Appendix F

Geotechnical

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Department of the Army
Seattle District, US Army Corps of Engineers

Appendix F-1

Geotechnical: Levee Characterization

Puyallup River Basin

Flood Risk Reduction Feasibility Study

Department of the Army
Seattle District, US Army Corps of Engineers

1.0 INTRODUCTION

The levee characterization section is intended to provide guidance on the historical performance, existing conditions, and make an assessment on the performance in future conditions without projects. The existing conditions evaluation stresses maintenance and levee reliability using a probabilistic method. The results of the levee fragility study is calibrated to historical performance and engineering judgment. A reliability analysis was completed to determine the probability of failure of the existing levees as a function of the floodwater elevation. The USACE ETL 1110-2-556, *Risked-Based Analysis on Geotechnical Engineering for Support of Planning Studies*, dated 28 May 1999 (Reference 2), was used as guidance.

The levees of the Puyallup River Basin includes 28 levee segments currently in the Corps' National Levee Database (NLD). This includes twenty-six non-federal levees and two Federally owned and operated levees. The levees will be assessed in geographic groups with similar levee composition, geometry, condition and similar loading and performance histories. Inspection condition ratings are current as of February 2013. Pierce County is participating in the USACE System Wide Improvement Framework (SWIF) which allows all of the levee segments in this study to be in an eligible status regardless of the condition rating.

2.0 GEOLOGIC SETTING

The Geologic Map of Washington – Southwest Quadrant (Walsh, et al, 1987) indicates that the sites are near a contact between Alluvium (Qa), Lahars (Qvl), and Pre-Fraser Glacial sediments (Qap & Qgp). Alluvium is described as sorted combinations of silt, sand, and gravel deposited in streambeds and alluvial fans. The Lahar unit is described as unsorted to poorly sorted, generally unstratified mixtures of cobbles and boulders supported by a matrix of sand or mud. Pre-Fraser Glacial Sediments are described as stratified clay, silt, sand, gravel of fluvial, deltaic, lacustrine and/or estuarine origin, contains mixtures of till and outwash.

The liquefaction potential of the ten boring sites is indicated as "high or moderate to high" on the Liquefaction Susceptibility Map of Pierce County, Washington (Palmer et al, 2004). [Puyallup River Basin Levee Geotechnical Investigation – URS Corp. – January 2012].

3.0 LOWER PUYALLUP

3.1 Introduction

The Lower Puyallup region spans from the mouth of the Puyallup River (RM 0.0) emptying into Commencement Bay to RM 10.3 at the confluence with the White River. The lower river had been straightened and lined with levees on both banks. This region incorporates five NLD levee segments. From downstream to upstream; Puyallup Authorized Left Bank and River Road on the left bank looking downstream and Puyallup Authorized Right Bank, North Levee Road, and Old Cannery on the right bank. The levees in this region are generally larger geometric prisms allowing for the high hydraulic head to load the structure. Due to the vast development behind the flood control structures, these levees protect the highest economic values in the Puyallup GI study area.

Name	Bank	River Mile Location	Authorization	PL84-99 Rating
Puyallup Authorized Left Bank	Left	RM 0.7 – RM 2.8	Federal	Minimally Acceptable
Puyallup Authorized Right Bank	Right	RM 0.7 – RM 2.8	Federal	Minimally Acceptable
North Levee Road	Right	RM 2.8 – RM 7.7	Non-Federal	Minimally Acceptable
River Road	Left	RM 2.8 – RM 7.4	Non-Federal	Minimally Acceptable
Old Cannery	Right	RM 9.7 – RM 10.3	Non-Federal	Minimally Acceptable

Table #: Lower Puyallup River Levees

Figure #: Puyallup Authorized Levee Left Bank

3.2 Geotechnical Properties

The Lower Puyallup River Valley is a broad low-gradient alluvial plain. Historically the river was once a complex area of river channels, wetlands, and thick riparian forests (Entrix 2008). Between 1914 and 1930 the river was altered to its present condition by channelization and levee construction projects. Since construction of the levees, there has been little change in the river's position and the threat of lateral channel migration is now low. Streambed elevation in this segment varies from – 10 feet at the mouth to +25 feet at RM 10.3.

The average channel gradient varies from 0.034 percent near the federal levees to 0.062 percent between for upper non-federal levees. Bed materials are primarily medium and fine sands with minor amounts of gravel. More than 95 percent of the sediment is less than one millimeter (mm) in diameter. The median particle diameter is 0.35 mm (medium sand) (Tetra Tech 2009). Available subsurface exploration portrays the levee foundation as layers of soft to medium stiff silt and medium dense silty sands. A 3-foot organic silt layer was noted in the Puyallup Authorized Right Bank levee 2011 URS boring log performed for the Puyallup GI study. The levee fill is typically a medium dense silty sand. (See Appendix F-4 for more detailed and specific geotechnical properties).

3.3 History

The non-federal levees were constructed through the joint efforts of the City of Tacoma, the Inter-County Improvement District, and the Washington State Department of Highways [SOURCE: 1973 Puyallup Basin Review Study], mostly between 1914 and 1916, although levee construction continued until 1934. These levees were built to protect the surrounding mostly agricultural areas from floodwaters of the White River and Puyallup River after the permanent diversion of the White River into the Lower Puyallup River in 1906. Prior to 1906, the White River flowed north through King County and joined the Green River.

The federally owned and operated levees from approximately RM 0.7 to RM 2.8 were designed and constructed in conjunction with Mud Mountain Dam. Mud Mountain Dam, located on the White River about 29 miles above its mouth, was authorized by the 1936 Flood Control Act and substantially completed in 1953. However, operation of the project began in 1943. The reservoir and its storage capacity are operated exclusively for flood control. The Puyallup federally authorized levee was a companion project of the dam. The project was authorized under the same act and completed in 1950. This project includes a 2.2 mile long straightened channel with levee improvements.

The right bank of the federal project was altered for the construction of an environmental restoration wetland in 1988 and expanded in 2007. The Gog-le-hi-te project now encompasses 3,500 linear feet of levee setback.

3.4 Historical Flooding & Performance

Major flooding occurred in the Lower Puyallup River in 1917, 1933, 1965, 1977, 1986, 1990, 1996, 2006 and 2009. The largest flood on record since construction of MMD occurred in January 2009, with a flow of 48,200 cfs.

Seepage boils with significant sand cones and landward slope stability sloughing were noted on the North Levee Road levee near the Sha Dux site in the January 2009 event, which loaded the levee to approximately 90% of its full height. This location has been repaired by the County with a seepage berm and toe drain in Fall of 2012.

Figure #: Seepage boils and landward toe sloughing during January 2009 high water.

The right bank of the Puyallup authorized federal flood reduction project suffered erosion damage in 2010 from large woody debris caught on the bank that had concentrated jet scour into the levee embankment. This damage was caused without a significant high water.

“The levee and revetment in the vicinity of 12th Street SE (approximately RM 9.3, left bank) has been overtopped on several occasions in the last 20 years, including 1996, 2006 and 2009, resulting in flooding and sediment deposition along the top of levee and adjacent areas. No significant damages were identified.” (Pierce County Rivers Flood Hazard Management Plan).

Levee Segment	Location	Damage	Length	Flood Event
North Levee Road	RM 5.3	Erosion	~150	2006
North Levee Road	RM 4.5	Seepage Boils and landward slope sliding	25	January 2009
Puyallup Authorized RB	RM 2.4	Large woody debris induced erosion	50	2010

Table #: Lower Puyallup River Levee Damages in the last 20 years.

The non-Federal levees along the lower Puyallup designated as “North Levee Road” and “River Road” Levees utilize concrete panels for erosion protection. The concrete panels were installed circa 1917 and utilized brush mats for toe protection. During the existing conditions analysis, these erosion control features were identified as a system vulnerability. The condition of concrete panels is degraded near RM 3.0 and RM 3.1 on the right bank. No major erosion performance issues have resulted to date. An upper slope concrete panel repair was completed in 2006. See Section 2.7 in Appendix F-2 for additional detail on the concrete panel erosion protection system and the implications of these features on levee fragility.

3.5 Maintenance & Condition

The levees in the Lower Puyallup region are in minimally acceptable condition. There have been no outstanding risks to levee integrity noted in the Corps' CEI inspections. Vegetation was the most common maintenance deficiency. "Since 1983, legal limitations have modified vegetation management practices and gravel and silt removal. In 1985, Pierce County and the Puyallup Tribe of Indians adopted an inter-governmental agreement for the Puyallup River Vegetation Management Program."(Pierce County Rivers Flood Hazard Management Plan). The Corps maintains the federal levee projects.

3.6 Failure Mode Assessment

The seepage and piping failure mode is the most likely in the Lower Puyallup region because of the geologic sediment deposition history. Layers of fine-grained soils are underlain by coarser soils have potential to produce high exit gradients during higher hydraulic loadings. Based on available subsurface explorations, the blanket condition is limited to areas of historic channel migration and soil stratigraphy indicates layers of not significantly dissimilar hydraulic conductivity characteristics (i.e. silt (ML) underlain by silty sand (SM). The blanket thickness is considered sufficient to dissipate the head from hydraulic loading in most situations. The levee sections are typically wide and relatively short, but some areas were found to be critical and in need of modification. Specifically, North Levee Road has experienced seepage problems in a critically tall and narrow levee section. The boils and landside slope sloughing gave indications of both through seepage and underseepage and piping. This critical location was repaired in the Fall of 2012 utilizing a seepage berm and toe drain to mitigate the seepage. Erosion and scour is not a predominant failure mode because the river bed slope is relatively shallow in comparison with the rest of the basin. However, large woody debris flushed through the system has led to damages when logs are caught on the banks and forces jet scour into the levee embankments. The ageing concrete panels on the non-Federal levees in the lower 8 miles have performed well in past events, but present an uncertainty to future performance. Overall, the levees along the Lower Puyallup River have performed well in past high water events and are relatively reliable against the primary failure modes. With this level of analysis, construction of shallower landside slopes in critically steep and tall areas, thereby increasing the seepage path, would be considered sufficient to mitigate the seepage potential.

4.0 <u>MIDDLE PUYALLUP</u>

4.1 Introduction

The Middle Puyallup River reach begins at the confluence of the White River at RM 10.3 and continues upstream to the confluence with the Carbon River at RM 17.4, downstream of the City of Orting. Throughout this reach the river channel is a combination of large meander bends with segments, which are straightened and confined by a combination of levees, revetments and valley walls. This region incorporates seven NLD levee segments. The levees in this region are generally smaller geometric prisms and do not provide a high return period level of protection against flooding.

Name	Bank	River Mile Location	Authorization	PL84-99 Rating
River Grove	Right	RM 11.0 –RM 11.4	Non-Federal	Unacceptable
Riverside	Right	RM 12.0 – RM 12.8	Non-Federal	Unacceptable
Bowman-Hilton	Left	RM 12.8 – RM 13.6	Non-Federal	Unacceptable
Sportsmen	Left	RM 13.6 – RM 14.4	Non-Federal	Unacceptable
McMillin	Left	RM 15.7 – RM 16.65	Non-Federal	Unacceptable
Bower-Parker	Left	RM 16.65 – RM 17.5	Non-Federal	Unacceptable
Lindsay	Right	RM 16.9 – Carbon RM 1.2	Non-Federal	Minimally Acceptable

Table #: Middle Puyallup River Levees

4.2 Geotechnical Properties

The Middle Puyallup River Valley is a broad low-gradient alluvial plain in which the river meanders and periodically floods. Review of the boring log from the Sportsmen levee dictates a foundation consisting of poorly graded sands with silt and silty sands with varying amounts of gravel to a depth of 65 feet below ground surface. The levee fill contains more gravel (GM) compacted to a very dense state. (See Appendix F-4 for more detailed and specific geotechnical properties).

4.3 History

"From the late 1920s to 1939, Pierce County River Improvement focused on channelization and bank stabilization using wooden bulkheads and debris barriers along the Puyallup and Carbon Rivers." (Pierce County Rivers Flood Hazard Management Plan). Bank protective works along critical points along the Puyallup River above Sumner, the Carbon River, and South Prairie Creek were constructed in 1936 as a WPA project under the direction of the Corps and transferred to local interests for maintenance. "In 1939, Pierce County approved a plan (Resolution No. 686) for flood control for the Puyallup above the mouth of the White River. The 1939 flood plan recommended creation of a single channel on the Puyallup River by excavating gravel and river sediments and side casting them to form levees that were armored with rock riprap. In the 1930-50s levees and revetments were constructed to prevent channel migration through agricultural lands."(Pierce County Rivers Flood Hazard Management Plan).

4.4 Historical Flooding & Performance

"The Middle Puyallup River experienced major flood events most recently in 1996, 2006, 2008, and 2009. The highest peak flow recorded at the Alderton Gauge occurred on January 7, 2009 with 53,600 cfs (based on the USGS calculation). However, this is thought to be an overestimate, because it is higher than the peak flow measured at the same time downstream at the Puyallup gauge in the Lower Puyallup River."(Pierce County Rivers Flood Hazard Management Plan).

The levees that have experienced repetitive damage include the Riverside Levee, Bowman/Hilton Levee, Sportsmen Levee, and Bower/Parker Levee. Damages sustained ranged from complete washouts resulting in the loss of several hundred lineal feet of flood control structure to localized moderate scour and erosion.

Levee Segment	Location	Damage	Length	Flood Event
Bowman/Hilton	RM 13.2 LB	Partial Washout	150	November 1995
Bowman/Hilton	RM 13.4 RB	Partial Washout	225	November 1995
Riverside	RM 12.8 RB	Toe/slope failure	600	February 1996
Bowman/Hilton	RM 13.2 LB	Toe/slope failure	500	February 1996
Bowman/Hilton	RM 13.2 LB	Total Failure	600	February 1996
Bower/Parker	RM 16.7 LB	Total Failure	100	February 1996
Bower/Parker	RM 16.8 LB	Toe/slope failure	800	February 1996
Bower/Parker	RM 17.4 LB	Toe/slope failure	100	February 1996
Bowman/Hilton	RM 13.2 LB	Toe scour	880	November 2006
Sportsman	RM 13.6 LB	Fracture	40	November 2006
Sportsman	RM 14.0 LB	Washout	300	November 2006
Bower/Parker	RM 17.3 LB	Face Erosion	220	November 2006
Lindsay	RM 17.4 RB	Face Erosion	50	November 2006
Riverside	RM 12.4 RB	Damaged Toe & Face	236	November 2008
Sportsman	RM 13.6 LB	Blocked Culvert	105	November 2008
Bower/Parker	RM 16.8 LB	Toe Rock Failure	125	November 2008
Bowman/Hilton	RM 13.2 LB	Scour facing rock failure, water leaking thru levee	200	January 2009
Bowman/Hilton	RM 13.2 LB	Top/Face Scour	390	January 2009
Bowman/Hilton	RM 13.3 LB	Top Scour	50	January 2009
Sportsman	RM 13.75 LB	Blocked culvert and scour	200	January 2009
Sportsman	RM 13.9 LB	Top Scour	250	January 2009
Sportsman	RM 14.0 LB	Major scour 40%	310	January 2009

		of facing rock missing for 100 lineal feet		
Sportsman	RM 14.1 LB	Head cutting on back side of levee adjacent to Sportsmen Access Road, scour	150	January 2009
McMillin	RM 16.1 LB	Access Road Grading / Debris Removal	900	January 2009
McMillin	RM 16.1 LB	Toe and face rock failure	60	January 2009
Bower/Parker	RM 16.7 LB	Toe/Face failure	300	January 2009
Bower/Parker	RM 16.8 LB	Toe rock failure	75	January 2009
Lindsay	RM 16.9 RB	Toe/Face failure	100	January 2009

Table #: Middle Puyallup River Levee Damages in the last 20 years.

4.5 Maintenance & Condition

The levees in the Middle Puyallup region are in minimally acceptable to unacceptable condition. Vegetation was the most common maintenance deficiency with many of the levees in this reach having large trees rooted within the levee prism. Encroachments and obstructions were noted on the Sportsmen and Bower/Parker levees.

4.6 Failure Mode Assessment

The erosion failure mode is the predominant most likely failure mode in the Middle Puyallup reach. Seepage is not a major concern due to the foundation properties and the low potential water head on the levees even under full loading. The unacceptable maintenance condition on a majority of these levees makes the overall risk of levee failure higher. However, due to the minimal height of the levee prism, such as the River Grove levee, some levees are expected to overtop before a breach prior to capacity exceedance would occur.

5.0 UPPER PUYALLUP

5.1 Introduction

The Upper Puyallup River begins at the confluence of the Carbon River at River Mile 17.4 and continues upstream to the Champion Bridge at RM 28.6, just downstream of Electron Road. In the lower portion of this reach, the river is confined by a combination of levees and revetments. Two setback levees, the Old Soldiers Home setback levee at RM 21.5 to RM 22.5 and Ford setback levee at RM 23.4 to RM 25.0 have reduced channelization through the Orting area. Above RM 25.0 few levees and revetments remain due to past flood damages and changes in flood management strategies. For example, the Larson levee on the left bank of the Puyallup River between RM25.7 and RM 28.5 breached in several locations in 1996 and in 2007. Only levee remnant islands remain. Scour, erosion, and channel avulsions are common in this reach.

The surrounding watershed and land use is mostly urban on the right bank of the Puyallup near the City of Orting between RM 17.4 to RM 21.8, but predominantly agricultural, rural residential and forested upstream of RM 21.8. This region incorporates eight NLD levee segments. The levees in this region are generally medium geometric prisms with average levee heights and typical 2H:1V slopes that meet Corps' standards.

Name	Bank	River Mile Location	Authorization	PL84-99 Rating
High Cedars	Right	RM 17.5 – RM 19.7	Non-Federal	Unacceptable
Calistoga	Right	RM 19.7 – RM 21.25	Non-Federal	Unacceptable
Leach Road	Left	RM 19.9 – RM 21.8	Non-Federal	Minimally Acceptable
Old Soldiers Home	Left	RM 21.25 –RM 23.1	Non-Federal	Acceptable
Jones	Right	RM 21.25 – RM 22.5	Non-Federal	Minimally Acceptable
McAbee	Left	RM 23.1 – RM 23.9	Non-Federal	Minimally Acceptable
Ford	Right	RM 22.5 – RM 24.9	Non-Federal	Minimally Acceptable
Neadham Road	Right	RM 26.4 – RM 26.9,	Non-Federal	Acceptable

Table #: Upper Puyallup River Levees

5.2 Geotechnical Properties

The Upper Puyallup River Valley is steeper and narrower compared with the Lower and Middle Puyallup River reaches. Above the confluence with the Carbon River, the width of the Puyallup River channel migration zone is defined by the remnants of the Electron mudflow, which was deposited as a thick layer of mud that blanketed the Puyallup valley bottom about 500 years ago. Review of the boring logs from this region reveal foundations soils consisting of medium dense to dense, coarse sands and gravels. Only the Leach Road boring displayed a silt layer in the foundation. Levee embankment material is typically dense gravels with sand and silt. (See Appendix F-4 for more detailed and specific geotechnical properties).

5.3 History

The levee construction history is similar to the Middle Puyallup with much of the historical channelization and bank armoring complete in the 1930s by local government interests.

5.4 Historical Flooding & Performance

The Upper Puyallup River experienced flooding most recently in 1990, 1996, 1999, 2000, 2006, 2008 and 2009. The largest flood event on record at the USGS gauge near Orting occurred on November 6, 2006 with a flow of 21,500 cfs

Significant and repetitive flood damages have been sustained by the levees in the Upper Puyallup region. The levees that have experienced the most damages include Neadham Road, Calistoga, and Old Soldiers Home.

Levee Segment	Location	Damage	Length	Flood Event
Neadham Road Levee	RM 26.8 RB	Reconstruction	250	November 1990
Neadham Road Levee	RM 25.4 RB	Reconstruction		November 1990
Neadham Road	RM26.3 RB	Reconstruction		November 1990
Leach Road Levee	RM 20.0 LB	Reshape and replace riprap and toe rock	195	November 1995
Old Soldiers Home Levee	RM 22.5 LB	Partial Washout	200	November 1995
Old Soldiers Home Levee	RM 22.9 LB	Partial Washout	200	November 1995
Ford Levee	RM 23.7 RB	Partial Washout	200	November 1995
Neadham Road	RM 25.1 RB	Partial Washout	200	November 1995
Neadham Road	RM 25.6 RB	Re-face RR grade as Setback Levee	200	November 1995
Neadham Road	RM 26.8 RB	Land Acquisition and Setback Levee	500	November 1995
High Cedars Levee	RM 17.6 RB	Toe/Slope Failure	400	February 1996
High Cedars Levee	RM 19.0 RB	Total Levee Failure	100	February 1996
Calistoga Levee	RM 19.8 RB	Total Levee Failure	1200	February 1996
Calistoga Levee	RM 20.2 RB	Mostly Toe with some Slope Failure	200	February 1996
Calistoga Levee	RM 20.7 RB	Mostly Toe with some Slope Failure	300	February 1996
Calistoga Levee	RM 20.8 RB	Mostly Toe Failure	100	February 1996
Calistoga Levee	RM 20.9 RB	Toe/Slope Failure	300	February 1996
Calistoga Levee	RM 21.2 RB	Toe/Slope Failure	200	February 1996
Old Soldiers Home Levee	RM 21.9 LB	Toe/Slope Failure	400	February 1996
Jones Levee	RM 22.5 RB	Total Failure	200	February 1996
Jones Levee	RM 22.9 RB	Toe/Slope Failure	300	February 1996
Jones Levee	RM 23.1 RB	Full Levee Washout	200	February 1996
McAbee Levee	RM 23.6 LB	Full Levee Washout	1200	February 1996
Ford Levee	RM 23.6 RB	Full Levee Washout	900	February 1996

Ford Levee	RM 24.6 RB	Full Levee Washout	1200	February 1996
Neadham Road	RM 26.4 RB	Full Levee Washout	600	February 1996
High Cedars Levee	RM 18.0 RB	Washout	50	November 2006
High Cedars Levee	RM 19.4 RB	Washout	150	November 2006
High Cedars Levee	RM 19.8 RB	Washout	100	November 2006
Leach Road Levee	RM 19.4 LB	Washout	50	November 2006
Leach Road Levee	RM 19.8 LB	Washout	200	November 2006
Old Soldiers Home Levee	RM 22.6 LB	Face Erosion	100	November 2006
Ford Levee	RM 22.8 RB	Washout	350	November 2006
McAbee Levee	RM 23.6 LB	Washout	600	November 2006
Neadham Road	RM 26.8 RB	Washout	1000	November 2006
Old Soldiers Home Levee	RM 22.01 LB	Toe/Slope Failure	80	November 2008
High Cedars Levee	RM 19.4 RB	Toe/Slope failure		
Neadham Road	RM 26.3 RB	Washout		
Leach Road Levee	RM 19.8 LB	Toe Scour, Overtopping Failure	550	January 2009
Jones Levee	RM 21.25 RB	Scour, Erosion	500	January 2009
Old Soldiers Home Levee	RM 21.3 LB	Scour, Erosion	290	January 2009
McAbee	RM 23.2 LB	Scour, Erosion	150	January 2009

Table #: Upper Puyallup River Levee Damages in the last 20 years.

Figure #: 2010 Jones Levee Rehabilitation

5.5 Maintenance & Condition

The levees in the Upper Puyallup region are in unacceptable to acceptable condition. All the levees in this reach were rated minimally acceptable with the exception of the Old Soldiers Home and Neadham Road levees that received acceptable ratings and High Cedars and Calistoga receiving unacceptable ratings. Vegetation was the most common maintenance deficiency; however, culvert condition and lack of interior inspection was noted. The inspection report for the Ford levee cited encroachments and the report for Leach Road cited existing minor scour damages.

5.6 Failure Mode Assessment

The most likely prior to capacity exceedance failure mode for the Upper Puyallup levees is due to erosion. The section of the Leach Road levee analyzed with the silt layer in the levee foundation produces some likelihood of seepage and sliding issues when loaded to the top of levee for an extended duration. Existing levee fill material and foundation materials are permeable, therefore, seepage is considered likely. But, probabilistic seepage analysis did not indicate high likelihood of a seepage and piping failure.

6.0 WHITE RIVER

6.1 Introduction

The Lower White River flows through the cities of Auburn, Pacific, and Sumner before joining the Puyallup River at RM 10.3. The White River is well known for its large sediment discharge and high turbidity levels. There is only one levee on the White River, the Potelco levee, commonly referred to by local entities as county-line levee. The levee is located on the left bank of the White River, looking downstream.

Levee Segment	Bank	River Mile Location	Authorization	PL84-99 Rating
Potelco	Left	RM 4.9 – RM 6.2	Non-Federal	Unacceptable

Table #: White River Levees

6.2 Geotechnical Properties

Lower White River Valley soils are composed of fine sand, silt, and peat. No additional subsurface exploration was performed on the White River for this existing conditions report.

The levee foundation is medium dense sand with silt. Portions of the foundation are layered with 1 to 2-inch thick lenses of silt deposits. The levee embankment material consists of medium dense to dense poorly graded gravel. (See Appendix F-4 for more detailed and specific geotechnical properties).

6.3 History

Prior to 1906, the White River flowed north past Auburn, where it joined the Green River and flowed to Elliott Bay in Seattle. Meanwhile, the Stuck River flowed south towards Sumner and joined the Puyallup River. Record flood flows in November 1906 caused a massive log jam that diverted flows into the Stuck River channel to the south and out through the Puyallup River to Commencement Bay. The diversion was made permanent by local interests through the construction of a barrier at Auburn in 1915.. The dam prevented the White River from flowing back to the north. Between 1914 and the mid 1930s the Lower White River was channelized and confined by a combination of revetments or levees.

6.4 Historical Flooding and Performance

In the last 20 years major flooding in the Lower White River occurred in 1990, 1996, 2006, and 2009 (see Table 5.21). The largest flood on record occurred in December 1933, prior to the construction of Mud Mountain Dam. Since the Auburn Gauge (#12100496) was installed in 1990, the largest flood occurred in 1996 with a maximum flow of 15,000 cfs.

The 1996 event caused toe scour and slope failure along 150 linear feet of levee near RM 4.2. During a high water event in 2009, the Potelco levee was overtopped due to aggradation of sediment in the channel, reducing the channel capacity. The 2009 flood peaked at 12,000 cfs; lower than flows in 1996 and 2006, which did not result in overtopping.

Levee Segment	Location	Damage	Length	Flood Event
Potelco	RM 4.2 LB	Toe/Slope Failure	150	February 1996
Potelco	RM 5.05 – RM 5.15 LB	Levee overtopping from adjacent wetland, 3ft of scour	650	January 2009
Potelco	RM 5.35 – RM 5.5 LB	Levee overtopping from adjacent wetland, 3ft of scour	570	January 2009

Table #: White River Levee Damages in the last 20 years.

6.5 Maintenance & Condition

The Potelco levee was given an unacceptable condition during the Corps' 2011 continuing eligibility inspection (CEI) for eligibility in the Corp's rehabilitation and inspection program. Overgrown vegetation, levee crown rutting, poor sod cover, and minor erosion were noted along the length of the levee.

6.6 Failure Mode Assessment

Erosion and scour is the most-likely failure mode prior to capacity exceedance based on the results of the probabilistic fragility curve analysis. The Potelco levee prism is small with minimal head loading on the structure. However, due to aggradation of sediment in the river channel, overtopping is a major concern. This levee is expected to overtop prior to any failure mode causing breach prior to capacity exceedance. However, maintenance deficiencies provide added risk to levee performance.

7.0 CARBON RIVER

7.1 Introduction

The Puyallup GI focuses on the lower portion of the Carbon River as it enters the trough-like valley below RM 8.4. This reach includes 8 levee segments in the Corps' NLD. The right bank is largely forested from RM 0.8 to RM 8.4. Below RM 0.8 the right bank is largely agricultural land. The left bank of the river from RM 0.75 to RM 3.54 is within the City of Orting and contains the Orting Wastewater Treatment Plant and single-family residential development. Between RM 3.4 and RM 8.3, the left bank land use consists mostly of agricultural and rural residential land.

Levee Segment	Bank	River Mile Location	Authorization	PL84-99 Rating
Lindsay	Right	Puyallup RM 16.9 –RM 1.2	Non-Federal	Minimally Acceptable
Riddell	Left	RM 0.0 – RM 1.7	Non-Federal	Unacceptable
Orting Treatment Plant	Left	RM 1.7 – RM 3.05	Non-Federal	Unacceptable
Bridge Street	Left	RM 3.05 – RM 3.7	Non-Federal	Unacceptable
Guy West	Left	RM 4.6 – RM 5.6	Non-Federal	Minimally Acceptable
Alward Segment #2	Left	RM 5.95 - RM 6.4	Non-Federal	Unacceptable
Water Ski Levee	Right	RM 5.95 – RM 7.0	Non-Federal	Minimally Acceptable
Alward Segment #1	Left	RM 6.55 – RM 8.26	Non-Federal	Minimally Acceptable

Table #: Carbon River Levees

7.2 Geotechnical Properties

From the confluence with the Puyallup River to just upstream of Orting at RM 4.0, the Carbon River flows next to the Cascadia plateau. The Electron mudflow deposited more than 15 feet of dense clay-rich mud across the Orting Valley.

The levees have been constructed on historical river floodways. Subsurface exploration along the banks of the Carbon River revealed medium dense to dense poorly graded sand with silt and gravel alluvium. The percentages of gravel and silt varied along the river. For example, the Orting Treatment Plant levee foundation is typically gravel with sand, while the Guy West foundation is typically silty sand. The levee embankment was constructed of similar materials. (See Appendix F-4 for more detailed and specific geotechnical properties).

7.3 History

On June 5th, 1939 Pierce County approved Resolution No. 686, a plan for flood control of the Middle Puyallup River, Upper Puyallup River and Carbon River. The plan was to establish a single channel on the Carbon River and Puyallup River (upstream of the White River confluence) by excavating gravel and river sediments and side casting them to form levees that were armored with rock riprap. This was the standard practice until the 1970s.

Beginning in the 1960s, river improvement policies focused on construction of levee and revetments along the Carbon River to straighten the river, increase sediment transport downstream, and prevent valley wall sediment from eroding into the river (GeoEngineers 2003). Current levees along the Carbon River were primarily built in the 1960s. The once meandering river channel was straightened and confined to an average width of 250 feet. The levee system was designed to prevent sediment sources from the banks and cliffs adjacent to the river from entering the channel contributing to increased sediment transport. It was believed that by constricting the channel width, there would be increased flow velocities to continue sediment transport downstream.

7.4 Historical Flooding and Performance

Major flooding of the Carbon River occurred in 1933, 1959, 1977, 1990, 1996, 2006, 2008, and 2009 (see Table 5.35). The November 2006 flood is the largest on record, with a measured flow of 14,500 cfs. The categorization of major flooding is based on a threshold of discharges in excess of approximately 10,000 cfs at the Fairfax gauge.

The occurrence of 6 major flood events since 1990 has resulted in numerous and recurring damage to levees on the Carbon River. Damages have been primarily associated with slope and toe failure mode that resulted in severe damages along several hundred feet of levee. The Upper Carbon levees (Alward, Water Ski, and Guy West) sustained the majority of the damages in recent history. Damages have been estimated at nearly $15 million dollars (based on 2010 dollars) (Pierce County Flood Hazard Management Plan).

A damage report from the November 2008 flood event cited seepage and piping along 1,200 LF of the Guy West levee.

Levee Segment	Location	Damage	Length	Flood Event
Bridge Street Levee	RM 3.2 LB	Washout	175	January 1990
Water Ski Levee	RM 6.4 RB	Partial Washout	300	January 1990
Water Ski Levee	RM 6.4 RB	Washout	450	January 1990
Alward Segment No 2 Levee	RM 5.9 LB	No Record Found	400	November 1990
Alward Segment No 1 Levee	RM 6.8 LB	No Record Found	750	November 1990
Alward Segment No 1 Levee	RM 7.2 LB	No Record Found	1,300	November 1990
Water Ski Levee	RM 6.4 RB	No Record Found	500	November 1990
Alward Segment No 2 Levee	RN 6.3 LB	Partial Washout	250	November 1995
Alward Segment No 1 Levee	RM 6.7 LB	Partial Washout	350	November 1995
Water Ski Levee	RM 6.9 RB	Partial Washout	200	November 1995
Alward Segment No 1 Levee	RM 7.1 LB	Washout	700	November 1995
Lindsay Levee	RM 0.2 RB	Toe/Slope Failure	500	February 1996
Lindsay Levee	RM 0.8 RB	Toe/Slope Failure	379	February 1996
Bridge Street Levee	RM 3.6 LB	Washout	350	February 1996
Alward Segment No 2 Levee	RM 6.05 LB	Toe/Slope Failure	250	February 1996
Alward Segment No 2 Levee	RM 6.25LB	Toe/Slope Failure	250	February 1996

Alward Segment No 1 Levee	RM 6.6 LB	Toe Failure	500	February 1996
Water Ski Levee	RM 6.9 RB	Washout	400	February 1996
Alward Segment No 1 Levee	RM 6.9 LB	Toe/Slope Failure	250	February 1996
Water Ski Levee	RM 7.1 RB	Washout	800	February 1996
Alward Segment No 1 Levee	RM 7.2 LB	Washout	850	February 1996
Bridge Street Levee	RM 3.6 LB	Face Erosion	200	November 2006
Guy West Levee	RM 4.6 – RM 4.9 LB	Toe Erosion	1,700	November 2006
Guy West Levee	RM 5.0 LB	Face Erosion	270	November 2006
Alward Segment No 2 Levee	RM 6.0 – 6.1 LB	Face Erosion	600	November 2006
Water Ski Levee	RM 6.0 RB	Washout	500	November 2006
Water Ski Levee	RM 6.0 RB	Washout	300	November 2006
Alward Segment No 2 Levee	RM 6.3 LB	Washout	600	November 2006
Water Ski Levee	RM 6.4 RB	Washout	500	November 2006
Water Ski Levee	RM 6.8 RB	Washout	550	November 2006
Alward Segment No 1 Levee	RM 7.5 LB	Washout	1,200	November 2006
Alward Segment No 1 Levee	RM 7.6 LB	Washout	700	November 2006
Alward Segment No 1 Levee	RM 8.3 LB	Face Erosion	300	November 2006
Riddell Levee	RM 0.4 LB	Toe/Face Scour	634	November 2008
Bridge Street Levee	RM 3.5 LB	Toe/Face Scour	300	November 2008
Bridge Street Levee	RM 3.6 – RM 3.7 LB	Toe/Face Scour	380	November 2008
Guy West Levee	RM 4.8 LB	Undercut Bank And Piping	1,200	November 2008
Guy West Levee	RM 5.0 LB	Toe/Face Scour	290	November 2008
Guy West Levee	RM 5.2 LB	Toe/Face Scour	196	November 2008
Guy West Levee	RM 5.3 LB	Toe/Face Scour	253	November 2008
Water Ski Levee	RM 6.0 RB	Toe/Face Scour	336	November 2008
Alward Segment No 2 Levee	RM 6.0 LB	Face Scour	824	November 2008
Alward Segment No 2 Levee	RM 6.25 LB	Toe/Face Scour	302	November 2008
Water Ski Levee	RM 6.45 – RM 6.6 RB	Toe/Face Scour	900	November 2008
Alward Segment No 1 Levee	RM 7.2 – RM 7.3 LB	Toe/Face Scour	796	November 2008
Water Ski Levee	RM 6.2 RB	Toe/Face Scour	255	January 2009
Water Ski Levee	RM 6.4 RB	Toe/Face Scour	310	January 2009

Water Ski Levee	RM 6.75 RB	Toe/Face Scour	200	January 2009
Riddell Levee	RM 0.9 LB	Toe/Face Scour	180	January 2009

Table #: Carbon River Levee Damages in the last 20 years.

7.5 Maintenance & Condition

The levees along the banks of the Carbon River are in acceptable to unacceptable maintenance condition in the Corp's rehabilitation and inspection program. This reach includes 8 levee segments in the Corps' NLD. Primarily, overgrown vegetation was the major maintenance issue. A few encroachments onto the levee prism were noted during the inspections. The Bridge Street levee, Orting Treatment Plant levee and the Riddell levee exhibited erosion and scour damages at the time of the routine inspections in 2013.

7.6 Failure Mode Assessment

Steeper gradients and larger historical channel migration zones in this Carbon River reach make the Carbon River levees more susceptible to erosion and scour damages. This can be supported by the extensive damage history for the reach. Similarly, erosion emerged as the primary failure mode is the probabilistic fragility curve analysis. Larger levee prisms, such as Guy West and Bridge Street, have a potential for seepage and landward stability issues when subjected to a high water head under full loading. Due to the permeable nature of the levee embankment and foundation soils, seepage of is anticipated, but not likely to initiate internal erosion.

8.0 FUTURE WITHOUT PROJECTS

With the City's current system of levees, risk of flooding from unexpected problems, larger floods, or uncertainty associated with the reliability of the existing levees will always remain. However, no quantifiable changes to the levee reliability can be inferred. Barring unforeseen events, Pierce County is expected to continue their maintenance and any minor damages done to the levees throughout this time period are expected to be repaired. Therefore, the levee structures are expected to perform similarly in the future without projects as they do currently.

9.0 REFERENCES

Duncan, J.M., 2000, *Factors of Safety and Reliability in Geotechnical Engineering*, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 4.

Pierce County. 2012. Pierce County Rivers Flood Hazard Management Plan. Pierce County, Washington.

Appendix F-2

Geotechnical: Levee Fragility

Puyallup River Basin
Flood Risk Reduction Feasibility Study

Department of the Army
Seattle District, US Army Corps of Engineers

EXECUTIVE SUMMARY

The main levee product is a potential breach location with fragility curve and breach characteristics.

The weakest point in each levee system was designated as the potential failure point (P.F.P.). The following items were produced for each potential breach location:

- Fragility curve: relationship between river elevation and probability of levee breach.
- Duration trigger: period of time that a flood event river elevation must be at or above the pre-determined breach elevation in order for the levee to breach. (The duration trigger does not apply to levee breach due to overtopping.)
- Breach characteristics: breach width, breach depth, breach side slope, and time for full breach to develop.

ABBREVIATIONS AND ACRONYMS

Corps	U.S. Army Corps of Engineers
GI	General Investigation
P.F.P.	Probable Failure Point

VARIABLES

d_{50}	Mean particle size of a sample by weight
n	Manning's Roughness Coefficient
S_o	Channel Bed Slope
V_c	Critical Velocity
y	Bank Full Depth

1.0 INTRODUCTION

1.1 Scope

Complete existing condition levee fragility curves for each of the levee segments within the Puyallup General Investigation Study Basin, 28 total, for use in the HEC-FDA model.

1.2 Levee Analysis Guidance

ER 1105-2-101, *Risk Analysis for Flood Damage Reduction Studies* provides guidance on the evaluation framework to be used in Corps flood risk analysis studies. "The ultimate goal is a comprehensive approach in which the values of all key variables, parameters, and components of flood damage reduction studies are subject to probabilistic analysis." The "structural and geotechnical performance of existing structures" is a variable that must be treated probabilistically.

EM 1110-2-1619, *Risk-Based Analysis for Flood Damage Reduction Studies* provides guidance for describing the uncertainty of levee performance. The uncertainty of levee performance is represented by a levee fragility curve expressing the relationship between probability of levee breach and river elevation.

There are two rescinded ETLs that discuss reliability analysis for geotechnical engineering: ETL 1110-2-547 and ETL 1110-2-556.

ETL 1110-2-547, *Introduction to Probability and Reliability Methods for Use in Geotechnical Engineering* (September 1997) provides an introduction to the use of probabilistic methods in geotechnical engineering. The ETL discusses the Taylor's series method which was used in this Puyallup GI levee analysis. The Discussion paragraph of the document says:

"This is the first in a series of ETL's that will provide guidance on the use and application of probability and reliability methods of analyses for use in the assessment of existing levees for benefit determination and the geotechnical portion of major rehabilitation reports."

A second ETL, 1110-2-556, discussed below, was published in May 1999 and in March 2003 the expiration date was extended to June 2004. These two ETLs have expired and no new guidance has been issued.

ETL 1110-2-556, *Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies* (May 1999) provides guidance on the application of probabilistic methods to geotechnical aspects of water resource planning studies. Appendix A provides a broad overview of methods and issues in geotechnical reliability analysis. Appendix B is a research report prepared by Thomas F. Wolff and provides methods for developing levee fragility curves. The following comments are made in the ETL:

"Both appendices provide overviews and examples of probabilistic methods, but should not be construed as definitive "how-to" guidance. The general approach in Appendix B, however, shall be used as the framework for evaluating the reliability of existing levees, … The experience of both the Corps of Engineers and the geotechnical profession with probabilistic methods continues to evolve. Published research includes a variety of methods, some elegant, but difficult to implement, and some overly simple. Furthermore, the appropriate choice of methods may be very problem-dependent. Hence, methodology should be developed on a case-by-case basis, using these examples as a reference point."

The Puyallup GI levee analysis follows the general framework in expired ETL 1110-2-556. Specifically, the Taylor's series method was used for evaluating the probability of the initiation of two failure modes: underseepage and piping and landward slope sliding. The method for evaluating surface erosion was also adopted from this ETL, with a separate equation for calculating critical velocity.

In addition, the method described in the ETL for combining probabilities for different failure modes was used. There are several aspects of levee reliability analysis for which there is no clear guidance, including the progression of failure modes from initiation to breach, the influence of flood fighting failure modes other than overtopping, and the influence of levee condition items such as vegetation and encroachments. For several of these aspects, the expert elicitation procedure discussed in Appendix A of ETL 1110-2-556 was used to subjectively determine levee analysis input values that could not be calculated or determined by other means. The expert elicitation was originally produced for the Columbia River Treaty Phase 2A Flood Risk Assessment Report and has been adopted for these purposes. The questions posed to the expert panel were not project specific and the experts were all from the Northwest area. The factors and guidance resulting from that expert elicitation session are logically applicable to this project.

In summary, the requirement to incorporate levee performance uncertainty in the form of a fragility curve is clear, but the methods to create a fragility curve that includes all the significant factors influencing the probability of breach are still evolving. For the Puyallup GI levee analysis, the framework outlined in the expired ETLs, and specific methods such as the Taylor's series method, were used. Expert elicitation originating from the Columbia River Treaty Phase 2A Flood Risk Assessment Report and further engineering judgment was used to fill information gaps.

2.0 FRAGILITY CURVES

Fragility curves on levees, which express the probability of levee failure in a particular mode caused by a stressing event.

2.1 Probable Failure Points

Weak reaches and subsequent failure points were selected by judgment considering crest elevation, past performance, levee geometry (crest width, landward slope ratio, levee height, etc.), levee and foundation materials, levee condition, and other relevant information. In most cases, the probable failure point was chosen in the vicinity of an existing subsurface exploration or the borings performed in conjunction with this project.

2.1.1 Cross Section Analysis

Cross section data of the levees in the project area originate from a survey contracted by the USACE in summer of 2010. Levee cross sections were surveyed on all PL84-99 eligible levees in the levee database approximately every 1000 feet. Ineligible levees not including in the surveying effort included Bowman Hilton, McMillin, River Grove, Riverside, and Sportsman. To supplement the cross section data and to field check the levee contractor surveyed levees for vulnerable levee sections, an additional effort was performed by USACE in July of 2011. Vulnerable cross sections are those sections with steeper slopes, shorter crown widths and tall landward heights. Design and construction drawings from various levee repairs were also consulted to give an accurate representation of the levee cross sections.

Levee cross sections with riprap armor on the riverward slope was assumed to have a 3' horizontal riprap blanket. This assumption was based on the typical d_{50} of the armor. For example, most levees in the Puyallup Basin have a Class IV riprap armoring. This armor has a d_{50} of approximately 16 inches. The assumption was made that the thickness of the blanket measured perpendicularly to the slope was one grain size, in this case, 16 inches. On a typical 2H:1V slope the horizontal distance was approximately 3 feet in width. This blanket thickness is not recommended in Corps design guidance for sizing riprap, EM 1110-3-136, but seemed to be a more accurate representation of what was encountered on site visits. The Puyallup Authorized levees, being federally constructed and maintained, were modeled with a blanket thickness as recommended by the engineering manual. A sensitivity analysis on the affect of the riprap blanket thickness on the phreatic surface through the levee embankment returned marginal impacts. Therefore, the assumptions made for the blanket thickness were found to be appropriate.

2.2 Potential Failure Modes

Levee failure is defined as a failure of the levee to provide the intended level of protection to people and property in the protected area behind the levee. Levee fragility curves give a probabilistic representation towards the reliability of the levee at various water surface elevations. The fragility curves can also be used as input to hydraulic modeling and economic analysis of levee failure consequences.

The levee fragility curve includes multiple failure modes that could lead to levee failure or breach. For the analysis of the Puyallup River, Carbon River, and White River levees in the Puyallup General Investigation project area the probable failure modes include landward slope failure under steady state seepage conditions, seepage and piping failure under steady state seepage conditions, and riverward slope erosion.

These failure modes govern until the capacity of the levee is exceeded. Overtopping is the predominant failure mechanism. Therefore, once a flood results in incipient overtopping, the levee is assumed breached at that potential failure point.

Seismic failure was not included because the likelihood of a significant seismic event occurring simultaneously with a significant flooding event is remote.

Static slope stability for both the riverward and landward slopes were not included in this analysis because slope stability has not been a prevalent failure mode. As the river stage rises, riverward slope stability should improve as hydrostatic pressure adds stabilizing forces to the soil mass; further resisting movement. This analysis was directed toward failure modes directly related to high water events; failure as a function of rising water surface elevation.

Similarly, rapid drawdown failure was not evaluated because the failure mode involves sliding of the riverward slope after the river has dropped in elevation and is generally no longer a flood threat. In order for this failure mode to lead to breach, the levee would have to remain unrepaired until the next large flood(s).

2.3 Geotechnical Investigation

A geotechnical investigation was completed to provide soil information for the fragility curve creation. This soil information and laboratory testing can be found in Appendix F-4.

2.4 Seepage Analysis

Seepage analyses for the Puyallup Basin levees in the Puyallup General Investigation project area were performed using GeoStudio's numerical modeling software, Seep/W.

2.4.1 Seepage Variables

Input variables into the Seep/W model include horizontal conductivity, a ratio of vertical to horizontal conductivity, and a volumetric water content function. Horizontal conductivity values were assigned to soil layers based on field test data and established relationships between soil type and hydraulic conductivity. Field testing included seven drive point piezometer slug tests and three completely developed well slug tests. The depth interval of these tests was planned in conjunction with the soil layer contact zone between the levee fill and the levee foundation. Approximately half of the tests would focus on the levee fill material permeability and the other half would target the levee foundation material. For the model, a most likely value (average) value for a soil layer would be established along with a standard deviation. The range of hydraulic conductivity in the field tests allowed for an order of magnitude standard deviation. The Hazen equation was used as a check to field tests.

The ratio of vertical to horizontal conductivity was introduced to soil layers that exhibit a characteristic for water to flow at different rate through a soil in one direction compared to another direction. Isotropic or anisotropic quality of the soil layers was used to distinguish the most likely value and standard deviation of the conductivity ratio. For layers that displayed several alternating thin layers of sand and silt, an anisotropic condition was assumed and a lower ratio of vertical to horizontal conductivity was applied. The range of these values was established using typical material values coupled with a sensitivity analysis to ensure values were realistic. Documentation in the Seep/W modeling handbook suggests using ratios closer to 1:1.

Finally, the volumetric water content function, which describes the volume of water stored in the void spaces of a soil in relation to the porewater pressure. Sample functions for typical soil types given in the Seep/W modeling handbook were used in the absence of laboratory test data.

2.3.2 Boundary Conditions

After the surveyed cross section geometry was input into the model, boundary conditions must be established. The riverside slope and channel bottom were assigned a constant head boundary equivalent to the water surface elevation analyzed. Depending on the levee height, the full analysis was run at top of levee and intervals ranging from one to three feet for a total of three points. For example, for an 8 foot levee, analyses were run at top of levee, two feet below the crest, and four feet below the crest. These water surface elevation points establish the fragility curve as a function of water height. The landward slope of the levee prism was assigned a seepage face boundary condition. The ground of the landward toe was assigned a zero pressure boundary. The mesh size of the numerical model varied by levee height, but the maximum mesh size was two feet by two feet. As a general practice the riverside end of the model was kept at 3 times the levee height with a minimum of 30 feet from the toe. The landside boundary was spaced at 5 times the levee height with a minimum of 50 feet from the toe. The stratigraphy below the toe of the levee was input to a depth of 3 times the levee height with a minimum of 30 feet. These distances prevent geometry from impacting the numerical model.

2.5 Slope Stability Analysis

Slope stability analyses for the Puyallup Basin levees located in the Puyallup General Investigation project area were performed using GeoStudio's limit equilibrium numerical modeling software, Slope/W.

2.5.1 Slope Stability Variables

Input variables into the Slope/W model to derive a factor of safety using a Mohr-Coulomb soil include the soil's total unit weight, angle of internal friction, and cohesion. Total unit weight includes the weight of the soil and water in a volumetric unit. Total unit weights for the predominantly sands and gravels of the Puyallup River alluvium did not require laboratory testing. Typical values for pervious soil types are well established. The standard deviation for the unit weights of the soils was established as 5 pcf. Friction angle and cohesion describe the shear strength of a soil based on the Mohr-Coulomb failure envelope. The granular materials that make up the levee foundation and the levee fill were assumed to be Cohesionless. The friction angle was estimated from a calculated $N1_{60}$ blow count number that incorporates the field blow count number and a correction factor for the overburden stress. (Method is based on the WSDOT Geotechnical Manual dated January 2010). Friction angles related to blow count numbers were also confirmed with soil strength relationships found in literature. The standard deviation for the friction angle of the soils was established as 3 degrees.

2.5.2 Slope Stability Slip Surfaces

A slope stability failure occurs when the mass of the soil and the driving forces overcome the strength of the soil and the resisting forces. Slope stability slip surfaces were found using the entry and exit method to allow the user to better define the region of interest. Slip surfaces were required to initiate through the levee crown and have a minimum slip surface depth of three feet. This negates local slips and slumps that would not likely initiate a total levee failure. No slip surface exit requirements were defined. Surfaces that exit through the landward slope, toe, or foundation were assessed.

2.6 Erosion Analysis

Risk and reliability analysis against the erosion failure mode was performed using the surface erosion method from ETL 1110-2-556, appendix B. As flood stages increase, the potential for erosion due to excessive velocities parallel to the levee slope increases. The probabilistic method similar to the Taylor Series method relates velocity calculated by an adaptation of Manning's equation to a critical velocity (V_c) that would transport a particle away from the bank. A few methods for assigning a critical velocity were reviewed. Ultimately, the Yang method was implemented.

$$V_c = 2.05*(\text{Fall Velocity})$$

$$\text{Fall Velocity} = 6.01*\sqrt{(d_{50})}$$

Information regarding the existing slope armor d_{50} size was found in levee as-built plans, historical levee rehabilitations, inspection reports, or field visits.

2.6.1 Erosion Variables

The input variables required to complete the probabilistic erosion analysis are those required to satisfy Manning's equation and include the slope of the energy grade line (S_o), Manning's roughness coefficient (n), and water depth (y).

As recommended in ETL 1110-2-556, the slope of the energy line is approximated by the slope of the river bed. This approximation assumes uniform flow through the system; an assumption necessary for the process to work, but one that also has its drawbacks. (See Section 2.12: Assumptions and Uncertainty). The river bed slope was calculated from USGS cross sections being used in the HEC-RAS model. An average slope was calculated for each leveed reach.

Manning's roughness coefficients (n) were carefully selected based on tabulated typical values experienced in natural streams (Mays, 2005). The upper Puyallup, lower Puyallup, Carbon, and White Rivers all differ in channel characteristics regarding roughness. A unique Manning's n was assigned to each region.

The input values for roughness and slope were also assigned coefficients of variation based on our confidence in selected most likely values. Typically, the coefficient of variation for slope was 10%, to account for any localized variation in slope. The coefficient of variation for roughness was 15% to account for uncertainty in selecting roughness values for such a wide array of river characteristics.

The water depth (y) was taken from the nearest available USGS cross sections to the potential failure point (P.F.P). If multiple cross sections existed in the general vicinity of the P.F.P the most critical, deepest, bank full depth was used.

2.6.2 Calibration

Historically, the erosion and scour failure mode has been the most prevalent within the Puyallup GI project area. Therefore, the probabilistic erosion method was calibrated to some historical failures, such as Neadham Road, in order to gauge accuracy of the method and have confidence in results.

2.7 Lower Puyallup Erosion Application: Concrete Panels

The Lower Puyallup levees rely primarily on concrete panels for erosion protection. The concrete panels are ageing and the foundation protection is unreliable and would not likely meet scour protection standards today; however, past performance suggests that there is not an erosion issue in the lower Puyallup reach. The levees are also very wide; therefore, a full breach would require significant channel migration or lateral erosion. Characterizing levee fragility for the erosion failure mode has proved difficult. Guidance is not documented on the procedure to account for these types of features in the erosion failure mode analysis. The critical velocity of ageing concrete panels and brush mats is not well established. The following section describes the existing conditions, construction history, and methodology to account for erosion potential in the Lower Puyallup Levees.

2.7.1 *Existing Conditions*

North Levee Road (right bank, RM 2.8 - 7.7) and River Road Levee (left bank, LM 2.8 – 7.4) are non-Federal levees in the lower Puyallup analyzed for modification under a General Investigation. The levees are proposed to be modified and setback to increase channel capacity and levee reliability.

The levees are aligned on both banks of a straightened reach of the lower Puyallup. The right bank has a defined 10-20 foot wide silt bench along the toe of the levee along most of its length (Figure 1). The left bank slope does not have a significant bench in most areas. River Road levee also functions as the SR-167 embankment and the levee crest is typically 60-75 feet wide. North Levee Road varies from 20-50 feet in crest width.

Figure 1: Flood loading on North Levee Road. Note concrete panels along riverside slope for erosion protection. Trees in stream are rooted in a silt bench riverward of panels. (Looking downstream).

2.7.2 *Construction & Performance History*

The levees were constructed on alluvial deposits of silt and fine sands and included filling stream meanders to straighten the reach. The foundation soils are highly to moderately erodible. The non-Federal levees were constructed with concrete panel erosion protection installed in around 1917. The panels do not extend below scour depth, but relied on brush mats to provide erosion and scour protection. In some areas silt benches have established covering the brush mats. Much of the condition of the concrete panels and brush mats are unknown. Along North Levee Road, silt is cleaned from the slope to expose concrete panels for spot inspections. No severe degradation has been noted. On the River Road side, modification to the riverside trail required vegetation removal from atop the panels. The stripping of vegetation caused damage to the panels that needed to be repaired. A 540 LF concrete panel repair near station 227+00 (RM 7.2) was performed in 2006. The repair included soil nails, rebar mesh, and 5” shotcrete overlay. Existing revetment thickness varied from 4-8 inches thick. The existing concrete was reported as friable and contained a large coal fraction as aggregate.

Figure 2: Typical concrete slope revetment, brush mats, and concrete blocks on mats constructed.

Historically, the levees have had good performance during flood loading. The 2009 flood event loaded the levees to approximately 90% of their height without major damages to the levee. Near RM 5.3, the silt bench eroded near to the concrete panels. The panels were not disturbed, but the county installed bank protection of the natural silt bench using a log and dolo matrix in 2009. The levee has experienced other large loadings in 2006 and 1996 without documented erosion damages.

Two instances of concrete panel degradation have been noted by Pierce County personnel on the left bank River Road Levee. See Figure 2. Concrete blocks are visible in the channel. These are either from the slope panel protection or from concrete blocks used to weigh the brush mats down during installation.

Figure 3: Concrete rubble at the toe of the River Road Levee slope near RM 3.1.

Figure 4: Concrete rubble at the toe of the River Road Levee slope near RM 3.0.

2.7.3 Methodology

The methodology of assigning probability of failure for these levees was developed from comparing the critical velocity of moving riprap particles based on the expected channel velocity using normal depth. The manning's equation velocity estimates correlated well with the HEC-RAS velocity modeling. A top of levee loading is estimated to result in channel velocities of 7-10 ft/s.

The critical velocity is not easily calculated for the panels or brush mats, but based on historical performance in the 2009 event, the velocity resulting in the initiation of erosion was approximately a top of levee event. Therefore, a critical velocity was set to be approximately 10 ft/s. This results in a likelihood of erosion failure at top of levee loading of 36%. Although based on engineering judgment, this revised methodology provides a realistic assessment about the erosion potential given the ageing erosion protection while calibrating to historical events.

2.8 Combining Failure Mode Probabilities

The fragility curves for piping, sliding, and erosion were combined by using the equation below to combine the probabilities at each river elevation.

$$P_{combined} = 1 - [(1 - P_{piping})\ (1 - P_{sliding})\ (1 - P_{erosion})]$$

2.9 Adjustment Factors

2.9.1 Levee Condition Factor

Levee condition factor is based on the maintenance level of the levee. This factor is added on the premise, all else being equal, a levee that is unacceptably maintained is more likely to result in failure than a levee in excellent condition. Information was gathered on the status of levees in the Public Law 84-99 rehabilitation program. The Corps performs biennial continuing eligibility inspections of the levees in this study area. A levee is rated according to a national maintenance and condition criteria. The most recent inspection of the levees in this study area was 2010. These rating and justifications were the basis of the levee condition factor. A levee with an acceptable (A) or minimally acceptable rating (M) was assigned a condition factor of 1 and deemed to have no additional impact to levee performance. A levee in an unacceptably maintained state was assigned a condition factor of 2 and deemed to be twice as likely to result in failure. The severity of the reason(s) for an unacceptable rating was also assessed. Categories rated in the continuing eligibility inspections include vegetation maintenance, sod cover, encroachments, slope stability, erosion/bank caving, settlement, depressions/rutting, cracking, animal control, culverts, riprap revetments and bank protection, and seepage. Major damage could result in a condition factor of 3 or greater. However, for this set of levees no major damage existed and unacceptably maintained levees were assigned a condition factor of 2.

2.9.2 Initiation to Breach[1]

The reliability analyses for the reference sections evaluated the probability of the *initiation* of the piping and sliding failure modes. There is a significant difference between the initiation of a failure mode and full levee breach. This step involves quantifying the difference between initiation of a failure mode and breach and the influence of specific methods of flood fighting.

Consider the development of a boil moving foundation material. A piping failure mode has initiated. The factor of safety against piping initiation is less than 1.0. In order for the levee to breach, however, the failure mode must progress with time. A pipe, for example, must advance from the point of the boil back toward the river. When enough foundation material has been removed the levee may collapse and breach. Given the duration of the flood event and the nature of the foundation conditions, there is a difference between the probability of failure mode initiation and the probability of breach.

The initiation versus breach factors from the expert elicitation session are 0.18 for piping and 0.83 for sliding. These factors were applied as follows. The probability of breach considering initiation versus breach is equal to the probability of failure mode initiation times the initiation versus breach factor. For example, if the probability of the piping failure mode initiating is 1, the probability of breach considering initiation versus breach is (1)(0.18) = 0.18, a smaller probability.

2.9.3 Flood Fighting

Although flood fighting is expected at all sites in the Puyallup GI project area, flood fighting factors developed to decrease the probability of failure were ultimately not included in this analysis. This decision was made because the relative fragility of a levee system should not depend on human intervention. The project team wanted to stay conservative in this regard.

2.9.4 Length Effect

The length of a levee system affects its reliability in that the longer the levee system, the greater the chance of a weak spot. The New Orleans IPET report described the length effect. The excerpts below are from *Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System Final Report of the Interagency Performance Evaluation Task Force, Volume VIII – Engineering and Operational Risk and Reliability Analysis* (USACE 2009):

"The HPS of New Orleans includes long lengths of embankment or wall extending many miles across ground that is poorly characterized from an

[1] Developed in Columbia River Treaty Flood Risk Assessment Report. Levee Analysis, Appendix F.

engineering perspective. Levees fail at locations where loads are high and strengths are low. If these critical locations are identified ahead of time, traditional methods can be used to analyze stability and calculate factors of safety. In such situations, the overall length of levee is immaterial, because the weakest spots have been identified and dealt with. The probability that the levee fails is that of these weakest spots.

The more common situation is that the levee system is not characterized with enough detail to know unambiguously where the weakest spots are. In this case, any reach of levee has some probability of experiencing higher than average loads or lower than average strengths, and as a result, of being a "weak spot." Since this critical combination cannot be uniquely identified before a failure occurs, the longer the levee, the greater the chance that a critical combination exists somewhere, and thus the higher the probability of a failure somewhere."

"The primary level of analysis of levee reliability is the two-dimensional levee section. The presumption is that this 2D section applies over a unit length of levee, defined approximately as the horizontal autocorrelation distance, and treated as a probabilistically independent characteristic length. As the total length of levee increases, the probability of systems failure rises in proportion to length and soon displays a classic exponential saturation shape trending asymptotically toward 1.0, according to the formula

$$P_s = 1 - (1 - p)^n$$

in which, $P_s = 1 - (1 - p)^n$ is the probability of system failure, p is the 2D probability of failure, and n is the number of characteristic lengths within the reach."

Recent studies have used both approaches. The IPET team performed an analysis of undrained shear strength data to estimate a characteristic length of 1,000 ft for earthen levees and used the equation referenced above. The study for a Post-Authorization Changes Report for the Natomas Basin in Sacramento District (USACE 2010) involved analyzing the weakest spots in the levee and no adjustments were made using the equation above for the levee length.

For the Puyallup GI levee analysis, efforts were made to identify and analyze the weakest spot on each levee system. As described above in the IPET report, if this can be done, "the overall length of levee is immaterial, because the weakest spots have been identified and dealt with. The probability that the levee fails is that of these weakest spots." The weakest spots were selected as described above in Section 2.1: Probable Failure Points.

2.10 Assign Breach Characteristics[2]

[2] Developed in Columbia River Treaty Flood Risk Assessment Report. Levee Analysis, Appendix F.

In order to assess the flood damage reduction capability of proposed alternatives within the Puyallup GI, a HEC-FDA model was developed. The previously assembled levee fragility curves for existing conditions were assigned to probable failure points (PFPs) along the identified hydraulic storage areas. To model inundation areas, breach characteristics for these PFPs were developed. The following characteristics were estimated for each potential breach location:

- Breach depth
- Breach width
- Breach side slopes
- Time required for full breach to develop

The breach depth for levees often extends into the foundation as has been documented in numerous earthen embankment breach studies. This characteristic would be anticipated for the Puyallup GI levees due to erodible foundation soils. However, in the HEC-RAS modeling of flow into a protected area through a levee breach, flow below the interior ground elevation is not evaluated. Therefore, there is no reason to estimate breach depths greater than the levee height. For all Puyallup GI levees, the breach depth was set equal to the levee height.

Information from a Southeast Region Research Initiative (SERRI) Report (70015-001), *Levee Breach Geometries and Algorithms to Simulate Breach Closure*, by Chris Saucier, Isaac Howard, and Joe Tom, Jr. from Mississippi State (October 2009) was used to help estimate breach widths, side slopes, and times required for full breach to develop. A Kansas District report entitled “Levee Performance and Breach Investigation, in response to June 2013 Flooding in the Lower Missouri River Basin” was also consulted to compare anticipated breach characteristics. This data was found to be applicable to the Puyallup Basin levees except for areas in the Upper Puyallup and Carbon River that have a breach history that is outside the scope of this data due to the low levee heights and the susceptibility to erosion and channel migration.

The SERRI Report provides maximum breach widths of 98 – 167 ft for 10 ft high levees and 150 – 525 ft for 15 ft high levees. These breach widths and heights correspond to width to height ratios ranging from 10 to 35. Levees comprised of cohesive soils have typically smaller breach widths than a cohesionless soil levee. Investigation into historical Puyallup Basin levee breaches provided additional data points. See Table 1 below.

Table 1: Historical Puyallup Basin Levee Breaches

Levee Name	Event Date	Mode of Failure	Levee Height	Breach Width	W:H Ratio
Water Ski	Nov 1990	Erosion	3’	700’	233.3:1
Alward Segment 1	Nov 1990	Erosion	4’	700’	175:1
Alward Segment 1	Nov 1990	Erosion	4’	850’	212.5:1
Alward Segment 1	Feb 1996	Erosion	4’	400’	100:1
Alward Segment 1	Nov 2006	Erosion	4’	1200’	300:1
Alward Segment 1	Nov 2006	Erosion	4’	800’	200:1

Ford	Feb 1996	Erosion	4’	1500’	375:1
Ford	Feb 1996	Erosion	4’	2200’	550:1
Leach Road	Jan 2009	Overtopping	5’	550’	110:1
Neadham Road	Feb 1996	Erosion	4’	600’	150:1
Neadham Road	Nov 2006	Erosion	7’	1500’	214.3:1
Neadham Road	Nov 2008	Erosion	4’	950’	237.5:1

Based on this breach performance history, the average breach prior to overtopping width to height ratio is approximately 190:1. (Ignores the Ford breaches as outliers). All of these levee breaches were caused by erosion and scour damages. The levees within the sample come from the Upper Puyallup and Carbon River areas. The average overtopping induced breach, based on a single data point from the Upper Puyallup, was 110:1.

For the Puyallup GI levees, the levees were broken into two groups based on location. The primary reasoning for this grouping is the river velocity characteristics that induced breaches in the Puyallup Basin samples. Levees along the Upper Puyallup and Carbon River were assigned the average breach width to height ratio of 190:1. Due to levee heights being relatively low, this ratio appears to be valid for these areas. Breach locations in the Lower Puyallup were assigned the upper range of the 10-foot levee breach data from the SERRI report: 17:1 because of similar size and cohesionless embankment soils. The embankment soils are typically cohesionless based on available subsurface exploration data. Although foundation soils for levee in the Lower Puyallup region portray cohesive properties, the breach depth is focused on the embankment fill height due to limitations with the HEC-RAS modeling.

The breach side slope for the Puyallup levees was assumed to be 2V:1H. This is a common side slope in the SERRI Report.

The SERRI Report provides initial breach formation times ranging from 2.23 – 5.23 hours and subsequent lateral erosion rates ranging from 30 – 200 ft per hour. It was assumed that the longer times from the SERRI Report might have been for more cohesive levees. Flood durations in the Puyallup River Basin typically last between 1-3 days. The Puyallup levees embankment materials are generally cohesionless and susceptible to erosion damages. To develop a time to breach for the Upper Puyallup and Carbon River based on historic trends, a breach progression time of 3 hours was estimated from the SERRI range. Based on a historical breach width average of approximatley1000 feet, the average breach development rate is 500 feet per hour for the Upper Puyallup and Carbon River levees with 1 hour for initiation. An average typical levee breach development time of 115 feet per hour was estimated for other locations based on the information in the SERRI report. An hour for initiation of the failure mode was added to the total time.

Based on the above, the following assumptions are used for the breach characteristics for Puyallup GI levees:

1. The breach depth will equal the height of the levee as measured from the top of the levee to the elevation of the interior toe. In other words, the bottom elevation of the breach will be the interior ground toe elevation.

2. The breach width will equal 190 or 17 times the breach depth depending on the location of the levee within the Puyallup Basin. The levee embankment material for all levees is assumed to be generally cohesionless.

3. The side slopes of the breach will be 2V:1H.

4. The time required for full breach to develop is the previously calculated breach width divided by the development rate of 333 or 115 feet per hour depending on the location of the levee.

Table 2: Breach Characteristics by Region

Breach Characteristic	Upper Puyallup & Carbon River	Other Locations
Breach Depth, D (ft)	Levee Height (H)	Levee Height (H)
Breach Width, W (ft)	190*H	17*H
Side Slopes	2V:1H	2V:1H
Breach Development (Hours)	1+W/500	1+W/115

2.11 Application for HEC-FDA Model

Within the HEC-FDA model, each flood storage area is associated with a relationship to the hydraulics. For each storage area, a fragility curve or incipient overtopping was selected depending on the most likely event to govern inundation of the leveed area. In some cases, a single flood storage area would include more than one levee segment and therefore, more than one potential failure point. To compare the levee overtopping and breach prior to overtopping probabilities, a most likely value was determined from the probability density curve (fragility curve) by converting it into a cumulative density curve. The most likely value (elevation) was compared to the incipient overtopping elevation and the lowest elevation in relation to the flood water surface was selected. The fragility curve was also selected if the comparison was within a 1 foot tolerance.

For use in the economic model, levee fragility was only applied for the Lower Puyallup reach. The other proposed alternative projects were justified without levee fragility; therefore, the fragility curves were not applied. Based on storage areas, three levee breach points were modeled and fragility curves associated with those points. The three fragility curves applied in HEC-FDA were North Levee Road, Puyallup LB, and

Puyallup RB. All three of these fragility curves were translated from their initial location based on modeling needs. Puyallup RB potential failure point was moved from station 85+04 to 85+00. Puyallup LB was translated from station 71+00 to 60+00. Finally, the North Levee Road curve was translated from station 57+80 to station 262+00. See section 2.12.4 for a discussion on uncertainty.

2.12 Assumptions and Uncertainty

"The methods described herein should not be expected to provide 'true,' or 'absolute' probability-of-failure values but can provide consistent measures of relative reliability when reasonable assumptions are employed. Such comparative measures can be used to indicate, for example, which reach (or length) of levee, which typical section, or which alternative design may be more reliable than another. They also can be used to determine which of several performance modes (seepage, slope stability, etc.) governs the reliability of a particular levee." (ETL 1110-2-556, B-11).

2.12.1 Levee Cross Section Uncertainty

The levee cross sections analyzed were chosen as vulnerable cross sections from previous survey work and limited field verification. More critical sections of the levees may be discovered through more detailed survey. In addition, the cross sections were simplified for model simplicity and to give a better representation of a typical cross section. Riprap layers were excluded from the seepage model analysis because the free draining, highly permeable layer does not have a significant impact on the steady state seepage model.

2.12.2 Subsurface Contact Elevation Uncertainty

The contact zone between levee fill and levee foundation was based on the limited subsurface exploration data. The prime source being the 10 boring logs done specifically for this project. Where boring data was available the contact layers were assumed horizontal. Where no subsurface data was available, the contact zone was drawn between the two known surface points; often making an angled contact zone. Since the failure modes analyzed were dealing with the landward slope, these contact zone approximations were found to be acceptable.

2.12.3 Erosion Uncertainty

The scour failure mode analysis, are based on straight, rectangular (channel width being ten times the channel depth) channels. The reality is the Puyallup project area has sections of rivers with bends, bars, and other natural features that alter the velocity and direction of flow. The effect these channel components have on the probability of failure can only be accounted for using semi-quantitative or qualitative means.

General approximations from USGS channel bathymetry cross sections were made to assign bed slope (S_o) and bank full depths (y) to the separate levee segments. The most critical bank full depth and steepest bed slope values were chosen.

The method set forth in ETL 1110-2-556 uses a simplified version of the Manning's equation assuming a rectangular channel for channels that are relatively wide compared to their depth (width ≥ 10 x depth). In addition, the slope of the energy grade line was assumed to be the bed slope. This assumption of uniform flow does not necessarily relate to the steep gradient mountainous streams in the Upper Puyallup and the Carbon River. However, the approximation was made based on the best available data.

Finally, erosion can be exacerbated by several means in a river channel. The location on a bend, constriction points, jet scour, etc. can all increase the possibility of erosion and/or scour. In this simplified method, these idiosyncrasies of the river channel cannot accurately be taken into account. Further detailed study should be done to fully assess the erosion resistance of the riprap protecting the riverside slopes of the levees.

2.12.4 Potential Failure Point Translation

Implementation of fragility curves in HEC-FDA modeling required the translation of fragility curves from a geotechnically designated potential failure point to a more hydraulically suitable point for modeling simplicity. Translating the fragility curve introduces significant uncertainty because the fragility curve was based on designated site soil conditions and levee geometry. Levee geometry was checked for applicability, but foundation soil similarities could not be confirmed.

<u>3.0 REFERENCES</u>

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