocity was measured to be about 0.89 fps at the time of the RSA field visit. In addition, there was no evidence of erosion or deposition.

The hydrologic and hydraulic analyses indicated that the post-project peak annual flood flow was reduced in 56 out of 57 years of record when compared to existing peak annual flood flow. Figure 4-14 shows that there is a small increase in velocity durations (about 1 hour/year) for velocities greater than or equal to 1.5 fps and a decrease in velocity durations (about 6 hours/year) for velocities greater than or equal to 2.0 fps. The velocities never exceeded 2.5 fps at this location. In the existing condition, the average peak velocity in the reach between Site 4 and Site 3 ranges from 1.9 fps to 2.0 fps. In the post-project mitigated conditions, the average peak velocity also ranges from 1.9 fps to 2.0 fps.

Photo 3. Performance Check Point Site 3

Photo 4. Performance Check Point Site 4

Site 5 Results

Site 5 is about 1000 feet upstream of the confluence with the Green River. Refer to Figure 4-1 and Photo 5. The channel profile steepens through this reach as it drops down the banks of the Green River. The RSA noted the channel-forming material was comprised primarily of silt, and that trees, shrubs, and grasses were growing on portions of the banks. The velocity was measured at 1.24 fps at the time of the RSA field visit. Bank erosion and slumping were evident but no channel scour.

The hydrologic and hydraulic analyses indicated that the post project peak annual flood flow was reduced in 52 out of 57 years of record when compared to existing peak annual flood flow. Figure 4-14 shows a small decrease (about 1 hour/year) in velocity durations for velocities greater than or equal to 1.5 fps and a small increase in velocity durations (about 1 hour/year) for velocities greater than or equal to 2.0 fps. Velocities never exceed 2.7 fps at this location for the post-project condition. In the existing condition, the average peak velocity in the reach between Site 5 and Site 4 ranges from 2.2 fps to 2.8 fps. In the post-project mitigated conditions, the average peak velocity ranges from 2.2 fps to 2.7 fps.

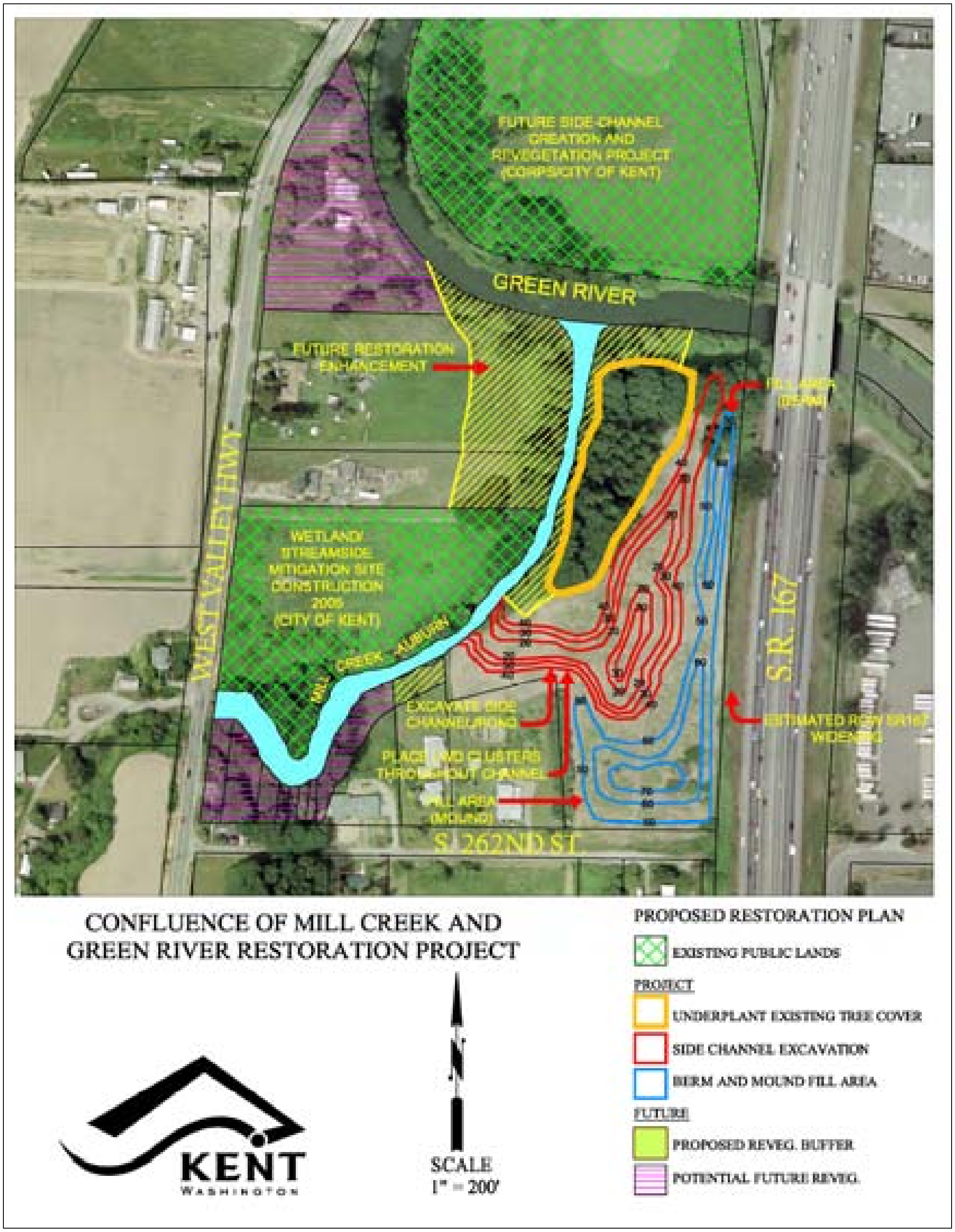
This area has also been identified by the City of Kent as the location for a future restoration project. As can be seen in the project concept shown in Figure 4-15, the future project includes an off-channel pond that would be used for fish rearing. This pond would also function as floodplain storage and may serve to reduce peak flows and velocities in this reach. Kent has not yet identified funding for this project.

Photo 5. Performance Check Point Site 5

Floodplain Storage Site Inundation Duration Changes

The identified floodplain storage site is adjacent to wetlands, some of which were created as mitigation for area projects. These wetlands are likely fed by hillside seeps and/or high groundwater on the valley floor and creek overflows. One of the goals of the floodplain storage design will be to minimize potential impacts to the adjacent wetlands.

Protection of wetland plant and animal communities depends on controlling the wetland's hydroperiod (i.e., the pattern of fluctuation of water depth and the frequency and duration of exceeding certain levels). The initial results from the Mill Creek HSPF model comparing the water-level fluctuations under existing conditions with fluctuations with the proposed project including floodplain storage indicate that the fluctuations for wetlands are reasonable. The change in mean annual and mean monthly water-level fluctuations from existing conditions does not exceed 5 cm at the floodplain storage site. In addition, the modeling indicated that water levels in the floodplain storage site deviate from existing levels by less than 15 cm. Both the fluctuations and the water-level deviations are within the limit set forth in Ecology's 2005 *Stormwater Management Manual for Western Washington* (Appendix 1-D, Wetlands and Stormwater Management Guidelines) for wetlands with high vegetation and species richness. Since there were no water-level excursions of 15 cm or more, a duration analysis of these excursions was not performed.



(Source: City of Kent)

Figure 4-15. Future Restoration Project at Performance Check Point Site 5

Groundwater Concerns

At the June 26, 2007, meeting with Ecology, a concern was raised regarding groundwater. Groundwater could impact the site in two ways. A high groundwater level could limit the available active storage at the site during a flood event. In addition, the excavation of the floodplain storage site could impact the wetland mitigation sites located to the north. However, the intent of this project is to design the floodplain storage in a manner that would avoid or minimize any impacts. Subsequently, groundwater data was collected and reviewed. This is discussed in Section 4.1.1.4.

Conclusions

Based on the Rapid Stream Assessment and the hydrologic and hydraulic analyses, the mitigated peak flows downstream of the floodplain storage are generally equal to or less than existing peak flows. In addition, it appears that erosion due to the project is unlikely because:

- The velocities are generally low throughout Mill Creek under pre- and post-project conditions.
- The floodplain storage is effective in reducing most post-project flows to be equal to or lower than pre-project conditions.
- The occasional slight increases in flow that are observed are very short in duration and still result in velocities in the non-erosive range (less than 3 fps).[1]
- Water quality treatment in the form of CAVFS, which will infiltrate some of the runoff volume, will be provided along portions of the road. This additional mitigation was not included in the hydrologic and hydraulic analyses discussed above and may reduce the peak flow of small storm events.

In addition, the wetland impacts are considered insignificant because the changes in water-level fluctuations are within acceptable levels.

4.1.1.4 Updated Analysis

Subsequent to the work described in the previous paragraphs, several updates to the floodplain storage analysis were performed to incorporate new data and to incorporate Ecology and MAP Team comments regarding the environmental function of the resulting wetland. The floodplain storage concept presented to Ecology (as described in the preceding section) was updated using updated information including field survey of the site and groundwater levels collected from piezometers on the site. The analysis also took into account the updated project impervious area for the 60 percent design. The grading plan was also modified to provide more opportunity for shade to mitigate any potential temperature impacts.

[1] Based on values for noncolloidal silt loam and noncolloidal alluvial silt in the table, “Maximum permissible velocities recommended by Fortier and Scobey and the corresponding unit-tractive-force values converted by the U.S. Bureau of Reclamation.” (McCuen 1989, p. 703.)

The objective of the updated analysis was to confirm available flood storage volume and related performance given more accurate site data, the 60 percent proposed floodplain storage site grading plan and existing Mill Creek channel conditions without any of the existing restrictions removed. Existing Mill Creek conveyance conditions were selected because that would provide information about the existing site conditions necessary to proceed with design. Predicted water levels based on existing conditions were necessary to develop a planting plan at the site.

The total new impervious surface area in the Mill Creek basin did not change significantly from 60 to 90 percent , and thus no further analysis updates were performed.

In addition, the City of Auburn would like to use part of the floodplain storage site to mitigate for the water quality and quantity impacts resulting from a future widening of West Valley Highway. Therefore, area was set aside on the north side of the site for use by the City of Auburn. Discussion of this area is included in Appendix G.

The piezometer data at the floodplain storage site collected from April 18, 2007, to March 19, 2008, indicates that groundwater levels at the site vary from about elevation 66.0 to 68.8 (see Table 4-3). The locations of the piezometers at the floodplain storage site are shown on Figure 4-16. These groundwater levels appear to correlate strongly with the typical water levels in the creek (varying from an ordinary high water elevation of about 66.0 to a predicted high of 67 to 68 feet at the floodplain storage site). Based on this information, it was surmised that Mill Creek exerts a strong influence on the groundwater level at the site and if the site is excavated below ground level, the water level in the excavation would come to equilibrium with the water level in the creek. As a result, it is likely that groundwater would not be higher than creek water levels once the site is excavated and would therefore not have much effect on storage at the floodplain storage site.

Table 4-3
Floodplain Storage Site Groundwater Conditions

Piezometer	High Groundwater Elevation[1]	Low Groundwater Elevation[1]
P4	68.8	66.6
P5	68.7	66.7
P6	68.5	66.0

1. Record from 4/18/07 to 3/19/08

Updated Floodplain Storage Site Layout and Design

Based on the creek water level, groundwater data, new site survey and considerations for the future West Valley Highway project, the floodplain storage site was reconfigured such that during normal flow levels in the creek, the permanent water pool would be confined primarily to a low-flow channel. In addition, the grading was modified to narrow the areas of open water such that plantings on the perimeter would shade more of the floodplain storage. The design layout is shown in Figure 4-17.

Design features, such as a log jam at the connection point to Mill Creek and turf mat reinforcement, were added to help stabilize the site over the long term but do not affect the functioning of the floodplain storage for flow control. These features will help maintain the site configurations, which should help keep it functioning as designed.

A log jam was added at the connection of the floodplain storage site to Mill Creek. The log jam is located where the floodplain storage connects to the creek and is aligned along the former creek channel bank. According to the HEC-RAS analysis, without the log jam, the velocity in Mill Creek decreases slightly as it passes the connection point with the floodplain storage. This is due to the increase in cross-sectional area at the connection point. The concern is that sediment may drop out where the velocity decreases and over time, this could inhibit the hydraulic connection between the creek and the floodplain storage site. Adding the log jam will help confine the Mill Creek active flow to the creek channel, reducing the potential reduction in velocity. At the same time, the log jam is porous, so it will not inhibit flow from entering the floodplain storage site. It should be noted that due to the sill downstream in Mill Creek near Main Street, the velocities throughout the portion of Mill Creek adjacent to the floodplain storage site are low; therefore, any velocity drops due to changes in channel shape are expected to be minimal. The log jam feature was added as a conservative measure and is shown in Appendix K.

Turf mat reinforcement was also added along the south bank of the floodplain storage site. Mill Creek takes a 90-degree turn immediately south of this area. The turf mat reinforcement is added to help prevent the southern slope of the floodplain storage from eroding if Mill Creek were to not make the 90-degree turn and overtop its banks. The HEC-RAS model indicates that the area between the floodplain storage site and the 90-degree bend in Mill Creek is high enough that it would not be overtopped in the 100-year event, so the turf mat reinforcement is being added as a precautionary measure. If the creek overtopped its banks in this area, it could cause head cutting of the south bank of the floodplain storage site, which might eventually lead to the creek being rerouted through the storage site. This would not affect the floodplain storage function for flow control; however, the existing channel through this area is one of the more shaded portions of Mill Creek and therefore, desirable for fish habitat. The preference is to maintain Mill Creek in its existing channel. It should be noted that the portion of land between Mill Creek and the floodplain storage site on the south end contains an existing stand of established trees that should remain and that will also aid in maintaining the creek in the existing channel. The landscape designer could also consider adding willow live stakes or similar vegetation to increase the roughness through this area. If the landscape design is hindered by the presence of the turf mat reinforcement, another option to reinforce the slope would be to bury a sill of large rock at the south edge of the floodplain storage site to act as a grade control.

Figure 4-16
Piezometer Locations at Floodplain Storage Site
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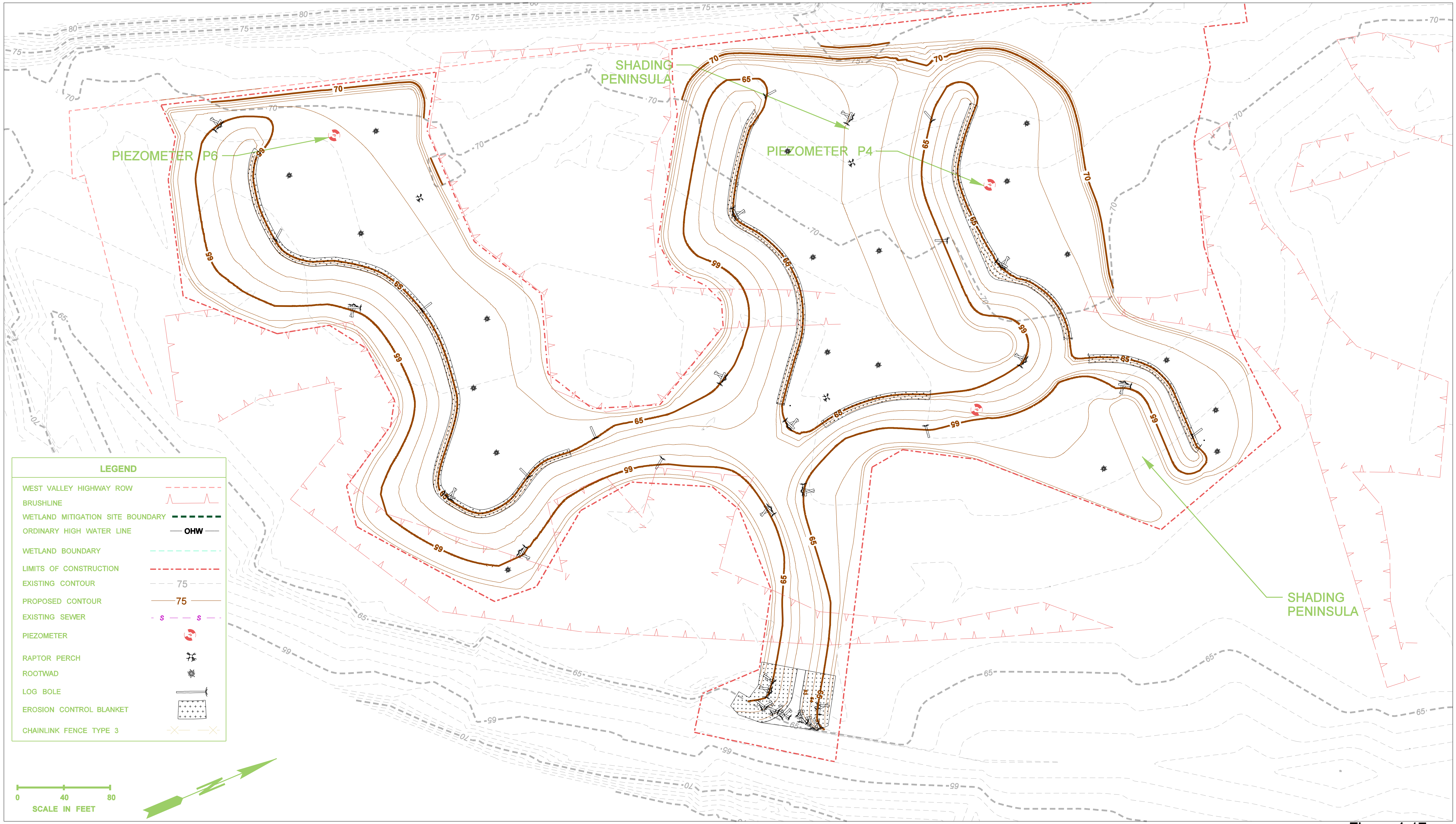

Figure 4-17
Floodplain Storage Layout
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Updated Floodplain Storage FTABLE

As before, HEC-RAS was used to help develop stage-storage-discharge relationships (FTABLEs) for Mill Creek for use in the HSPF model with a couple of modifications. As discussed above, the HEC-RAS model of the existing Mill Creek channel conditions without any of the existing restrictions removed was used. However, to develop the FTABLE at the floodplain storage site, the model was truncated upstream of Main Street and the water levels recorded by WSDOT maintenance at this location were used as the tailwater conditions. Using actual recorded water surface elevations downstream of the floodplain storage site increased accuracy of the water level predictions. The more accurate the water level predictions, the easier it is to identify appropriate plants for the site that will survive. In addition, instead of using the storage estimated in the HEC-RAS model, InRoads was used to determine the storage volume in the stage-storage-discharge relationship. While this assumes that the water surface elevation is level throughout the site, this assumption is reasonable because the velocities in the creek are minimal. In addition, using InRoads volumes provides a much more accurate assessment of the volume associated with a particular elevation than interpolating between HEC-RAS cross sections.

4.1.1.5 Updated Results

The updated results were prepared for the same five performance check point sites as with the previous analysis that was presented to Ecology (see Figure 4-1). Note that the updated hydraulic and hydrologic results are based on the HSPF model with the existing restricted conveyance represented, while the previous results (presented to Ecology) were based on conditions with conveyance restrictions removed. Existing Mill Creek channel conditions were used in order to provide data based on existing conditions that could be used to move forward with the design. As before, the CAVFS were not included in the hydrologic analysis and these may reduce flows, particularly from smaller frequent storm events. More specific results are described below. As before, the term “average peak velocity” refers to the peak annual velocity averaged over a given reach from the noted site to the next upstream site. The velocity was averaged over the reach length in order to provide a velocity that represented the whole reach rather than a specific location.

Site 1 Results

Based on existing restricted conveyance in Mill Creek, the hydrologic and hydraulic analyses indicated that the peak annual flood flow was increased in 57 out of 57 years of record when compared to existing peak annual flood flow. As before, this is because Site 1 is upstream of the floodplain storage site and therefore does not receive the benefits of the floodplain storage mitigation. The maximum increase in peak flow was about 0.7 cfs or 0.4 percent. However, the velocities at this site never exceed 1 fps, which is below the threshold velocity of about 3 fps.[2] In the existing condition,

[2] Based on values for noncolloidal silt loam and noncolloidal alluvial silt in the table, “Maximum permissible velocities recommended by Fortier and Scobey and the corresponding unit-tractive-force values converted by the U.S. Bureau of Reclamation.” (McCuen 1989, p. 703.)

the average peak velocity in the reach upstream of Site 1 is 0.3 fps to 0.4 fps. In the post-project mitigated conditions, the average peak velocity is also 0.3 fps to 0.4 fps.

Site 2 Results

Based on existing restricted conveyance in Mill Creek, the hydrologic and hydraulic analyses indicated that the peak annual flood flow was reduced in 36 out of 57 years of record when compared to existing post-project peak annual flood flow. The maximum increase in peak flow was about 0.5 cfs or about 0.3 percent. However, the velocities at this site never exceed 2.5 fps, which is below the threshold velocity. In the existing condition, the average peak velocity in the reach between Site 2 and Site 1 ranges from 1.2 fps to 2.4 fps. In the post-project mitigated conditions, the average peak velocity also ranges from 1.1 fps to 1.8 fps.

Site 3 Results

The hydrologic and hydraulic analyses indicated that the post-project peak annual flood flow was reduced in 51 out of 57 years of record when compared to existing peak annual flood flow. The maximum increase in peak flow was about 0.1 cfs or 0.1 percent. However, the velocities at this site never exceed 1 fps, which is below the threshold velocity. In the existing condition, the average peak velocity in the reach between Site 2 and Site 3 ranges from 0.6 fps to 0.7 fps. In the post-project mitigated conditions, the average peak velocity is 0.6 fps.

Site 4 Results

The hydrologic and hydraulic analyses indicated that the post-project peak annual flood flow was increased in 48 out of 57 years of record when compared to existing peak annual flood flow. The maximum increase in peak flow was about 0.1 cfs or 0.1 percent. However, the velocities at this site never exceed 1 fps, which is below the threshold velocity. In the existing condition, the average peak velocity in the reach between Site 4 and Site 3 is 0.8 fps. In the post-project mitigated conditions, the average peak velocity is also 0.8 fps.

Site 5 Results

The hydrologic and hydraulic analyses indicated that the post-project peak annual flood flow was increased in 36 out of 57 years of record when compared to existing peak annual flood flow. The maximum increase peak flow was about 0.1 cfs or 0.1 percent. The duration of velocities greater than 1 fps but less than 1.5 fps in this reach increased less than 1 hour per year. Velocities never exceed 1.5 fps, which is below the threshold velocity. In the existing condition, the average peak velocity in the reach between Site 5 and Site 4 ranges from 0.8 fps to 1.2 fps. In the post-project mitigated condition, the average peak velocity also ranges from 0.8 fps to 1.2 fps.

Volume Comparison

An evaluation of runoff volume for the largest simulated flow was conducted to determine how increases in runoff attributed to the Stage 4 project compare to the volume of flood storage being created at the floodplain storage site. The largest storm event during the 57-year precipitation record is 1951. Based on the statistical analyses, this equates to approximately a 100-year event

The effects of the project (converting impervious surface) are predicted to result in the runoff volume for the 1951 storm, increasing from 7.4 to 12.5 acre-feet of runoff for the collective storm period lasting 5 days (from February 6 to February 11, 1951). This is an increase in runoff volume of about 5 acre-feet.

Based on the proposed grading, the active storage used at the floodplain storage site for the 1951 storm is also about 5 acre-feet. Therefore, the flood storage volume provided by the site compensates for the increased runoff generated by the proposed roadway improvements.

Water Levels for Wetland Planting

Figures 4-18 and 4-19 show the average and maximum extent of the floodplain storage during the period of record. March represents the beginning of the growing season and September represents a dry weather month. Figure 4-20 shows typical cross sections of the site plotted with the March and September water levels. These figures were prepared to facilitate a planting plan that assures adequate shading to help prevent temperature adverse impacts at the site.

Conclusions

The updated analysis shows that, like the analysis originally presented to Ecology, erosion due to the project is unlikely because the velocities are generally low throughout Mill Creek under pre- and post-project conditions. The occasional slight increases in velocity that are observed are very short in duration and are still in the non-erosive range (less than 2.5 fps). Most peak flood flows are reduced. When they do increase, the increase is slight, on the order of only 0.1 to 0.7 cfs or about 0.4 to 0.1 percent of the corresponding pre-project peak flood flow.

4.1.1.6 Existing Detention Facilities in the Mill Creek Basin

Design documents provided by WSDOT show that flow control facilities were constructed as part of the SR 167 Stage 2 and 3 HOV projects that lie within the Stage 4 project corridor where some roadway widening is proposed. This section includes an evaluation of the potential impacts to these existing facilities from the project.

Facilities Constructed in Stage 2

The information regarding the existing Stage 2 detention facilities were taken from the following WSDOT-provided sources:

- SR 167 15th Street SW to South Grady Way Hydraulic Report (CTS Engineers 1993)
- Hydraulic Report Supplement: SR 167 15th Street S.W. to South Grady Way—Stage 2 (WSDOT 1995)
- Contract Plans for Construction of SR 167 15th NW to 84th Avenue So. HOV and SC & DI—Stage 2 (1995)

Under the Stage 2 contract, several roadside ditches within the Mill Creek basin were modified to serve as linear flow control ponds. The detention criteria for these ponds were as follows:

- **City of Auburn section:** Detention of the 25-year post-project runoff volume with a peak release rate not exceeding that from the 2-year pre-project event (24-hour storms using SBUH methodology).
- **City of Kent (Zone #1) section:** Detention of the 25-year post-project runoff volume with a peak release rate not exceeding that from the 10-year pre-project event (24-hour storms using SBUH methodology).

It should be noted that the present configuration of these ditches and outlet controls is unknown at this time because an as-built survey has not been conducted on these facilities.

The following facilities constructed as part of Stage 2 are shown in Appendix A.

Table 4-4
Existing Stage 2 Detention Facilities

Stage 2 Facility ID	Approximate Location
Pond-M2-2	Northbound Shoulder Just North of 15th St NW
Pond-M3-2	Southbound Shoulder Just North of 15th St NW
Pond-M4-2	Northbound Shoulder Just North of Pond-M2-2
Pond-M5-2	Southbound Shoulder Just North of Pond-M3-2
Pond-M6-2	Northbound Shoulder North of Pond-M4-2
Pond-M7-2	Southbound Shoulder North of Pond-M5-2
Pond-M8-2	Southbound Shoulder North of Pond-M7-2
Pond-M9-2	Northbound Shoulder North of Pond-M6-2
Pond-M10-2	Northbound Shoulder North of Pond-M9-2
Pond-M11-2	Southbound Shoulder North of Pond-M8-2

Facilities Constructed in Stage 3

Information regarding the existing detention facilities designed as part of Stage 3 was taken from the following WSDOT-provided sources:

- The drainage portion of a plan set dated 11/30/2005 for the SR 167 15th St SW to S 180th Street—Stage 3 Project
- Addenda 3, 5, and 7 for the SR 167 15th Street SW to S 180th Street—Stage 3 Project
- Hydraulic Report Supplement #3: SR-167 15th Street SW to S Grady Way HOV and SC&DI (WSDOT 2005)

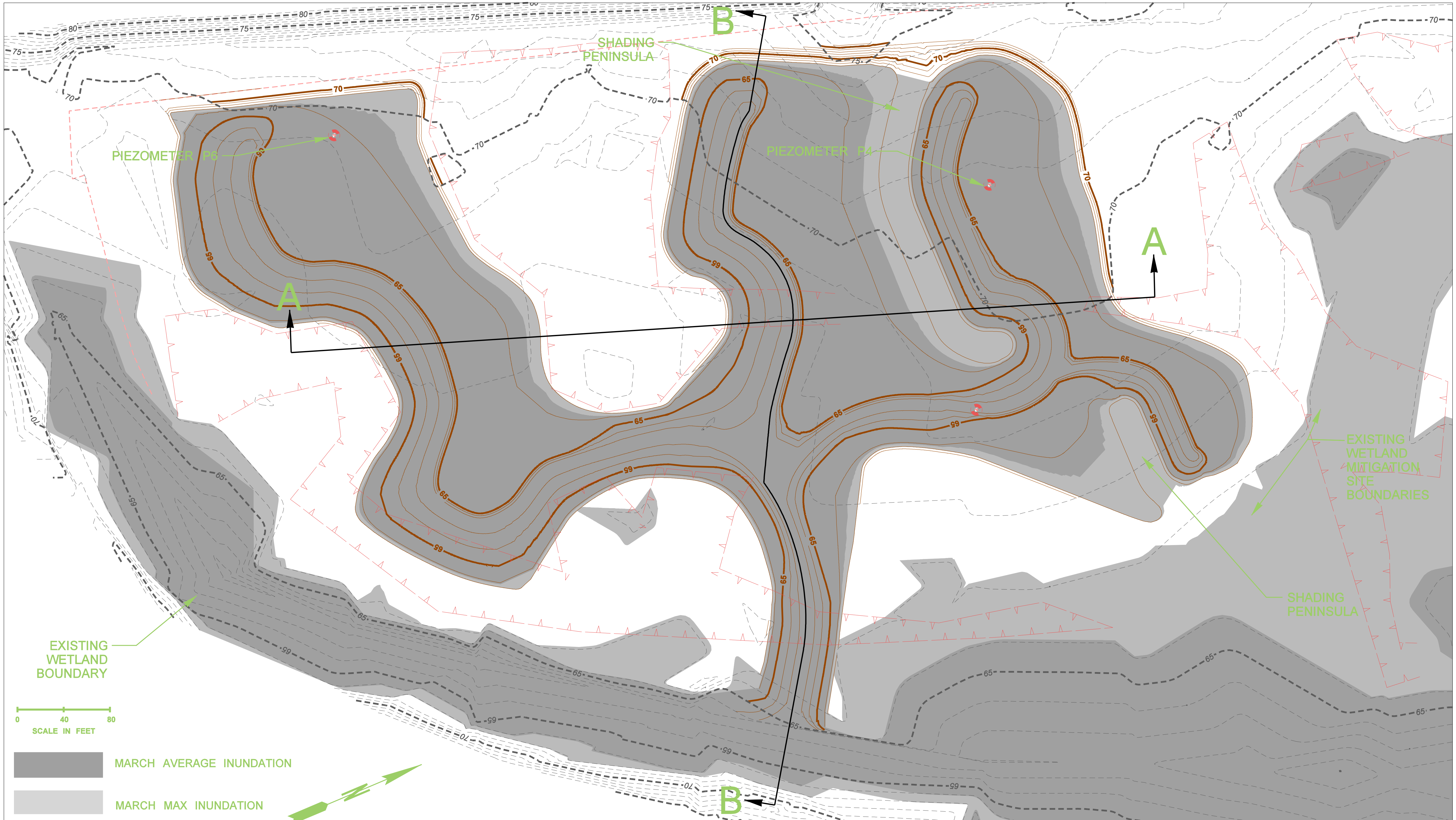

Figure 4-18
Floodplain Storage March Inundation
SR 167 8th to 277th Southbound HOT Lane Project
Final Type A Hydraulic Report
R·W·BECK

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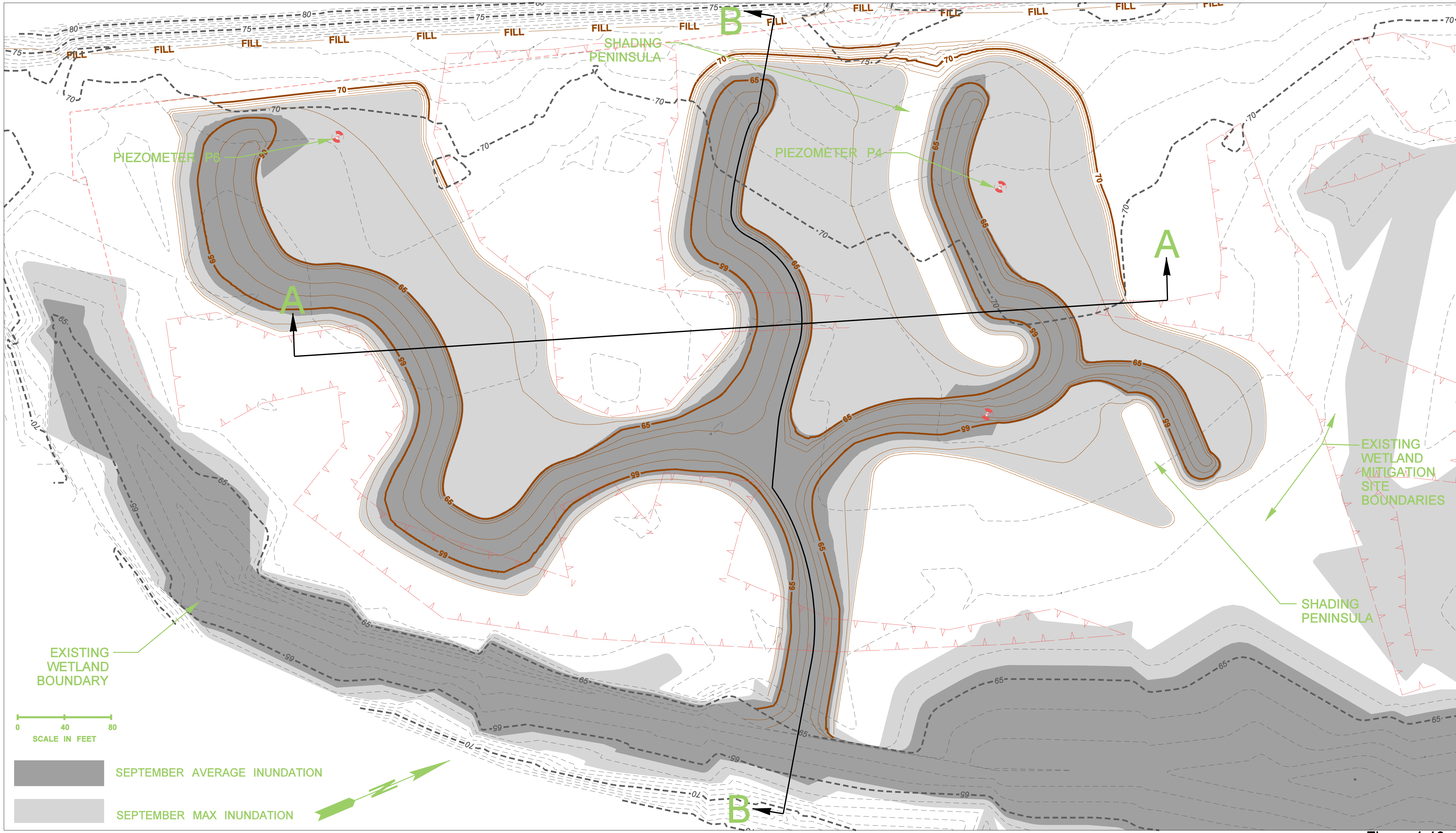

Figure 4-19
Floodplain Storage September Inundation
SR 167 8th to 277th Southbound HOT Lane Project
Final Type A Hydraulic Report
R·W·BECK

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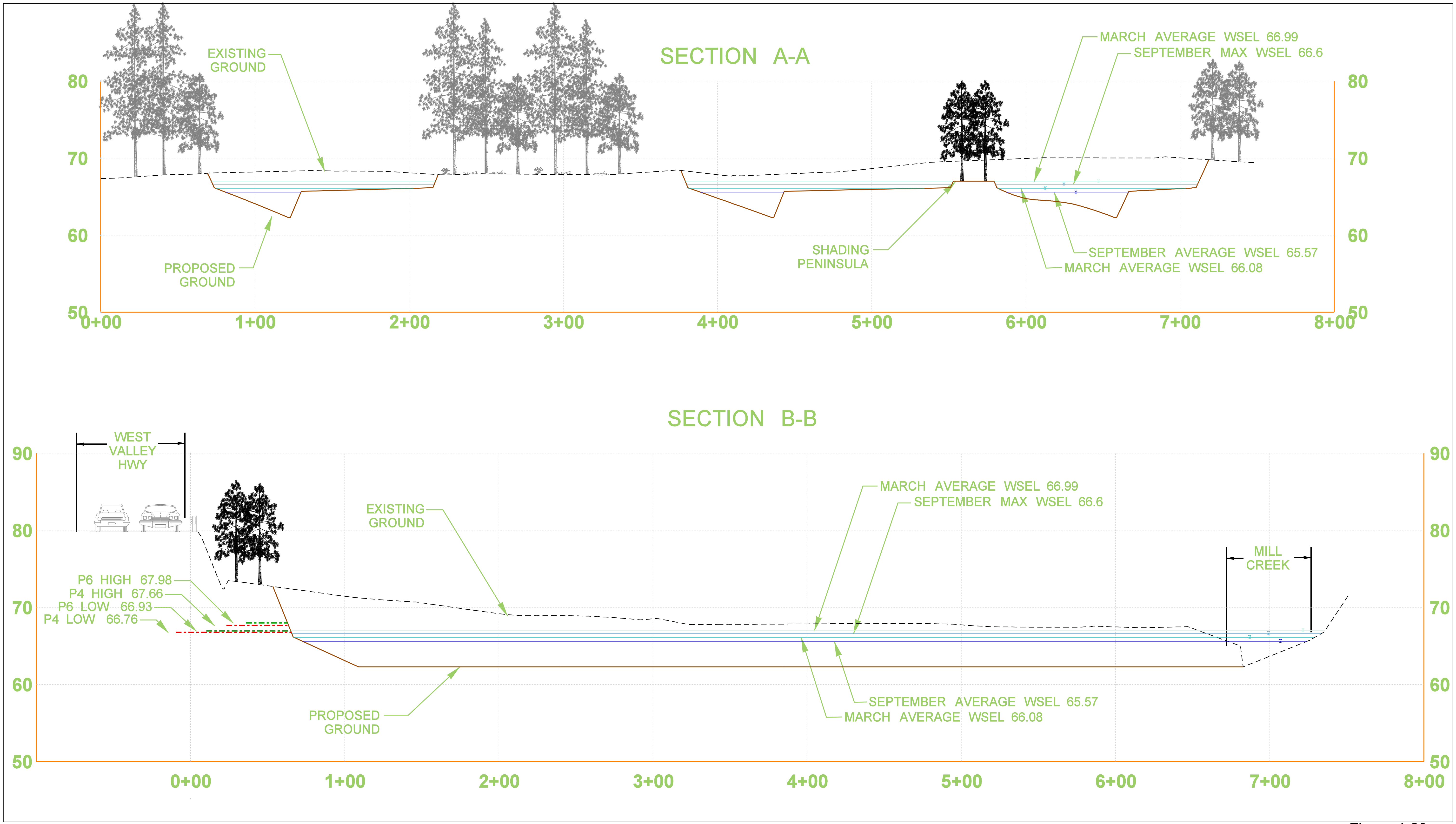

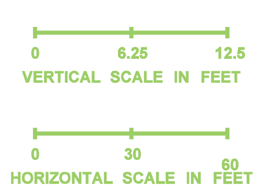

Figure 4-20
Floodplain Storage Cross Sections
SR 167 8th to 277th Southbound HOT Lane Project
Final Type A Hydraulic Report

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The Stage 3 flow control ponds constructed at the SR 18 interchange and at the 15th Avenue NW interchange are within the Stage 4 project limits. The Hydraulic Report Supplement #3 indicates that the stormwater facilities were sized to be in accordance with WSDOT's 2004 HRM. No new pavement is proposed within the SR 18 pond basin and so this pond is not affected. The existing Stage 2 and 3 facilities that will have additional impervious area draining to them as a result of the project were analyzed and are discussed below.

The following facilities constructed as part of Stage 3 are shown in Appendix A.

Table 4-5
Existing Stage 3 Detention Facilities

Stage 3 Facility ID	Approximate Location
Pond-M1-3	Northeast corner of SR167 and SR18 interchange
Pond-M12-3	Northwest corner of SR167 and 15th St NW interchange

Analysis of Existing Stage 2 Facilities

Because none of the existing detention ponds are to be used to detain additional impervious area resulting from the Stage 4 project (flow control for Stage 4 is being achieved using the floodplain storage), the ponds within the project paving limits were analyzed to ensure that the flow control objectives of the existing detention ponds will still be met with the additional impervious area draining to them.

The existing Stage 2 ponds were analyzed using a methodology provided to WSDOT by Ecology (2007). The guidance suggests that the ponds be analyzed to determine if modifications are necessary to allow the ponds to detain the Stage 2 added impervious area for half of the 2-year peak flow event, the 10-year peak flow event, and the 100-year peak flow event. However, since the existing ponds are not being used for Stage 4 detention, the runoff from the added Stage 4 impervious area was modeled as flow that passes through the facilities without being detained. This was accomplished by including the Stage 4 added impervious area to the "Pre-Project" condition and routing it through the ponds using the StormShed computer program (see Table 4-6). Including the proposed Stage 4 added impervious area to the "Pre-Project" condition ensures that the model does not reflect the need for modifications to the existing pond to detain runoff from the Stage 4 areas. Note that "existing impervious" in Table 4-6 refers to the existing impervious area prior to the Stage 2 project.

Table 4-6
Land Use for Existing Stage 2 Ponds Analyses

Facility	Original Land Use for Sizing Stage 2 Facilities				Stage 4 New Impervious (acres)	Original Land Use for Sizing Stage 2 Facilities Adjusted to Reflect the Stage 4 Impervious Area			
	Pre-Project		Post Project			Pre-Project		Post Project	
	Impervious (Acres)	Till Grass (Acres)	Impervious (Acres)	Till Grass (Acres)		Impervious (Acres)	Till Grass (Acres)	Impervious (Acres)	Till Grass (Acres)
M3-2	1.03	0.28	1.31	0	0.2	1.23	0.28	1.51	0
M5-2	1.03	0.51	1.54	0	0.08	1.11	0.51	1.62	0

As can be seen in Table 4-6, the analysis showed that Pond M3-2 had increases in outflow and does not meet the criteria. To achieve the criteria, the lower circular orifice on the flow control structure of Pond-M3-2 will need to be modified to accommodate the additional flow from the Stage 4 pavement. The current 5-inch-diameter lower orifice will need to be restricted to 4.4 inches. The modeling also showed no modifications to Pond-M5-2 will be needed. The results of the Stage 2 existing pond analysis are shown in Table 4-7. The detailed modeling input and output are included in Appendix E.

Table 4-7
Stage 2 Ponds Analysis Results

Return Period (Years)	Pre-Project Peak Flow (cfs)	Post-Project Peak Flow (cfs)	Target Peak Flow (cfs)[2]	Pond Outflow (cfs)
Pond M3-2[1]				
2	0.65	0.71	0.33	0.32
10	0.99	1.06	0.99	0.36
25	1.18	1.25	0.65	0.40
100	1.39	1.46	1.39	0.43
Pond M5-2				
2	0.65	0.76	0.32	0.28
10	1.01	1.13	1.01	0.36
25	1.24	1.33	0.65	0.31
100	1.44	1.56	1.44	0.45

1. Pond M3-2 lowest orifice diameter resized to 4.4-inch-diameter.
2. Target flow for the 25-year event is the 2-year Pre-Project flow.

The City of Auburn also requires that the 25-year post-project peak flow match the 2-year pre-project peak flow. Both ponds meet the City of Auburn's standards as shown in Table 4-7.

Analysis of Existing Stage 3 Facility

Ponds for Stage 3 were designed to the same standard as in the current HRM. Therefore, Pond M12-3 was modeled with the additional impervious area from the project using MGS Flood for duration control. The pre-project and post-project drainage areas for Pond M12-3 are shown in Table 4-8. The modeling results, included in Appendix E, indicate that no modifications to Pond M12-3 are required. Note also that Pond M12-3 may have extra capacity depending on future criteria that is used.

Table 4-8

Land Use for Existing Stage 3 Ponds Analyses

Facility	Origingal Land Use for Sizing Stage 3 Facilities				Stage 4 New Impervious (Acres)	Original Land Use for Sizing Stage 3 Facility Adjusted to Reflect the Stage 4 Impervious Area			
	Pre-Project		Post Project			Pre-Project		Post Project	
	Impervious (Acres)	Till Grass (Acres)	Impervious (Acres)	Till Grass (Acres)		Impervious (Acres)	Till Grass (Acres)	Impervious (Acres)	Till Grass (Acres)
M12-3	2.25	0.26	2.51	0	0.54	2.79	0.26	3.05	0

No new impervious area that would drain to Pond M1-3 is proposed. Therefore, this pond was not analyzed.

4.1.2 Detention

Conventional detention is proposed for the White River basin portion of the project. While floodplain storage is likely feasible in this area, gaining agency approval in the White River basin may be more difficult. Compared with the Green River basin, the White River basin has fewer factors, such as adjacent wetlands, that would limit the use of conventional detention.

4.1.2.1 Flow Control Criteria

The amount of storage volume required depends on several key criteria, including the flow control standard and the pre-project conditions, which form the basis for the target flows. The WSDOT *Highway Runoff Manual* (HRM) requires that a flow-duration standard be used. The flow-duration standard requires that the discharge durations from the pond, for the range of flows from half of the two-year peak flow to the full 50-year peak flow, match pre-project flow durations for the same flow range. Flow-duration standards have been developed to prevent increases in the stream channel erosion rates above the rates that are characteristic of natural stream conditions.

According to the HRM, the runoff duration curve from the proposed projects needs to match the duration curve of the pre-project condition. However, the Washington State Department of Ecology (Ecology) has sometimes required WSDOT projects to assume "forested" pre-developed conditions, resulting in ponds that are approximately 55 percent larger. On some recent projects, WSDOT has decided to assume forested pre-developed conditions based on the assumption that this will become a requirement under the next National Pollutant Discharge Elimination System (NPDES) permit and to avoid potential delays in permitting when Ecology approvals are necessary.

For this project, pre-developed conditions were assumed to be forested for the extent of the proposed new roadway pavement in the White River basin. In addition, the proposed detention pond footprints were assumed to be forested under the pre-developed conditions. The detention ponds were sized to meet the flow-duration standard. Flow control was only applied to the new impervious surfaces and the converted pervious surfaces because the new impervious surface will not add 50 percent or more to the existing impervious surfaces within the project limits.

4.1.2.2 Detention Siting

As discussed previously, the flat terrain and low profile roadway, as well as adjacent wetlands and stream corridors, make it difficult to site detention facilities. As a result, detention ponds were located where space, preferably within the right-of-way, was available. The proposed detention ponds are shown in Appendix M. Because it was not feasible to route all of the new impervious surface to proposed detention sites, some existing impervious surface was routed to the ponds and detained (i.e., equivalent area option, 2008 HRM Section 4-3.6.1). The existing plus new impervious area routed to and detained at the ponds is equivalent to the total new impervious area added by the projects, thus offsetting the impact of the new impervious surface.

4.1.2.3 Hydrologic and Hydraulic Analysis

The detention ponds were sized using MGS Flood in order to meet the performance guidelines, as discussed previously in this section and in the 2008 HRM. A summary of the detention pond characteristics is provided in Table 4-9. The detailed MGS Flood reports for the pond sizing are included in Appendix E. The pond layouts are shown in Appendix M. Note that only the active storage volume is used in the models. The 100-year water surface elevation was used as the design water level.

4.1.2.4 Detention Design

High groundwater, flat terrain, and low-profile roadway sections all limit the depth at which conventional detention facilities can operate, resulting in relatively shallow ponds spread over large areas to create the required volume.

The pond site plans are shown in Appendix M. Due to the relatively high groundwater elevation in the project area, ponds were generally created by adding berms to contain the required storage volume. Excavating was avoided as much as possible in order to reduce the extent of dewatering that may be required. The addition of a pond liner would increase the cost and difficulty of construction, due to the cost of the liner itself and to the additional dewatering that would be required to install the liner.

Pond W1-4 was laid out with 3 horizontal to 1 vertical side slopes. However, due to site constraints at Pond W3-4, a portion of the berm embankment is set at a 2 horizontal to 1 vertical side slope. The detention ponds are immediately adjacent to the roadway and make use of the roadway embankment to create one side of the pond in order to reduce the total footprint. Note that it was requested that the roadway embankment that serves as one side of Pond W3-4 be set at a 3 horizontal to 1 vertical side slope in order to provide the needed volume at that pond.

No new impervious surface was proposed for TDA W2 under the current design. Therefore, no detention is required in this TDA.

Table 4-9
Detention Pond Sizing

		Pond W1-4	Pond W3-4	Total Area Served
Stage 4 Roadway Drainage Area Served	(acres)	0.30	2.96	3.26
Pond Bottom Elevation	(feet)	81.26	74	
Piezo Reading		76.77	68	
100-Year Pond Water Surface Elev.	(feet)	82.77	80.5	
Top of Berm Elevation	(feet)	84.0	82.25	
Berm Top Width	(feet)	6	6	
Berm Side Slopes	(H:V)	3:1	2:1	
Pond Wetted Area	(acres)	0.42	0.54	
Pond Access Road Draining to Pond	(acres)	0.03	0.03	
Pond Access Road Bypassing Pond	(acres)	0.05	0.08	
Unmitigated Grass Area to Pond	(acres)	0.14	0.34	
Total Impervious Area Draining to Pond[1]	(acres)	0.75	3.53	
Pond Volume Provided	(acre-feet)	0.41	1.60	

1. This includes the roadway drainage area served, pond wetted area, and the pond access draining to the pond.

4.2 Runoff Treatment BMPs

Runoff treatment BMPs are divided into two main types: flow rate-based and volume-based. Both types of BMPs are designed to treat 91 percent of the mean annual runoff volume. Flow rate-based BMPs can either be upstream or downstream of detention facilities. Those BMPs upstream of detention facilities can be designed as on-line or off-line systems. Compost-amended vegetated filter strips (CAVFSs) and media filter drains (MFDs) are examples of flow rate-based runoff treatment BMPs. Volume-based runoff treatment BMPs provide a dead storage that treats 91 percent of the mean annual runoff. This volume is greater than or equal to *91 percent of all the modeled daily inflow volumes* to the pond based on the extended time series. Sizing for all of these BMPs requires use of an approved continuous hydrologic simulation model based on HSPF. For this project, MGS Flood (version 3.12) was used to model flows and adequately size BMPs to comply with the requirements in the 2008 HRM. For all TDAs the following model variables were used:

- Puget East 40 precipitation time series (based on precipitation station 960040).
- Puget East 40 evaporation time series (based on evaporation station 961040).
- Till soils unless geotechnical data suggested otherwise.
- Extended time series selected. (This time series relies upon combining and scaling records from distant stations to create a simulation record of over 90 years.) (MGS Software 2005)

4.2.1 Compost-Amended Vegetated Filter Strips

CAVFSs are flow rate-based BMPs sized for the water quality flow rate (i.e., the flow rate that captures 91 percent of the mean total annual runoff volume). Treatment is via infiltration (the primary treatment mechanism) and filtration by the amended soil medium. The following variables are utilized by MGS Flood to calculate the volume of runoff treated by a CAVFS:

Site Characteristics

- Cross slope of CAVFS
- Underlying infiltration rate
- Precipitation and evaporation
- Area draining to CAVFS

CAVFS Parameters

- Depth and width of compost-amended soils and gravel strip at pavement edge
- Saturated hydraulic conductivity and porosity of the compost-amended soils
- Saturated hydraulic conductivity and porosity of gravel ballast at edge of pavement

This project relied extensively on CAVFS to reduce the amount of dissolved metals from the existing and proposed highway surfaces reaching local receiving waterbodies. Due to the highly variable soils used in the original highway fill, the design team for the Stage 4/5 projects coordinated with WSDOT's Northwest Region Materials Laboratory to develop a conservative design infiltration rate that could be used to represent the soils throughout the project corridor. Based on an analysis of 26 near-surface soil samples, an infiltration rate of 0.2 inches per hour was recommended for design purposes (see October 5, 2007 email from Nabil Dbaibo in Appendix E4). Based on MGS Flood modeling, the 0.2 inches per hour infiltration rate equates to 90 percent of the mean annual runoff infiltrating into the soil underlying the CAVFSs.

Table 4-10 outlines the values of the input parameters that were used for CAVFS modeling on this project.

Table 4-10
Input Parameters for CAVFS Modeling

Parameter	Value	Units
Site Specific Information		
Width of pavement	varies	feet
Length of pavement	varies	feet
Pavement area	varies	ac
Length of CAVFS	varies (same as length of pavement)	feet
Side slope	4:1 or less steep	H:V
Constants		
Amended soil depth	1.0	feet
Amended soil porosity	20	percent by volume
Amended soil saturated hydraulic conductivity	2.0	feet/day
Underlying soil infiltration rate	0.02	inch/hr
Width of gravel at pavement edge	2.0	feet
Gravel porosity	30	percent by volume
Gravel saturated hydraulic conductivity	4.0	feet/day
Precipitation data set used	Puget East 40	
"Include Precipitation and Evaporation of CAVFS" button	activated	

To size each CAVFS the following basic steps were followed:

1. Basin areas were calculated as the adjacent pavement and pervious area draining to the CAVFS on the highway embankment.
2. Under the network tab, a CAVFS link was defined (in the post-development scenario) with the parameters shown in Table 4-10. The CAVFS length was equal to the pavement length. A preliminary CAVFS width was selected.
3. The extended time series was routed through the CAVFS and the MGS Flood report was created.
4. In the "Post-Development Link Statistics" section of the report, the CAVFS Treatment Statistics were checked to confirm that at least 91 percent of the total runoff volume was either filtered or infiltrated by the CAVFS. If this standard was not met, the CAVFS width was increased and time series routed again. This procedure was followed until the 91 percent treatment standard was met for each CAVFS.

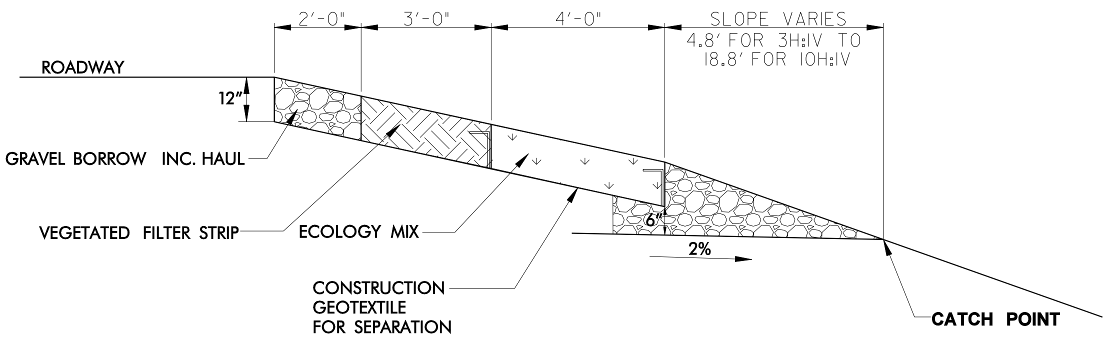
4.2.2 Media Filter Drains

Media Filter Drains (MFDs) (formerly referred to as ecology embankments) are flow rate-based BMPs sized for the water quality flow rate (i.e., the flow rate that captures 91 percent of the mean total annual runoff volume). The water quality flow rate from the contributing highway area is used to calculate the required width of the media filter drain. Typically, the media filter drain is constructed along the pavement edge and has the same length as the contributory pavement area. The design infiltration rate is set at 14 inches per hour, which is the approved design rate when using a prescribed medium (Ecology Mix) within the media filter drain.

According to the 2008 HRM, "in almost every case, the calculated width of the media filter drain does not exceed 1.0 foot." This was based on modeling exercises conducted by WSDOT over the range of typical highway pavement widths. However, due to constructability and maintenance issues WSDOT specifies widths greater than 1.0 foot for media filter drains. These minimum widths are summarized in Table RT.07.1 in the 2008 HRM. Based on this table, for pavement areas exceeding 35 feet in width the media filter drain shall consist of an Ecology Mix treatment bed 4 feet in width, preceded by a 3-foot-wide vegetated filter strip (see Appendix E4).

Existing media filter drains within the Mill Creek basin are assumed not to require any modification to treat the new pavement areas since they are already sized at least 4 feet wide. This width is adequate to treat any pavement width exceeding 35 feet. As-builts of the Stage 3 media filter drain facilities were not available for review at the time of this report.

Staff from WSDOT's HQ and UCO Hydraulics Offices recommended deleting underdrains from the media filter drains where these BMPs are proposed for the Stage 4 project. A suggested detail of the media filter drain without underdrain was provided by WSDOT and is shown in Figure 4-21. The key difference between the two different media filter drain configurations is that in place of the underdrain an aggregate drainage layer is to be included which extends beyond the downslope edge of the ecology mix media. While preferable in terms of installation and maintenance, the widths of media filter drains without underdrains were significantly wider than those with underdrains. On this project, media filter drains are proposed due to their relatively narrow width to avoid impacting adjacent wetlands. Therefore, all media filter drains are designed with underdrains.



Source: WSDOT

Figure 4-21. Media Filter Drain Without Underdrain

4.2.3 Constructed Stormwater Treatment Wetland Cell

Constructed stormwater treatment wetlands are volume-based runoff treatment BMPs. This type of facility is proposed as part of detention pond W3-4 within the White River basin. The rationale for using a constructed stormwater treatment wetland at this specific location is based on the following considerations:

- The detention pond already required a conveyance be installed to collect water from a sizeable highway area.
- There is state right-of-way available at Pond W3-4 to add the constructed wetland without excavating into areas of shallow groundwater.
- Roadside BMPs such as media filter drains and CAVFS are not feasible in this area because storm drains pick up runoff directly from the highway pavement so that it can be conveyed to Pond W3-4 or the embankment is not suitable.

The constructed wetland and forebay at Pond-W3-4 was sized using the procedure in the HRM. The first step in the sizing procedure is to determine the surface area of the constructed wetland and forebay. This is done by dividing wet pond volume required to treat 91 percent of the flow for the post developed conditions by a water depth of 3 feet. The wet pond volume was calculated to be 15,250 cubic feet using MGSFlood. The resulting necessary surface area for the constructed wetland and forebay was 5084 square feet.

After calculating the surface area, the next step is to divide the volume between the constructed wetland and the forebay. The design criterion requires that the forebay contain approximately 25 percent to 35 percent of the total constructed wetland volume. The general layout dimensions of the constructed wetland and forebay follow the guidance provided on Figure RT.13.1 in the HRM and the site constraints. The general layout was used to calculate the constructed wetland and forebay volume. From this layout it was determined that the forebay volume would meet the requirement, occupying 34.5 percent of the total volume.

The final step was to determine the water depth distribution in the wetland cell. The constructed wetland is shown in the W3-4 Detention Pond Site Plan included in Appendix K.

4.3 Gutter Design

An enclosed storm drain system is required in order to route flow from the roadway into Pond W3-4 for water quality treatment and detention. A plan of the drainage system and contributing subbasins is included in Appendix C. The inlet spacing for the gutter design was determined using WSDOT's *Side Flow Calculations* spreadsheet. This spreadsheet was used because the longitudinal slope of SR 167 in the vicinity of the Pond W3-4 is less than 2 percent. The completed spreadsheet for the gutter design is included in Appendix E.

Several "gutter runs" are included in the spreadsheet analysis running from a crest point to a sag or to the downstream end of the system. The first three gutter runs are for the main system and the fourth gutter run is for the branch bringing flow to the pond from the Ellingson overpass. The area where main system is proposed is fairly flat with some minor undulations.

The gutter run extends from the first crest at Station LM' 396+87 to the first sag at Station LM' 395+73. The second gutter run goes from the second crest at Station 400+68 to the second sag at Station 397+13. There is a sag at Station 401+46 between crests at Stations 400+69 and 402+79. The sag designs are addressed in Section 4.4. The third gutter run is from the crest at Station LM' 402+79 to the low point in the system at Station LM' 406+28 (ERS' 19+04). The fourth gutter run is the branch system from the Ellingson overpass starting at the crest at Station LM' 412+18.00 back to Station LM' 404+56, where the system crosses over the onramp and connects into the main system.

Profiles along the gutter line were provided by the roadway designer for use in the gutter analysis. The profile provided is very flat in a number of places. Because there may be some inaccuracies in the 3-D model used to generate the profiles, it is recommend that extra care be taken during the construction to ensure that there is a catch basin placement at all low spots.

The spreadsheet shows that "Zd" (width of flow in the gutter) is less than the shoulder width for the 10-year storm. In addition, the velocities are less than 3 fps. Also, the bypassed flow at the downstream end of the system (Station 405+57) is less than 0.1 cfs.

4.4 Sag Design

Three sag points were identified as part of the gutter design located at Station 395+73, 397+13 and 401+46. The sag at 395+73 is located between the start of curb at Station 394+28 and the inlet at Station 396+42. The sag at 397+13 is located between the crest at Station 396+87 and the inlet at Station 398+15. The third sag at 401+46 is located between two crests at Station 400+69 and 402+79. The WSDOT Sag worksheet was used to determine if flanker inlets are required, and if so, where to locate them. The Sag Analysis spreadsheets are included in Appendix E. The results of the analysis indicated that flanker inlets are not required for any of the sags.

4.5 Enclosed Drainage Design

The design for the enclosed drainage system is included in Appendix C (plan) and also in Appendix J (profiles). The analysis used to size the system is included in Appendix E. WSDOT's Storm Sewer Design spreadsheet was used to check capacity, velocities, and cover for the pipe system. The drainage system discharges into Pond W3-4 and will be under backwater conditions during design flow events. Therefore, the King County Backwater model (KCBW) was used as a supplemental check of the system design WSDOT's spreadsheet was not used to check velocities or check cover. Cover was checked manually. The 100-year pond water surface elevation in Pond W3-4 (elevation 80.5) was used as the tailwater elevation for the backwater analysis. The pipes were sized based on the 25-year flow event. The results of the KCBW analysis is also included in Appendix E. The appendix also contains the profile design sheets with notations on each pipe referencing which KCBW model was used
Note that this design requires curb from Station LM' 394+28 to ERS' 19+04 as well as from Station LM' 477+38 (41.8 LT) to LM' 404+55 (59.3 LT) in order to collect the runoff required to be routed into Pond W3-4.

Because the roadway being served by the proposed drainage system is so flat, there are minor undulations in the roadway. As a result, the drainage basin breaks were not always defined at the inlet location.

Note that this system was design based on the proposed roadway surface at the time of the report. It is expected that the proposed roadway surface will be updated with the recent survey. Once the roadway surface is updated, this design should be rechecked and adjusted as needed.

4.6 Culvert Design

WSDOT met with the Washington State Department of Fish and Wildlife (WDFW) in July 2007, at which time it was determined by both agencies that no culverts would need to be replaced as part of the Stage 4 project. However, improvements are proposed at both SI 65/95 (Jovita Creek culverts) and SI 73 to improve fish passage by backwatering flow through the lengths of the culverts during all flow events. These improvements are discussed below.

4.6.1 Culvert SI 73

According to WDFW, insufficient flow depth occurs at the upstream end of the culvert for the low fish passage flow (0 cfs). In addition, there was a fairly significant drop between the culvert outlet and the UTWR. WDFW requires that there be a depth of at least 0.8 feet during the low fish passage flow. In order to obtain sufficient depth at the upstream of the culvert and provide fish passage from the UTWR to the culvert outlet, log v-weirs are proposed downstream as well as a metal weir plate at the end of the culvert to create backwater depth through the culvert. The proposed design is included in Appendix L.

Due to the short length of channel between the downstream end of the culvert and UTWR, there was not enough room to place a sufficient number of weir structures to backwater the low flow to a depth of 0.8 feet at the upstream end of the culvert. As a result a compromise arrangement was negotiated with WDFW that provides an upstream depth of about 6 inches during a zero-flow condition.

Because the downstream channel length is short, the weirs are proposed to be placed at about 10 feet on center. There was a concern that since the weirs were spaced fairly closely that scour could be an issue. The concentrated flow from an upstream weir could result in scour around a downstream weir if the weirs are spaced closely. Scour analysis for the 100-year flow show that the estimated scour depth may range from 1 to 3 feet. The scour analysis is documented in Appendix E. As a result, the weirs and grade control structures were extended to 3 feet below ground. It is also noted that field investigations indicated that the ordinary high water level for the downstream UTWR extends up to the downstream end of the culvert. This suggests that the weirs will be under water during peak flow events with high scour potential. With the weirs submerged, the scour potential during these events would be reduced because the highwater would help dissipate the flow energy.

The addition of a weir plate at the downstream end of the culvert affects the conveyance capacity of the culvert. To check the conveyance capacity of the culvert with the proposed modifications, an HEC-RAS model was developed. A rating curve based on weir flow over the weir plate was used as the tailwater condition. The analysis showed that the 100-year peak water surface elevation through the culvert is below the crown of the pipe and therefore there is adequate capacity under proposed conditions. The HEC-RAS analysis is included in Appendix E.

4.6.2 Culvert SI 65/95 (Jovita Creek)

The Jovita Creek culvert crossing consists of twin 84-inch-diameter culverts with the northern culvert set higher than the southern culvert. In addition, the existing southern culvert contains concrete baffles. These concrete baffles are 2 feet thick in the direction of flow, which may impede fish passage. As a result, it is proposed to replace the existing concrete baffles with steel corner baffles. The proposed design is included in Appendix L and the baffle design calculations are included in Appendix E.

4.7 Ditch Design

Ditches were added between the southbound on and off ramps at Ellingson and mainline SR 167. Refer to Sheets DR7 and DR8 in Appendix C. The runoff calculations and the ditch sizing analysis are included in Appendix E.

In addition to the new ditches, the areas where the median ditches will receive additional runoff under the proposed conditions were evaluated. The 10-year runoff for these ditches was calculated based on the Rational Method. A summary of the flows are shown in Table 4-11. The flows listed represent the total flow at the receiving median drain at the downstream end of the noted ditch reach. This represents the maximum possible flow in the ditch. The actual flow in the ditch at any location is proportional to the tributary area at that location and therefore is a lesser distance upstream from the median drain. The cross-sectional area of the contributing ditches was sampled and the minimum ditch area was used to determine the ditch capacity. In all but two locations, the minimum ditch capacity was sufficient to pass the maximum total 10-year ditch flow with 0.5 foot of freeboard.

In two locations (LM’ 481+21 and LM’ 591+76), the minimum ditch capacity was not sufficient to pass the maximum total ditch flow. As noted above, the total ditch flow is the total flow at the downstream end of the reach and over-estimates the actual flow at the sampled locations. Therefore, at these two locations, a more accurate assessment of the flow at the sampled cross section was determined and compared to the minimum ditch capacity. In both of these cases, the ditch was able to pass the 10-year flow at that location with 0.5 foot of freeboard. These ditches were also checked at a second sampled location, and both ditches could pass the conservative total ditch 10-year flow with 0.5 foot of freeboard at these locations. Therefore, it was concluded that the median ditches that will receive additional flow as a result of the project will have sufficient capacity to meet the requirements of the WSDOT *Hydraulics Manual*.

It was also noted that at two locations (LM’ 498+99 and LM’ 499+35), the proposed surface was below the existing surface such that the proposed embankment did not “catch” the existing surface. This is because the proposed roadway surface was developed based on a previous version of the existing conditions base map and has not yet been updated with the new survey information. As a result, an approximation of the ditch area was made, assuming that the edge of the roadway cross section would be raised until the edge of roadway matched. The approximated median ditches had sufficient capacity to pass the maximum total 10-year flow with 0.5 foot of freeboard. However these locations should be re-evaluated once the roadway design is updated.

Table 4-11
Median Ditch Capacity Check

Median Ditch Begin Station	Median Ditch End Station	Median Drain Station	Existing Drainage Area: Impervious Area (acres)	Existing Drainage Area: Pervious Area (acres)	10-year Existing Flow (cfs)	Proposed Drainage Area: Impervious Area (acres)	Proposed Drainage Area: Pervious Area (acres)	10-year Proposed Flow (cfs)[1]	Minimum Ditch Area (SF)	Minimum Ditch Capacity (cfs)
LM' 591+21	LM' 597+73	LM' 597+00	0.66	0.68	1.85	0.91	0.44	2.25	5.5[2]	6.32[2]
LM' 582+34	LM' 591+21	LM' 589+02	0.79	0.92	2.27	1.12	0.60	2.80	10.0	14.7
LM' 575+07	LM' 582+34	LM' 580+97[5]	1.09	0.77	-	1.02	0.50	-	-	-
LM' 499+96	LM' 507+55	LM' 501+50[5]	0.65	1.08	-	0.45	0.68	-	-	-
LM' 498+93	LM' 499+96	LM' 498+99	0.15	0.15	0.42	0.21	0.09	0.51	[3]	[3]
LM' 491+63	LM' 498+93	LM' 493+98	1.06	0.96	2.89	1.31	0.70	3.27	10.5	16.3
LM' 486+02	LM' 491+63	LM' 489+01	0.63	1.01	1.98	0.90	0.73	2.40	7.2	9.2
LM' 479+77	LM' 486+02	LM' 483+49	0.92	1.14	2.69	1.22	0.82	3.15	6.3[4]	5.94[4]
LM' 473+81	LM' 479+77	LM' 478+46	0.54	1.10	1.84	0.77	0.79	2.15	17.4	31.30
LM' 454+03	LM' 462+07	LM' 460+02	0.17	1.16	1.07	0.36	0.80	1.14	5.0	9.27

[1] This is the total flow to the median drain and is typically more than the flow at the location of the minimum ditch area which is often located in an upstream tributary.

[2] There is a section of ditch in the vicinity of LM' 591+76 with less cross sectional area, however, it has sufficient capacity for the portion of flow that is tributary to it.

[3] The area and capacity shown are an estimate based on anticipated modifications to the roadway design. The current roadway design conflicts with the revised survey in this location. The approximate capacity is 2.46 cfs.

[4] There is a section of ditch in the vicinity of LM' 481+21 with less cross sectional area, however, it has sufficient capacity for the portion of flow that is tributary to it.

[5] These median drains are in the same vicinity of the other drains that receive additional runoff under proposed conditions, however the tributary areas to these drains actually decreases under proposed conditions. Therefore, flow was not calculated for these structures.

4.8 Special Stream Design

No special stream design was required for this project.

4.9 Floodplain Mitigation

4.9.1 Green River

The Federal Emergency Management Agency (FEMA) is in the process of updating the floodplain maps to reflect the fact that the Green River levees are not certified and FEMA has prepared preliminary maps. The FEMA maps are not official yet and are expected to become effective September 2008. However, some jurisdictions are already using the preliminary maps for floodplain regulations and WSDOT has decided to use the preliminary maps to see if the SR 167 project will cause fill to be placed in the floodplain. WSDOT compared the new floodplain delineations and elevations with the fill associated with the Stage 4 project and found that there is no fill proposed in the floodplain. As a result, no compensatory storage is required for this project.

4.9.2 White River

The White River floodplain does not extend such that it impacts the SR 167 project corridor. Because the proposed projects do not impact the White River floodplain, no compensatory storage is required for this project.

4.10 Bridge Scour Evaluations

No bridges are included in this project. Therefore, no bridge scour evaluation was conducted.

4.11 Channel Changes

Channel changes were required downstream of SI 73 to facilitate fish passage. This design is discussed in Section 4.6.1.

4.12 Median Drains

There are approximately 22 median drains that need to be adjusted as a result of the addition of fill into the median. The fill into the median will move the location of the bottom of the ditch. As a result, either the median drain rim will need to be adjusted or the whole drain will need to be relocated in order to ensure it will collect the ditch drainage.

In addition, runoff from the new proposed pavement flows to eight median drains. Drainage areas to each of these eight inlets were delineated and runoff for the proposed conditions was calculated using the rational method. There are two other median drains in this vicinity, at LM' 501+50 and LM' 580+97, but no new impervious surface drains to these. The 25-year flow to each of these drains is shown in Table 4-12. The calculations used to determine the runoff are included in Appendix E. The capacity of the median drains will be evaluated when the survey information becomes available.

Table 4-12
Median Drain Hydrology

Median Drain Station	Existing Drainage Area (acres)		25-year Existing Flow (cfs)	Proposed Drainage Area (acres)		25-year Proposed Flow (cfs)
	Impervious	Pervious		Impervious	Pervious	
LM' 597+00	0.66	0.68	2.22	0.91	0.44	2.70
LM' 589+02	0.79	0.92	2.74	1.12	0.60	3.37
LM' 498+99	0.15	0.15	0.50	0.21	0.09	0.62
LM' 493+98	1.06	0.96	3.48	1.31	0.70	3.94
LM' 489+01	0.63	1.01	2.39	0.90	0.73	2.89
LM' 483+49	0.92	1.14	3.24	1.22	0.82	3.79
LM' 478+46	0.54	1.10	2.21	0.77	0.79	2.59
LM' 460+02	0.17	1.16	1.29	0.36	0.80	1.37

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4.13 Bridge Drains

The bridge overpass over SR 18 is proposed to be widened by about 22 feet as part of this project. This will increase the impervious area draining to the existing bridge drains. The capacity of these drains was checked using the WSDOT gutter analysis spreadsheet. The transverse slope and longitudinal grade of the bridge were derived from the WSDOT-provided Digital Terrain Model (DTM). It should be noted that the DTM indicates that the bridge slopes from north to south, where the as-builts indicate that the bridge is crowned at about LM 522+27.44. For this analysis, it was assumed that the DTM, which was developed from recent survey, is correct. In addition, the as-builts indicate existing bridge drains at Stations LM 520+47.44 and LM 522+85.44, where the preliminary design indicates only one bridge drain at Station LM 522+85.44. The DTM also indicates a drainage structure at the end of the bridge (approximately station LM 519+95.51 as well as curb extending down the SR 167 off-ramp to another drainage structure at Station NE 10+22 [17.0' RT]). The DTM does not indicate what type of structures these are.

From the as-builts, the bridge drains appear to be single vaned grates. These were entered into the WSDOT Side Flow Calculations spreadsheet to check the gutter spacing. Under the proposed conditions, assuming that both the bridge drain inlets shown on the as-builts exist and that the structure at the end of the bridge and on the ramp are grate inlets, the bypass flow at the downstream end exceeds 0.1 cfs. If the bridge drain shown on the as-builts (LM 520+45.51) has been paved over, Zd also exceeds the allowable spread, where the allowable spread equals the 8-foot shoulder plus 2 feet. Therefore, it is recommended that the bridge drain at Station LM 520+45.51 be re-established if it has been removed and that the drains at LM 519+95.51 and NE 10+22 be replaced with grate inlets if they are not grate inlets already. This analysis is shown in Appendix E.

4.14 Traffic Analysis Data

The average daily traffic (ADT) count for this project is > 30,000 vehicles.

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Section 5
PERMITS AND ASSOCIATED REPORTS

5.1 Environmental Issues, Fish and Other Endangered Habitat

The direct effects of the SR 167 Project include impacts associated with water quality and quantity, vegetation and habitat modification, fish exclusion and removal, and channel dewatering. Based on the analysis as part of the Biological Assessment (WSDOT 2009) of the potential effects of the SR 167 Project on federally protected species that may occur within the project action area, it was determined that the project may affect, but is not likely to adversely affect the following species:

- Chinook salmon of the Puget Sound Evolutionary Significant Unit (ESU)
- Steelhead of the Puget Sound Distinct Population Segment (DPS)
- Bull trout of the Coastal/Puget Sound DPS

Critical habitat for Chinook salmon of the Puget Sound ESU has been designated within the project action area. Based on the analysis of the potential effects of the SR 167 Project on critical habitat, it was determined that the project may affect, but is not likely to adversely affect critical habitat for Chinook salmon of the Puget Sound ESU. The two other federally protected species have not had critical habitat designated within the project action area; therefore the SR 167 Project will have no effect on designated critical habitat for these species.

No culvert extensions are required for this project. However, two culverts located within the limits of the project corridor were identified for fish passage improvements.

Culvert 65 (WDFW ID 105 R050320a) is on Jovita Creek and provides partial fish passage. The existing baffles in the culvert and upstream sandbag weir make this culvert a partial barrier due to the resulting drop in water surface which is greater than 0.8 feet. The existing concrete baffles are infrequently spaced and are wide making them difficult for fish to jump over. The baffles are proposed to be replaced with metal corner baffles spaced 16.8 feet apart through out the culvert. The proposed baffles will roughen the surface of the culvert and increase the depth of flow downstream of the sandbag weir such that water surface drop across the weir is fish passable.

Culvert 73 is a precast box culvert that appears to be a partial barrier during low flow periods due to lack of depth at the upstream inlet. Repair to this culvert will include installing log weirs in the 50-foot channel between the downstream end of the culvert and the confluence with UTWR. In addition, a weir plate will be bolted on to the end of the wingwalls and apron at the exit of the culvert. The series of downstream weirs will backwater flow through the culvert and provide sufficient flow depth at the upstream of the culvert to provide fish passage.

5.2 Permits/Approvals

Several permits and approvals are required for this project. They include:

- NEPA Documented Categorical Exclusion
- SEPA Determination of Non-Significance
- ESA Section 7 Compliance
- Clean Water Act Section 404 Permit (U.S. Army Corps of Engineers)
- Clean Water Act Section 401 Certification (Washington State Department of Ecology)
- Coastal Zone Management Act Consistency Determination
- National Historic Preservation Act Section 106 Concurrence
- Hydraulic Project Approval
- NPDES General Construction Permit
- Local Flood Hazard Permit
- Local Noise Variance
- Local Critical Areas Compliance

The Coastal Zone Management Consistency Determination and Nationwide Permit 23 (Clean Water Act Section 404 Permit) are included in Appendix N.

5.3 Easement

The proposed drainage improvements are located within WSDOT right-of-way with the exception of the Floodplain Storage site. WSDOT is in negotiations with the City of Auburn, who currently owns this site, to purchase this site to use for stormwater mitigation for the Mill Creek portion of the project.

5.4 Additional Reports or Studies

Several environmental report and studies were prepared for this project, including:

- Wetland Biology Report
- Wetland & Stream Mitigation Report
- Biological Assessment
- Stream Survey Technical Memorandum
- Culvert Inventory Report

- Air Technical Memorandum
- Noise Discipline Report
- Geology/Soils/Geotech Discipline Report
- Environmental Justice Discipline Report
- Visual Quality Report
- Public Services and Utilities Tech Memorandum
- Hazardous Material Technical Memorandum
- Traffic Technical Memorandum
- Ecosystem Report

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Section 6
INSPECTION AND MAINTENANCE SUMMARY

6.1 Maintenance of Existing Drainage Facilities

The existing drainage facilities along the SR 167 corridor were described in Section 2 and consist of drainage ditches, median drains, detention ditches and media filter drains. Discussions were held with maintenance staff from WSDOT and King County. The primary maintenance concerns in the area are associated with beaver activity and sedimentation due to lack of velocity in the creek.

6.2 Proposed Drainage Features to Be Maintained

The following list summarizes the proposed drainage features to be added as a result of the project

- Compost-Amended Vegetated Filter Strips
- Media Filter Drains
- Piped Stormdrain System
- Detention Ponds with Control Structures
- Constructed Wetland

The first four features should be maintained according the WSDOT Highway Runoff Manual Chapter 5. It should be noted that beavers and beaver dams have been seen in the area and could cause backwater issues. Maintenance should be on the look out for beaver dams. The Constructed Wetland should be maintained in a similar manner as a wetpond with the following additions or modifications:

- Maintenance should be scheduled around sensitive wildlife and vegetation seasons.
- Plants may require water, physical support, mulching, weed removal, or replanting during the first three years.
- Nuisance plants should be removed and desirable species should be replanted.
- Sediment accumulations shall be removed from the first cell only.

In addition to the drainage features above, floodplain storage is proposed to provide flow control for the portion of the project in the Mill Creek basin. The floodplain storage is proposed to act as an extension of Mill Creek providing storage to mitigate for the increase in flows resulting from the project. Once the vegetation in the floodplain storage area is established, it should become a natural part of the creek system such that maintenance is not required. Like the Constructed Wetland, plants may require water, physical support, mulching, weed removal, or replanting during the first three years. About 4 feet of storage is included below the approximate ordinary high water mark such that significant sedimentation would need to occur before active storage is affected. In addition, Mill Creek should overtop its west bank in the 10- to 25-year storm event such that flows which will help sediment that will accumulate in the floodplain storage site be transported downstream.

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