HYDRAULIC EVALUATION AND CONCEPTUAL DESIGN OF MITIGATION MEASURES FOR YAKIMA RIVER GRAVEL PIT CAPTURE

FINAL REPORT

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HYDRAULIC EVALUATION AND CONCEPTUAL DESIGN OF MITIGATION MEASURES FOR YAKIMA RIVER EAST BANK GRAVEL PIT CAPTURE DOWNSTREAM OF SR24 BRIDGE

FINAL REPORT

Prepared for:

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The following NHC staff participated in the study:

Andrew Nelson: fluvial geomorphology

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Steve Kwan: SRH-2D modeling

Dave McLean: river engineering

Christine McKim: CAD

EXECUTIVE SUMMARY

Yakima County Flood Control Zone District (the County) is engaged with the US Corps of Engineers in a Section 1135 Ecosystem Restoration Feasibility Study to mitigate for environmental impacts resulting from the 8 mile long Yakima Federally Authorized Flood Control Project levee (YFAFCP) and the 2.1 mile long Drainage Improvement District # 1 levee (DID #1). The alignment of the YFAFCP was placed largely along old DID #1 levee alignments with its southern terminus on both east and west banks at Birchfield Road. In 2006 the SR24 Bridge, located at the downstream end of the YFAFCP and upstream extent of DID #1 levees, was replaced due to scour failure and its span was expanded from 690 to 1590 feet through eight spans separated by piers. In 2011, a 2,300 foot levee setback of the YFAFCP was made on the eastern bank to tie the levee into the new bridge. The goals of a Corps 1135 project are to restore the ecosystem affected by past Department of Defense actions. The Yakima 1135 Feasibility Study entails the setback of the DID #1 levee south of SR24, any other YFAFCP setbacks, and restoration both upstream and downstream of SR24. The major goal of the Section 1135 project, when implemented, is to restore, as much as possible, the natural river processes to reduce risk to infrastructure and fish.

Setback of DID #1 levee along its entirety, the major action of the Yakima Section 1135, poses some risk to channel stability due to three former gravel pits (locally known as the Newland Ponds) located behind the levee downstream of the SR24 Bridge. The County and Corps require a mitigation plan and designs to minimize the hazard associated with river capture of the three gravel pits following planned removal of the DID #1 levee. The mitigation measures are intended to minimize the risk of the river avulsing into the pits, which would create channel instability and degradation upstream and downstream from the site. The County retained Northwest Hydraulic Consultants Inc. (NHC) to assess the hydraulic and geomorphic effects that could occur as a result of the pit captures and to develop conceptual designs of mitigation measures to limit the effects of pit captures on adjacent infrastructure, as part of the Section 1135 Ecosystem study. The investigations included:

- Geomorphic studies to review the effects of past pit captures and to develop a conceptual "process-response" model to characterize past hydraulic and morphological impacts as a predictive tool.
- One dimensional morphodynamic modeling for assessing future scenarios by updating and adapting the SRH-1D model that was developed by the US Bureau of Reclamation for previous geomorphic studies that were a precursor to the Section 1135.
- Two dimensional hydrodynamic modeling using the program SRH-2D to assess the effects of initial measures concepts at the ponds.
- River regime and hydraulic analysis to develop preliminary design parameters and conceptual designs of various mitigation measures and some modeling of projected impacts.

The results of these investigations led to an evolution of design concepts and their combinations that could not be fully modeled within the limited time frame and require further refinement using 2D morphodynamic modeling. Those proposals are described in Chapter 7.

The impact of a levee set-back alone that does not involve pit capture would be relatively small. The scenario also corresponds to the situation where the ponds have been completely filled in by design as a mitigation measure. The morphodynamic simulations indicate that average bed degradation amounted to 1 to 2 feet (maximum value of 3 feet) and extended up to 2,000 feet upstream of SR24 Bridge after 25 years. This scenario is unlikely to produce a significant threat to nearby infrastructure.

The geomorphic analysis indicated a high likelihood of channel avulsion and capture of the Newland Ponds if mitigation measures are not provided as part of the levee set-back project. The worst-case scenario involved the channel fully avulsing through all three ponds. The predicted headcut was estimated to be approximately 9 feet and the resulting headcut could propagate up to 13,000 feet upstream (reaching the Federal levee near Buchanan Lake). The recovery time scale for geomorphic conditions to resume along the reach was estimated as being on the order of half a century. In comparison, NHC's SRH-1D model predicted the average bed degradation was 3 to 5.5 feet, with the maximum degradation being 7 feet. The degradation extended upstream over 14,000 feet, (past Buchanan Lake and the Terrace Heights Bridge). The Newland Ponds had not completely filled in after a simulation time of 50 years.

The geomorphic-based predictions and 1D model results indicated that a full avulsion and capture by the three ponds without mitigation measures could induce significant upstream degradation and could potentially threaten the revetment protecting the Federal levee which separates the river and Buchanan Lake. Therefore, mitigation measures need to be incorporated into future levee set-back plans to prevent this situation from occurring.

The investigation showed it is technically feasible to mitigate the risk of an avulsion and to restore the channel to a more natural system, while still improving the overall stability of the reach and lowering flood levels by setting back the levees. The mitigation measures are intended to reduce the potential impacts of future channel changes after the levees are set back while restoring the river to a more natural form that existed prior to extensive channelization works.

The study showed that a combination of mitigation measures will be required to achieve the objectives of this project. The measures will need to extend over a total length of three miles (one mile downstream of SR24 Bridge and two miles upstream, past Buchanan Lake to the Terrace Heights Bridge). A broad range of mitigation measures were considered, including:

- Infilling a substantial portion of the three ponds,
- Installing a buried grade control sill below the floodplain surface on the left overbank at the SR24 Bridge,

- Retaining portions of existing bank protection revetments along sections of the bankline,
- Excavating accumulated gravel at selected bars to allow the river to occupy its preferred routes and re-direct flow attack away from structures,
- Excavating a major new side channel through the island upstream of the SR24 Bridge to minimize scour and degradation at the toe of the YFAFCP, and
- Installing a series of river training spurs along the Federal levee adjacent to Buchanan Lake.

Not all of these components will need to be installed, depending on the final mitigation plan that is adopted. One alternative included installing a buried sill near the site of the existing cross-dike and retaining some of the existing bank revetment riprap on the left bank upstream of SR24 Bridge. This alternative is intended as an interim measure to open overbank floodplain flows on the left floodplain but to prevent at least in the short term (next five years) the main channel of the Yakima River from creating a cut-off across the left bank through the present location of Pond 1 and Pond 2. After a period of years when the river has adjusted to the levee set-backs and other mitigation measures, the sill could be either removed or would be allowed to deteriorate. An alternative plan is to eliminate the buried sill and allow the river to develop a new channel through the left floodplain (essentially forming a chute cut-off). This new alignment would result in upstream degradation of approximately 3 feet. Depending on the sequence of future flood events it is likely that this alternative would speed up the river's response time to the mitigation measures. Ultimately, the selected measures to reduce the risk associated with pit capture should also further the environmental restoration goals of the Corps 1135 project.

Additional detailed hydraulic investigations need to be carried out to finalize the mitigation plan and to design the mitigation measures. This should include conducting 2D morphodynamic modeling over the 3 mile study reach to develop a detailed layout for re-grading the floodplain at the three ponds, assess the morphodynamic response of the Sportsman's Park side channel and evaluate scour conditions along the YFAFCP levee near Buchanan Lake.

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1 INTRODUCTION

1.1 Background

Yakima County Flood Control Zone District (the County) is engaged with the US Corps of Engineers (Corps) in a Section 1135 Ecosystem Restoration Feasibility Study to mitigate environmental impacts resulting from 8 miles of the Yakima Federally Authorized Flood Control Project (YFAFCP) levee and the 2.1 mile Drainage Improvement District # 1 levee (DID #1). The alignment of the YFAFCP was placed largely along old DID #1 levee alignments with its southern terminus on both east and west banks at Birchfield Road. In 2006, SR24 Bridge was replaced due to scour failure and its length was expanded from 690 to 1590 feet through eight spans separated by piers. In 2011, a 2,300 foot levee setback was made on the eastern bank to tie the levee into the new bridge. The 1135 Feasibility Study includes the setback of the DID #1 levee south of SR24, any other YFAFCP setbacks, and restoration both upstream and downstream of SR24. The goal of the Section 1135 is to restore, as much as possible, the natural river processes to reduce risk to infrastructure and fish.

Setback of DID #1 poses some risk due to three former gravel pits (locally known as the Newland Ponds) located behind the levee downstream of the SR24 Bridge. An avulsion of the Yakima River mainstem through the Newland Ponds (an uncontrolled pit capture) has the potential to damage and threaten current infrastructure including the new (2006) Highway SR24 Bridge and the YFAFCP levee, both located upstream of the ponds.

The County and Corps require a mitigation plan and designs to minimize the hazard associated with river capture of the three gravel pits following planned removal of the DID #1 levee. The mitigation measures are intended to minimize the risk of the river avulsing into the pits, which would create channel instability and degradation upstream and downstream from the site. This could produce bridge failures at SR24 and Terrace Heights bridges and YFAFCP levee failures over several miles, including triggering a potentially catastrophic failure of the levee in front of Buchanan Lake. The County retained Northwest Hydraulic Consultants Inc. (NHC) to assess the hydraulic and geomorphic effects that could occur as a result of the pit captures and develop conceptual designs of mitigation measures to limit the effects of pit captures on adjacent infrastructure, as part of the Section 1135 Ecosystem study. Figure 1 shows the project location and the configuration of the existing levees, the proposed set-back levee and the three Newland Ponds.

The County plan for the Gap to Gap reach levee set-back is part of a long term plan to reactivate the Yakima River floodplain in this reach in order to increase access to back channel habitat while also reducing flood elevations and velocities. The mitigation measures are intended to reduce the potential impacts of future channel changes after the levees are set back while restoring river forms which existed prior to extensive channelization works.

1.2 Scope of Work

In May 2014, the County retained Northwest Hydraulic Consultants Inc. (NHC) to assess the hydraulic and geomorphic effects that could occur as a result of the pit captures at the Newland Ponds. A second objective of the investigation was to develop conceptual designs of mitigation measures to limit the effects of pit captures on adjacent infrastructure. Specific tasks of the study were set out in the terms of reference prepared by the County and dated April 4, 2014. The main tasks were as follows:

1) Evaluate the likelihood and effects of uncontrolled river capture of the pits and the geomorphic and infrastructure impact of channel re-grading and/or avulsion through the subsequent river stabilization period.

2) Provide a sediment and geomorphic model of the pit capture phenomenon that can be used to assess infrastructure risk and mitigation measures.

3) Assess effects of related actions including the recent levee removal by the City of Yakima at the Yakima Regional Wastewater Treatment Plant (WWTP) and proposed levee removal of a 350 foot segment of the Federal Levee at the Nob Hill Wrecking Yard.

4) Develop conceptual mitigation measures that will reduce risks over the river stabilization period.

5) Identify temporary mitigation features required for existing infrastructure.

6) Conduct a risk mitigation review meeting with the County and stakeholders to identify potential mitigation features required for existing infrastructure.

7) Select, following County review, a preferred design concept for preliminary design and present to County and stakeholders to allow potential modifications in a final report.

Interim results were issued over the course of the study in the form of technical memoranda and presentations. These included:

- Meeting in Yakima: May 20-21, 2014.

- Technical Memorandum: Uncontrolled Pit Capture Likelihood and Risk, dated August 25, 2014.

- Presentation: Review of Yakima River Geomorphic and Sedimentation Study, September 24, 2014.

- Technical Memorandum: Hydraulic and Morphodynamic Modeling, DID #1 Levee Setback, DRAFT, dated November 30, 2014.

- Presentation: Gap to Gap Partners Meeting with USACE and Northwest Hydraulic Consultants, January 28, 2015.

This report supersedes and updates the preliminary results that have been presented earlier.

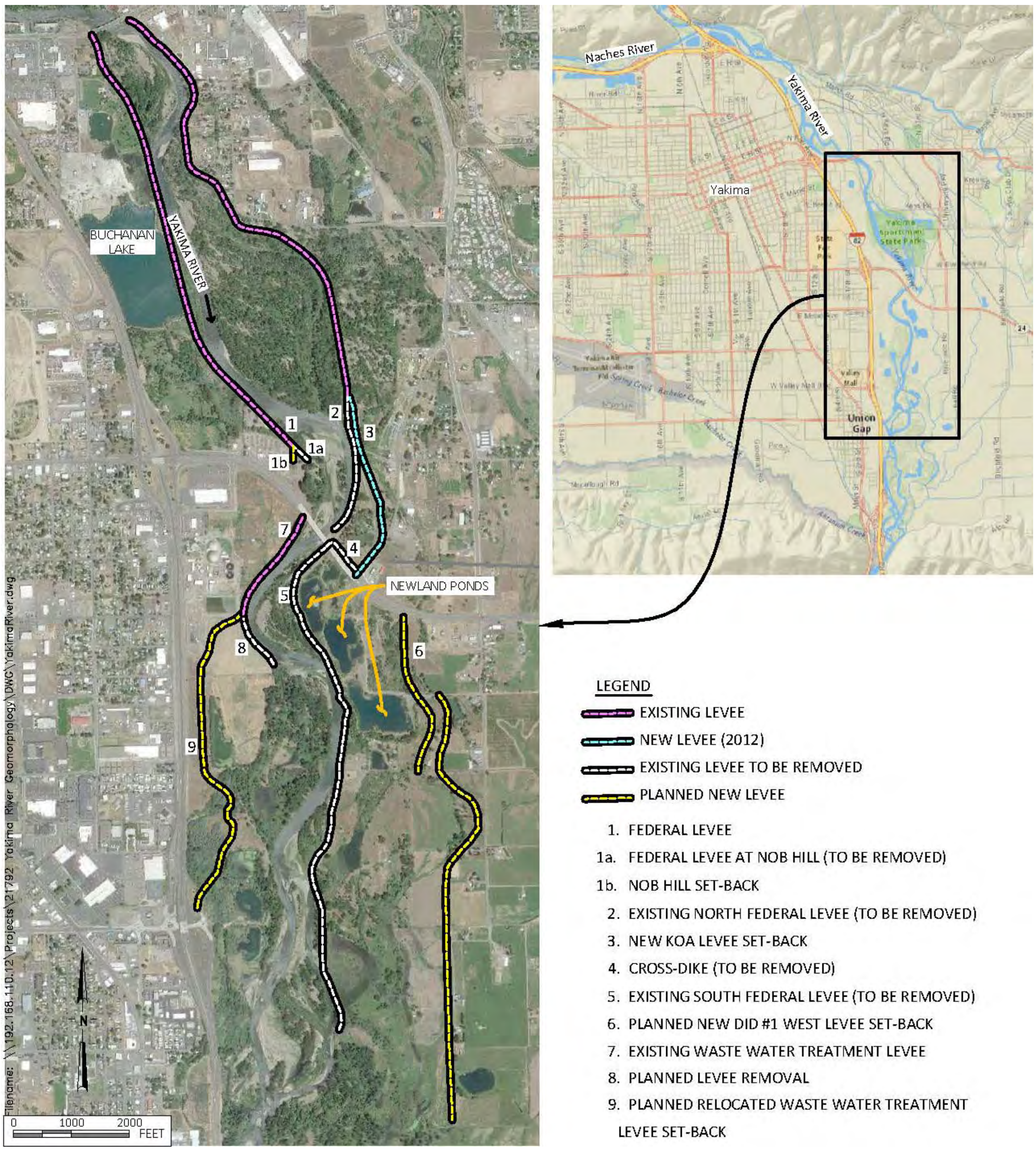

Figure 1: Project location

1.3 Work Carried Out

1.3.1 General Approach

River processes that govern sediment transport, bank erosion and channel pattern change are still incompletely understood. Therefore, predicting the physical responses of rivers to human activities such as gravel mining and past channelization activities is best addressed by using a combination of methods and then checking the results against field observations and past experience. In this investigation, we have used interpretative geomorphic methods, analytical methods and numerical modeling (one dimensional morphodynamic and two dimensional hydrodynamic modeling) to assess the effects of pit captures. All of the methods have certain inherent limitations. However, the combination of methods provides a basis for assessing the uncertainty in the results and for bracketing the expected response.

1.3.2 Field Studies

Bathymetric surveys of the Yakima River and Newland Ponds were conducted in July 2014 to provide current data to compare with previous channel surveys and to update the topography in the hydraulic and morphodynamic models. Channel bathymetry was collected from approximately 4,000 feet upstream and 5,500 feet downstream of the SR24 Bridge by Pacific Geomatic Services Inc. (PGS), under supervision by NHC. Bathymetry of Newland Ponds was collected by NHC. Both surveys were conducted using boat-mounted electronic sounding and GPS-RTK equipment. NHC used ArcGIS software to construct a combined bathymetric and topographic surface of the study reach. Topographic data consisted of 2013 LiDAR data provided by the County. A comparison of individual bathymetric survey point elevations and the constructed surface suggests an error band of less than +/- 1 foot[1].

1.3.3 Geomorphic Assessment of Pit Captures

A review of previous pit capture events on the Yakima River and other gravel bed rivers was carried out to describe the fundamental river processes driving pit capture and the long term channel response. The goal was to develop a general process-response model as well as to develop predictive relations that would be useful for quantitatively evaluating the geomorphic response.

The analysis was based on field observations and published results from rivers in Washington and Oregon. The compiled information is summarized in Appendix A. The results of the assessment are detailed in Sections 4.1 to 4.5.

1.3.4 1D Modeling

Yakima County provided NHC with an existing HEC-RAS model of the Yakima River. This model was used for general one dimensional hydraulic modeling and to set boundary conditions for subsequent two dimensional modeling work. The Bureau of Reclamation’s Sedimentation and River Hydraulics-One

[1] All elevations are referenced to NAVD88 vertical datum in this report unless explicitly noted otherwise.

Dimension (SRH-1D) model was also provided to NHC. This model was subsequently modified to refine predictions of levee set-back effects and to represent some cases of breaches into the Newland Ponds as well as some mitigation concepts. Results of the morphodynamic modeling are presented in Section 5.

1.3.5 2D Modeling

The SRH-2D hydrodynamic model was used to assess the local flow conditions in the main channel and floodplain upstream and downstream of the SR24 Bridge at the Newland Ponds. The analysis was made for the case of the set-back DID #1 levee and for three different floodplain re-grading scenarios. The mitigation scenarios were developed at a conceptual level during this phase of the work. The modeling was intended to help identify the feasibility of specific measures and to provide guidance for more detailed design-level investigations in the future. A description of model development and results are presented in Appendix B.

1.3.6 Mitigation Measures

The mitigation measures were intended to reduce the effect of a potential pit capture at the Newland Ponds after the DID #1 levee was set back. Initially, the mitigation measures were considered only for the area between the SR24 Bridge and the Newland Ponds. Subsequently, the geomorphic studies and morphodynamic modeling indicated that other mitigation measures were required upstream of the SR24 Bridge. Ultimately, the spatial extent of the mitigation measures extended from below the SR24 Bridge upstream to near the Terrace Heights Bridge.

1.4 Previous Investigations

The US Bureau of Reclamation (USBR) prepared the Yakima River Geomorphology and Sediment Transport Study: Gap to Gap Reach, Yakima WA in November 2010 (Hilldale and Godaire, 2010). The investigation included developing a one dimensional morphodynamic model of the Yakima River using the program SRH-1D. The model was used to assess future aggradation and degradation patterns along the river under existing conditions and with the levees set back. The study documented past gravel mining along the river but did not assess the effect of pit captures on the river hydraulics or channel morphology. The SRH-1D model has been adopted in this present study for estimating degradation due to pit captures.

Yakima County prepared the Upper Yakima River Comprehensive Flood Hazard Management Plan in 2007 (YCFHMP, 2007). The report describes the flooding history and flood impacts, flood management strategies and flood mitigation alternatives.

Clark (2003) assessed the morphological changes to the Yakima River caused by past gravel mining and levee construction. The author showed that the complexity of the channel has decreased over time in response to channelization and gravel mining.

Eitemiller et al. (2002) documented the historic changes to the floodplain of the Yakima River due to levee construction and other anthropogenic developments. This study showed that the active floodplain area has been substantially reduced since 1927.

The Yakima River Regional Greenway Foundation (1976) prepared a comprehensive plan to restore portions of the Yakima River. The study included a geomorphic assessment (Dunne et al., 1976), which assessed the structural/tectonic controls on the river and described the impacts of past gravel mining.

1.5 Terminology

This section briefly defines the terminology and concepts used in this report, particularly as they relate to short-term and long-term river response and morphological changes.

Degradation

Degradation usually occurs over relatively long time periods and results in a lowering of the general bed profile over a long reach length. Degradation is usually initiated by a reduction in sediment supply or an increase in discharge. These changes may occur as a result of land-use changes that increase flows from the drainage basin, construction of upstream reservoirs that trap bed sediment, lowering of downstream water levels caused by natural or artificial cut-off of meander loops and from channelization over long sections of the river. Such changes are often difficult to predict in the absence of detailed knowledge of the river system and its functioning.

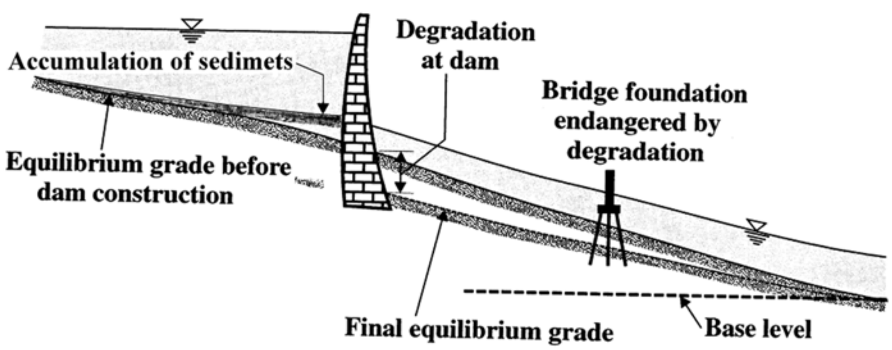

Figure 2: Example of channel profile change due to degradation (from Melville and Coleman, 2000)

Natural Scour

Scour is defined for present purposes as erosional lowering of the channel bed below its normal level. Depth of scour refers to the depth of material removed below the normal bed level. Natural scour occurs at sections of a natural channel under the influence of varying flows, sediment transport, channel shifting and other fluvial processes. There are several common types of natural scour, as illustrated in Figure 3.

Bend scour generally develops near the outer bank in meandering or curved channels. It is caused by 3-D helical flow generated by the intersection of curved streamlines with a vertical velocity gradient. In bends with fast bank erosion rates, the scouring may be partly compensated by deposition of sediment from the eroding bank. However, if erosion is halted by bank protection works or exposure of a resistant formation, the bend scour may increase.

Confluence scour occurs where two channels join, inducing helical flow and vortex systems. The augmented depth depends on the angle and flow distribution between the two channels.

Protrusion scour occurs when the main flow impinges on a bank or natural hard point protruding into the flow. Such features may result naturally from erosional exposure of a resistant geological formation.

Alternate bars may develop in straight river reaches in the form of segmental bars attached to alternate banks, causing the flow to follow a winding or meandering path. Scour holes tend to develop near the bank opposite the widest part of each bar.

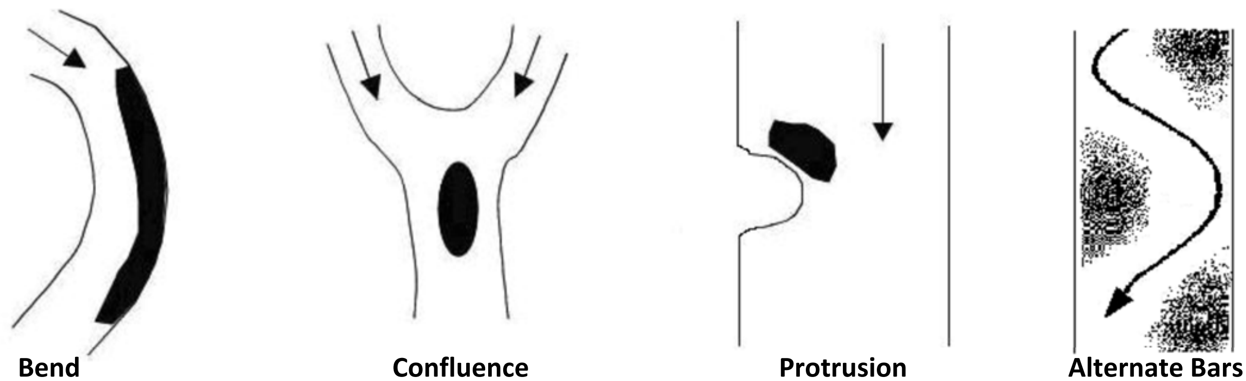

Figure 3: Examples of natural scour features

Constriction scour

Constriction (or contraction) scour tends to occur across a river section that is narrower than the average, usually as a result of artificial features (Figure 4). Situations may include:

- bank protection works
- deliberate narrowing of the river by blocking of overbank flow due to construction of flood control levees or bridge abutments.

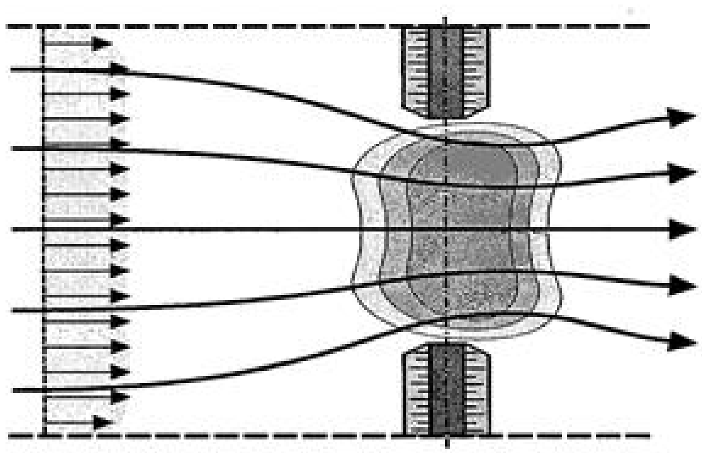

Figure 4: Examples of constriction scour (Melville & Coleman 2000)

Local Scour

Local structure-induced scour occurs near man-made structures like piers, abutments, spurs and barbs, and bank revetments that obstruct the natural flow or resist lateral erosion. It is caused basically by 3-D flow fields and local vortex systems generated by such facilities, often with strong rotational or vertical velocity components.

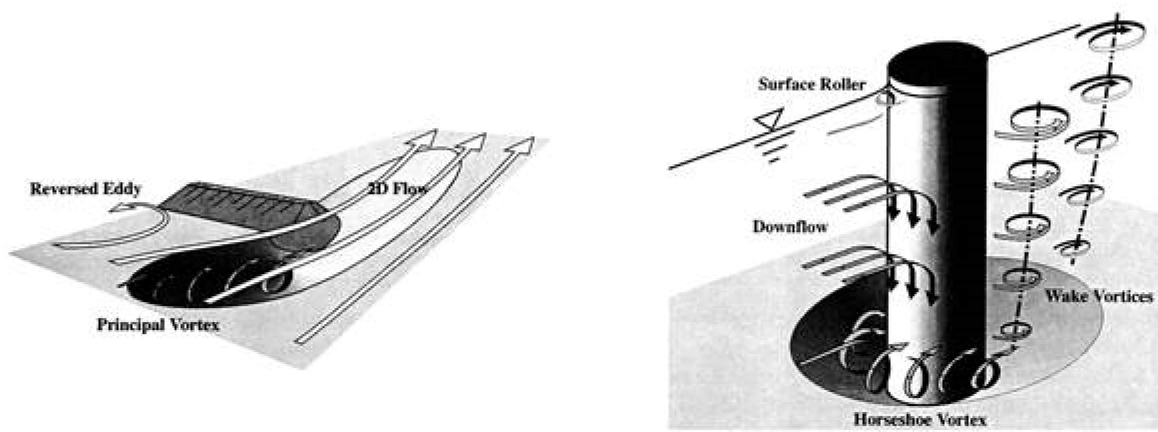

Figure 5: Structure-induced local scour at a groin and at a bridge pier (Melville & Coleman 2000)

2 BACKGROUND INFORMATION

2.1 Setting

The Yakima River is situated in a deep structural synclinal valley between the Yakima Ridge anticline on the north and the Ahtanum Ridge-Rattlesnake Hills anticline on the south. The Yakima River's largest tributary, the Naches River, enters just north of the City of Yakima and has an important role in supplying sediment to the study reach.

During Pleistocene times glaciers advanced from the Cascades down the Yakima Valley as far as Cle Elum (Dunne, 1976). Large quantities of coarse sediment from the Naches and Yakima basins were deposited on the floor of the valley. Since the retreat of the glaciers, the river has cut down approximately 10 feet into these sediments, leaving a terrace of sand and gravel along both sides of the valley. This terrace defines a natural corridor within which floodwaters are confined (Dunne, 1976).

2.2 Hydrology

The headwaters of the Yakima River are in the Cascade Range and the river joins the Columbia River at Richland. The river is regulated by a system of dams to provide irrigation. Keechelus, Kachess, Cle Elum, Rimrock, Clear Lake and Bumping reservoirs capture about one third (1.1 million acre-feet) of the basin's average runoff. The dams have reduced the frequency and magnitude of floods that formerly inundated the floodplain (Washington Department of Natural Resources, 2004). Water managers release water until early September. "Flip flop" occurs when the dams on the Yakima River are ramped down and dam releases increase on the Naches River to accommodate spring Chinook salmon spawning.

The Yakima River experiences rain on snow-generated floods during the winter as well as snowmelt-generated floods in the spring. Winter floods generally produce higher peak flows but their duration is typically shorter.

There are several gages on the Yakima River in the study area that have been operated by the US Geological Survey (USGS) and the USBR. Figure 6 shows the locations of the gages; Table 1 summarizes the periods of operation. The closest active USGS gage to the study reach is Yakima River above Ahtanum Creek (12500450), which is 3.5 miles south of the SR24 Bridge and has a drainage area of 3,479 square miles.

The previous SRH-1D sediment modeling studies conducted by the USBR (Hilldale and Godaire, 2010) used input data from the Yakima River at Umtanum and Naches River below Tieton River, after correcting for diversions and local inflows. The combined flows are approximately the same as the recorded flows at Yakima River above Ahtanum Creek and Yakima River at Parker.

Figure 7 shows the time series of peak discharges on the Yakima River near Parker and Yakima River above Ahtanum Creek. The time series of annual peak flows exhibits a marked cyclical pattern that was attributed by Hilldale and Godaire (2010) to fluctuating climatic conditions induced by the Pacific Decadal Oscillations (PDO). Characteristic periods of “wet” and “dry” years were identified:

- Wet Periods: 1909-1912, 1931-1938, 1945-1952, 1974-1982, 2006 to present
- Dry Periods: 1960-1972, 1982-1990, 1999-2004

Since 1894 the flow in the Yakima River has exceeded flood level 47 times. Since 1970, the river was declared a federal disaster area due to flooding eight times in 27 years, most recently in 1990, 1995, 1996 and 1997 (YCFHMP, 2007). The highest floods occurred in 1933, 1972, 1974 and 1996. The largest floods, including the flood of record in December 1993, are the result of winter rain-on-snow events. The December 23, 1933 flood had an estimated instantaneous maximum discharge of 65,000 cfs at Parker. The flood of February 9, 1996 reached a peak discharge of 53,300 cfs. These values are reported as instantaneous maximum discharges. The corresponding average daily discharges are typically about 10% less than the instantaneous maximum values. YCFHMP (2007) considered the 1933 flood to be a 200-year event and the 1996 flood a 100-year event.

Table 2 summarizes the adopted flood frequency estimates of peak discharges at the study reach that were provided to NHC by Yakima County.

Table 1: Key gages on Yakima River in study reach

Gage ID	Gage Name	Start Date	End Date	Drainage Area (square miles)
12500450	Yakima R. above Ahtanum Cr.	1966	Present	3,479
12505000	Yakima R. near Parker	1908	Present	3,660
12503000	Yakima R. at Union Gap	1898	1966	3,652
12484500	Yakima R. at Umtanum, WA	1908	Present	1,594
12494000	Naches R. below Tieton R	1908	Present	941

Table 2: Adopted flood frequency estimates on Yakima River

Return Period (Years)	Discharge (cfs)
2	16,150
5	22,100
10	26,600
100	57,300

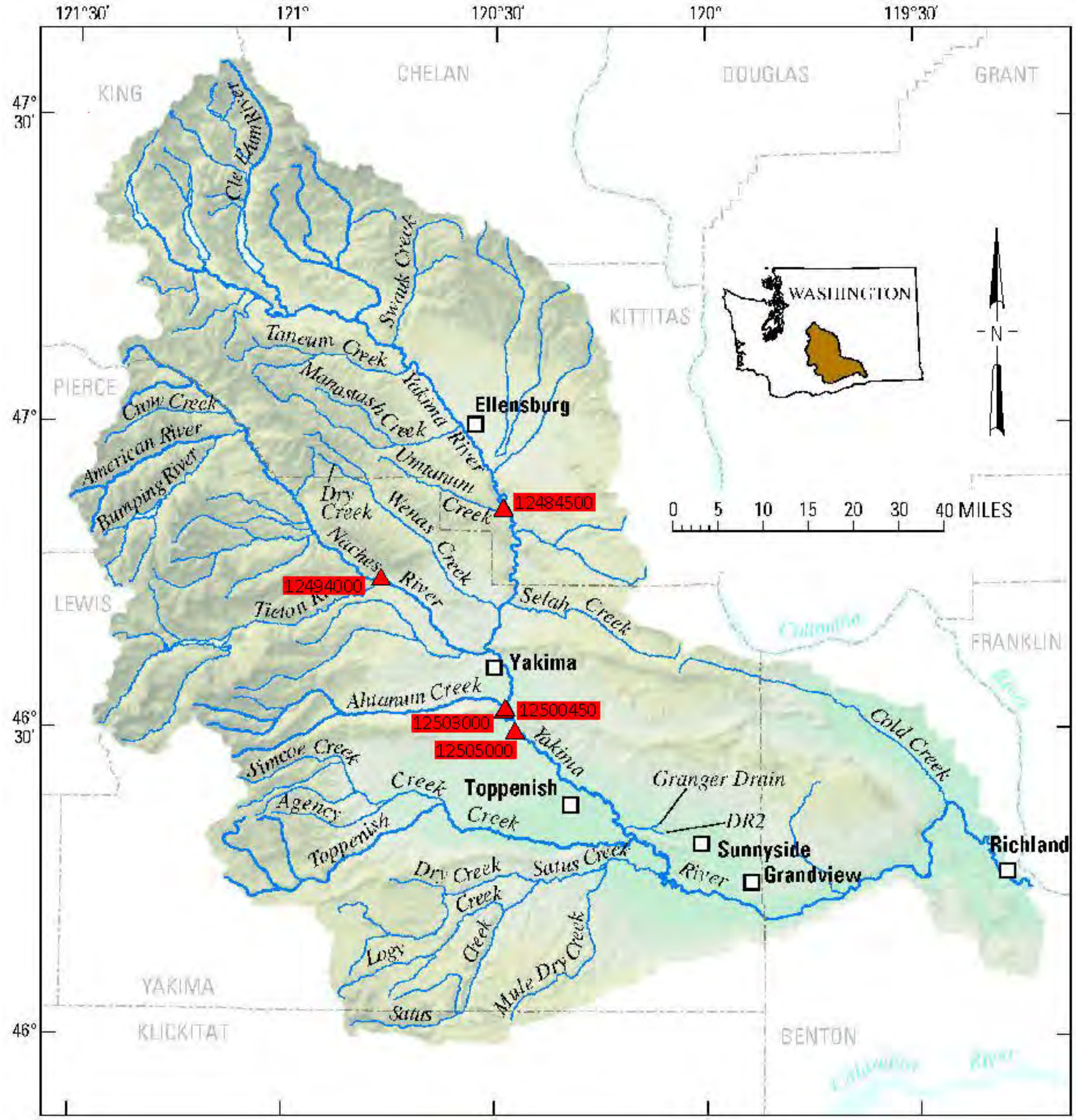

Figure 6: Surface water gaging stations in Yakima basin

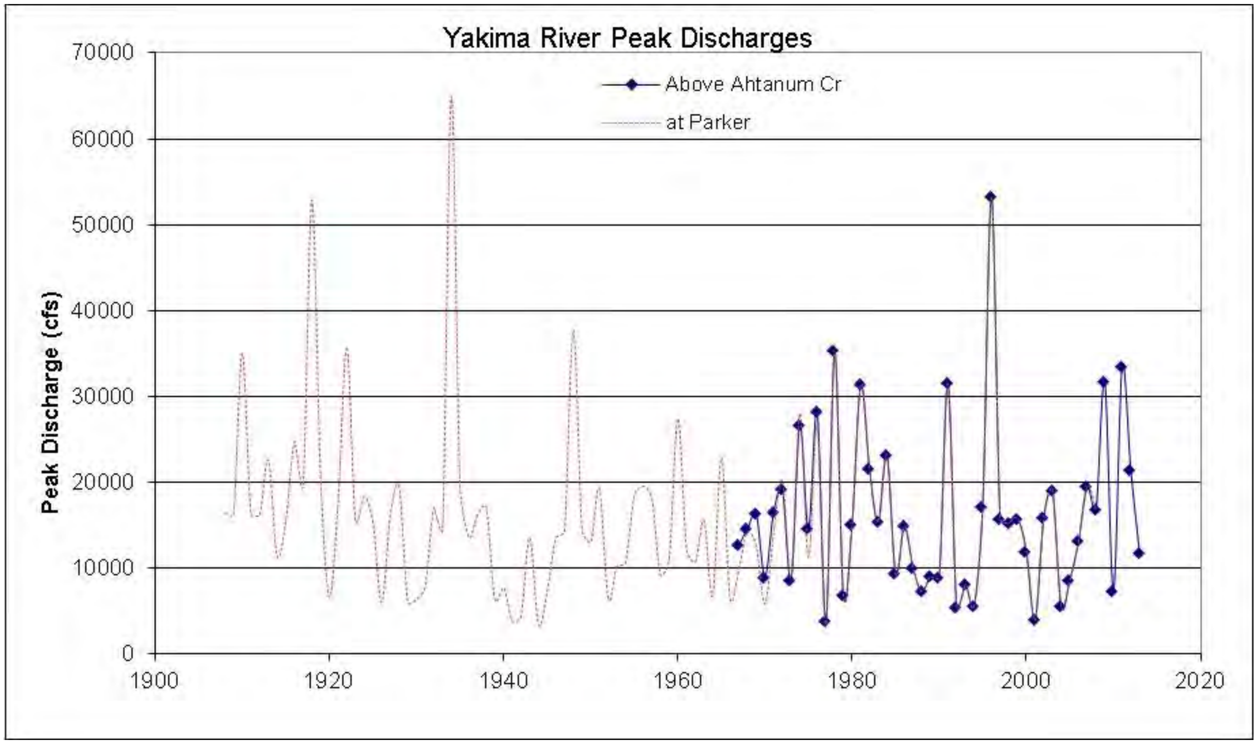

Figure 7: Time series of peak (instantaneous) discharges recorded on Yakima River

2.3 Sediment Loads

Dunne et al. (1976) assessed the sediment transport characteristics of the Yakima River as part of planning studies undertaken for the Yakima River Regional Greenway (Jones and Jones, 1976). The major sources of sediment available to the Yakima River are from erosion of the mountain slopes and glacial and fluvio-glacial sediments in the Upper Yakima and Naches River basins. Abundant supplies of sand and gravel are also available in the Pliocene and Pleistocene sediments on the floor of the valley itself. The sediment load on the Yakima River was estimated using suspended sediment data collected at the USGS Parker gage. The authors estimated the total suspended load was 183,000 tons/year. The bed load was estimated by formula at approximately 57,000 tons/year.

The sedimentation studies on the Gap to Gap Reach conducted by the USBR included limited bedload and suspended load measurements by the USGS in 2008 on the Naches and Yakima Rivers (Hilldale and Godaire, 2010). Table 3 summarizes the measured sand and gravel transport rates during moderately high flow conditions on the Naches River at the I-82 Bridge (12499010) and Yakima River at the I-82 Bridge (12487010). These measurements indicated most of the sand and gravel load in the project reach is supplied by the Naches River.

Annual bed material loads on the Yakima River were estimated by Hilldale and Godaire (2010) using the SRH-1D model. The model predictions averaged 19,000 tons/year for the period 1985 to 2009. Other estimates were made using observed suspended load measurements and assuming the bed material load constituted 25% of the total load. This result indicated a total bed material load of 24,000 tons/year (roughly half of Dunne's previous estimate). The authors assumed that wash load consists of material finer than 0.063 mm (silt and clay) and bed material load consists of all material coarser than 0.063 mm (sand and gravel) is debatable. Comparing the sub-surface bed material samples in Table 4 with the size distribution of the loads in Table 3 strongly suggests that much of the sand load on the river is supply-limited and behaves as wash load.

Table 3: Summary of USGS bedload and suspended load measurements in 2008

River	Date	Q	Bed Load			Suspended Load		Bed Material Load			
		cfs	GB ton/day	GB sand ton/day	GB gravel ton/day	GSS ton/day	GSS sand ton/day	Gsand ton/day	Gsnd+grav ton/day	%Sand	%Gravel
Naches	May-17	8250	1310	524	786	10300	4120	4644	5430	86%	14%
River	May-18	9300	1430	572	858	14100	4794	5366	6224	86%	14%
	May-20	8290	772	355	417	4440	1554	1909	2326	82%	18%
Yakima	May-17	6710	72	54	18	4120	659	713	731	98%	2%
us of	May-18	6170	81	60	21	3350	503	562	584	96%	4%
confluence	May-20	6130	162	104	58	1430	172	275	334	83%	17%
Yakima	May-17	14960	1382	578	804	14420	4779	5357	6161	87%	13%
ds of	May-18	15470	1511	632	879	17450	5297	5928	6808	87%	13%
confluence	May-20	14420	934	459	475	5870	1726	2184	2660	82%	18%

2.4 Hydraulic and Morphological Characteristics

2.4.1 General Features

Yakima River is a wandering gravel bed river with a split or anabranching channel pattern. The water surface slope at flood stage averages 0.0025 (13.2 feet/mile) and the river generally has a low sinuosity (1.25). Table 4 summarizes surface and sub-surface bed material samples from channel bars (from Hilldale and Godaire, 2010).

Table 4: Surface and sub-surface grain size characteristics (from Hilldale and Godaire, 2010)

River Mile	Distance from SR24 Bridge (Miles)	Surface Grain Size (mm)			Sub-surface Grain Size (mm)		
		D84	D50	D16	D84	D50	D16
111.5	1.6 DS	77	45	17	65	29	3.9
113.3	0.2 US	112	82	53	122	48	6.6
115.0	1.9 US	63	40	27	67	43	27

2.4.2 Hydraulic Geometry

Empirical hydraulic geometry charts developed by Neill and published in USACE (1994) and recent analytical relations developed by Parker (Chapter 3.4 in ASCE, 2008) were used to predict the bankfull hydraulic properties for single-thread channels and for branched channel segments. The Parker relations are based on field data from gravel-bed rivers but have been generalized by expressing the relations in non-dimensional form. Hydraulic properties (mean depth h and top width at bankfull stage, B) were expressed in terms of the discharge and bed material size:

$$\hat{H} = 3.01\hat{Q}^{0.405} \text{ and } \hat{B} = 4.87\hat{Q}^{0.461} \qquad \text{Eq. 1}$$

$$\hat{H} = \frac{H}{D_{50}}, \hat{B} = \frac{B}{D_{50}}, \hat{Q} = \frac{Q}{\sqrt{gD_{50}}}\frac{1}{D_{50}} \qquad \text{Eq. 2}$$

where H is the mean depth, B is the top width, Q is the discharge and D_{50} is the median grain size.

Results are summarized in Table 5. It was assumed the "channel-forming" discharge corresponds closely to a 2-year flood flow (Table 2).

Table 5: Predicted bankfull hydraulic geometry of channels on Yakima River

	Top Width (feet)	Mean Depth (feet)	Mean Velocity (feet/sec)
Single thread channel Q_{bf} = 16,150 cfs	270	9.5	6.3
2 branched channels Q_{bf}= 8,075 cfs	190	7.1	6.0

The results from Parker (ASCE 2008) are virtually identical to the results from the empirical methods in USACE (1994).

Using the 100-year discharge, the natural floodway opening that is required to pass extreme flood flows is approximately 500 feet. Most of the river is presently much narrower than the natural regime geometry due to channelization from the past levee construction.

3 FLOOD CONTROL LEVEES AND CHANNELIZATION

3.1.1 History of Levee Construction

Figure 1 shows the existing and proposed levees in the study reach. Information on the history and performance of these structures was compiled from the Upper Yakima River Comprehensive Flood Hazard Management Plan (Otak-KCM, 2007) and information provided by Yakima County.

Designs for a Federal levee system on the Yakima River were carried out in the 1930s but were not implemented until the end of World War II (Otak-KCM, 2007). The project was completed in 1948 and repaired and extended after the 1948 flood. The levees were raised in the 1970s to allow them to be accredited by FEMA for the 100-year flood. The Wastewater Treatment Plant Levee, built in 1958, has been re-constructed eight times due to scour at the toe of the levee. There were three major flood events during the 1990s culminating in the flood of 1996. During this flood emergency reinforcement was carried out at several locations, particularly:

- The west bank levee at Buchanan Lake
- The east bank levee immediately downstream of the Terrace Heights Bridge
- The DID #1 levee immediately downstream of SR24 Bridge.

3.1.2 Present Status of Levees

The existing Federal levee located on the west bank extends 8,500 feet from the Terrace Heights Bridge downstream to the historic SR24 Bridge (Figure 1). A 3,000 foot long segment of the levee forms a narrow barrier between Buchanan Lake and the river to the immediate east. The upstream 600 feet of this segment is exposed to direct attack by the river and is vulnerable to scour along the toe. The downstream end of the Federal levee terminates approximately 800 feet upstream of the new SR24 Bridge abutment. The former SR24 Bridge approach embankment is the only feature that provides flood protection in this 800 foot gap. A 350 foot segment of the Federal levee, adjacent to the Nob Hill Wrecking Yard, will be removed as part of this project and a set-back levee will be constructed connecting the former SR24 Bridge approach embankment and the Federal levee.

On the east bank upstream of the SR24 Bridge, a new 3,500 foot long Federal set-back levee was constructed in 2010 by the Corps (Figure 1). The former levee along this reach remains in-place but is being actively undermined by bank erosion as the river migrates eastwards (Photo 1). The former levee will be removed in conjunction with other planned downstream modifications.

The east abutment of the new SR24 Bridge is set back 830 feet from the existing river bank (Figure 1). The length of the bridge opening is 1,350 feet measured perpendicular to the flow. However, at present, the net waterway opening is only 500 feet, due to a cross-dike at the upstream end of the DID #1 levee which blocks the left floodplain. The cross-dike is a temporary, armored earth-fill structure that was placed along the upstream side of the bridge in 2010 (Photo 3) when the upstream Federal levee set-back was constructed. The cross-dike is intended to prevent overbank flow from outflanking the downstream DID #1 levee. The cross-dike will be removed as part of future downstream levee modifications.

The existing DID #1 levee extends 9,400 feet downstream of the SR24 Bridge along the left bank of the Yakima River. The Newland Ponds are located in the floodplain immediately landward of the upper 3,700 foot segment of the levee (Photo 5). Approximately 1,200 feet of this levee segment extends along the outer (concave) bank of a meander and is exposed to direct attack by the river (Photos 7 and 8).

The new DID #1 set-back levee will be located 1,200 to 2,700 feet landward of the existing structure and consist of two segments. Based on a 10% Submittal prepared by the Corps, the first segment will be 2,800 feet long and the second 7,800 feet. However, the total effective length of the levee will only be 9,300 feet, as the two segments will overlap by 1,300 feet.

3.2 Floodplain Changes

The hydrologic, hydraulic and geomorphic characteristics on the Yakima River have been altered as a result of flow regulation, channelization and gravel mining. Eitemiller et al. (2002) summarize the history of anthropogenic alterations to the floodplain of the Yakima River in the Gap to Gap Reach. Their study shows that human impacts to riverine ecosystems have severed the physical connectivity of the river during the past 120 years. Railroads, roads and levees have dissected the river landscape. The analysis shows a reduction in riverine habitat once inundated by small, frequent flood events. Eitemiller et al. characterized the reduction in floodplain area surficially connected to the Yakima River (Table 6). The pre-settlement Gap to Gap Holocene floodplain encompassed 2,325 hectares. Currently, only about 40% of the Gap to Gap Holocene floodplain remains surficially connected to the Yakima River.

Table 6: Loss of connected floodplain area in Gap to Gap reach

Date	Surficially Connected Floodplain Area	
	(hectares)	(acres)
Pre-settlement	2325	5745
1915	1840	4546
1964	1592	3933
2002	921	2276

Furthermore, the hydrologically active portion of the floodplain, defined as areas recently exposed to flow (as interpreted from aerial photos) was reduced from 808 hectares in 1927 to 584 hectares in 2002, a loss of approximately 28% (Figure 8).

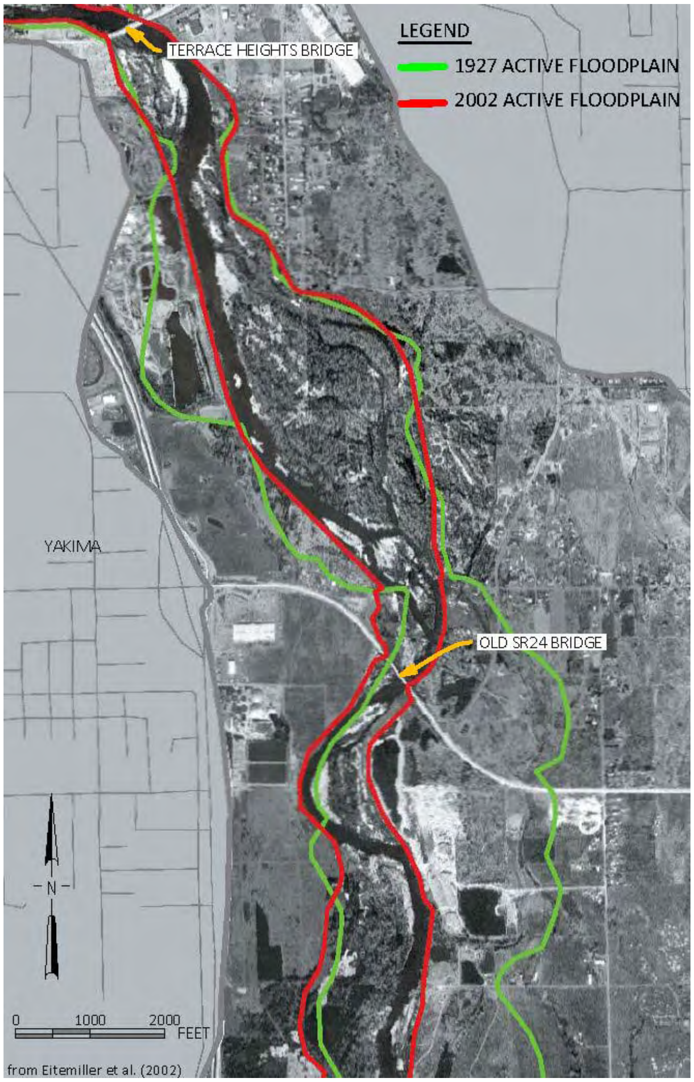

Figure 8: Change in hydrologically active floodplain area from 1927 to 2002

The authors also noted that the overall length of main stem channel on the Yakima River has decreased by 43% over the period of analysis. This difference was the result of the extensive system of levees built by the Corps that protects infrastructure behind the flood revetments.

3.3 Channel Pattern Changes

The river has been channelized over time, being converted from a relatively complex, multi-channel system in the 1920s into a more confined and frequently single-channel system by the 1960s.

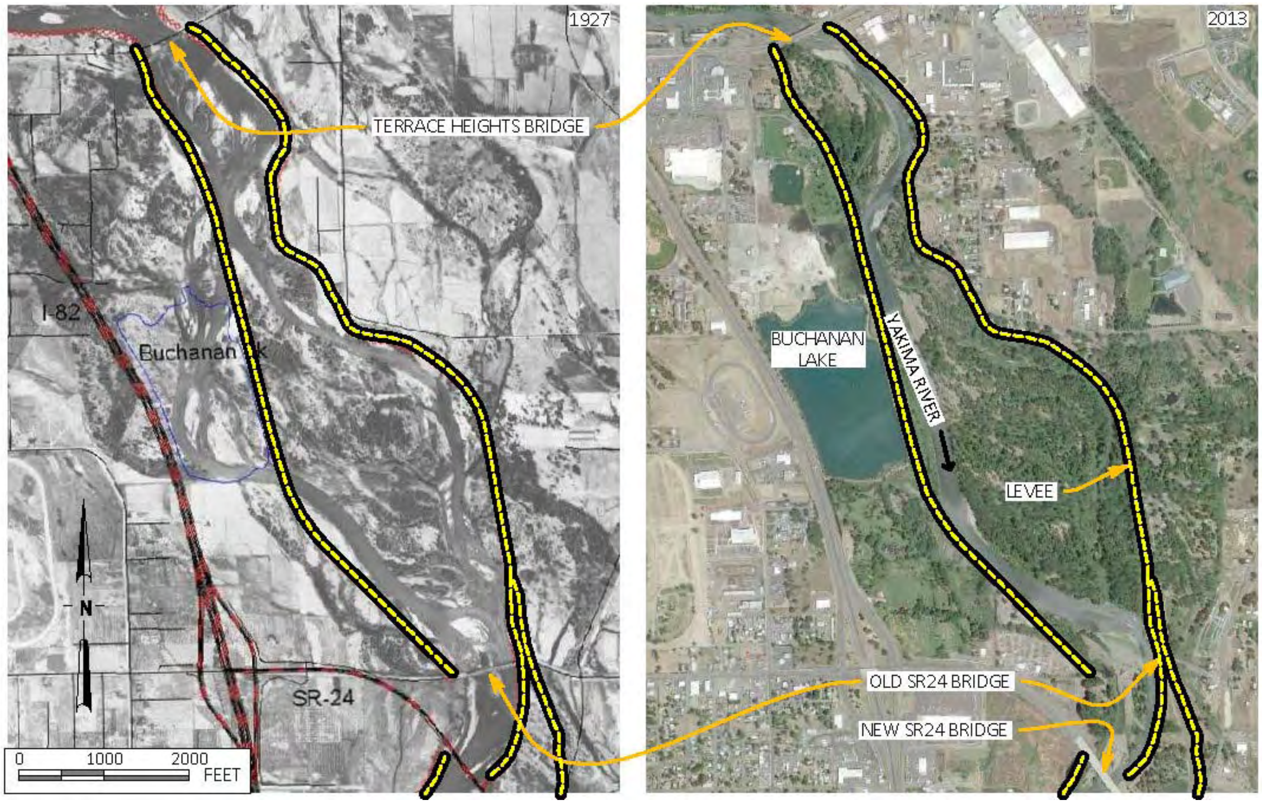

Figure 9: 1927 air photo overlain with present infrastructure and 2013 image

Figure 10: Buchanan Lake-Sportsman's Park reach 1996 to 2013

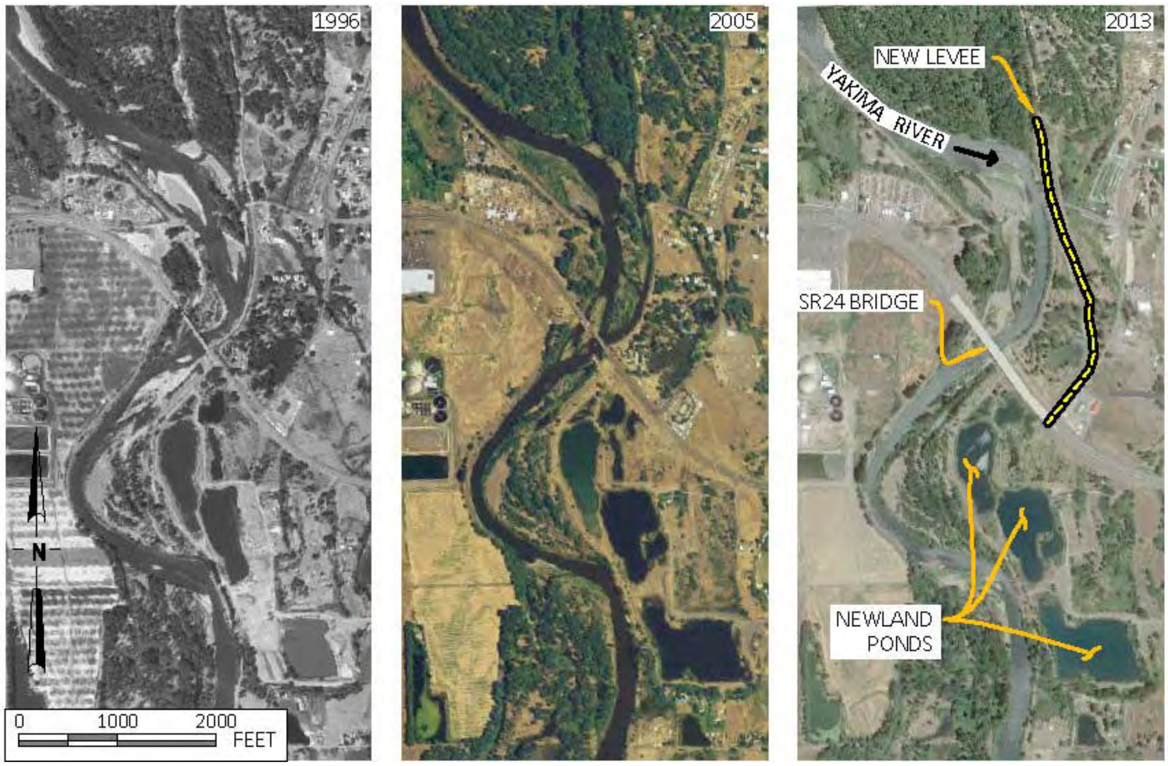

Figure 11: SR24 Bridge Reach 1996, 2005 and 2013

3.4 Changes to Channel Complexity

Clark (2003) assessed the historical changes using a procedure termed “nodal analysis”. Nodal analysis involves identifying and counting the number of distinct anabranch nodes in a channel for specified time periods and within specified reach zones. The number of nodes is a measure of the complexity of the channel and changes to the complexity of the channel pattern are inferred from the changes to the number of nodes over time. The analysis was replicated for the Gap to Gap Reach by Hilldale and Godaire (2010) using air photos and maps dating back to 1927. The analysis showed the largest number of nodes and greatest channel complexity was in 1927 (Figure 12). The channel complexity decreased to a minimum in 1962, reflecting the levee construction and gravel mining that channelized the river system. Some recovery of the channel complexity has occurred since the 1960s. A more detailed analysis by sub-reaches showed some of the greatest reduction in complexity occurred in the reach between the Terrace Heights Bridge down to near the SR24 Bridge (Reach 3A in Hilldale and Godaire, 2010) and from below the SR24 Bridge to the Edler Ponds area (Reach 4). This area corresponds to the reach where mitigation measures are proposed in Chapter 7.

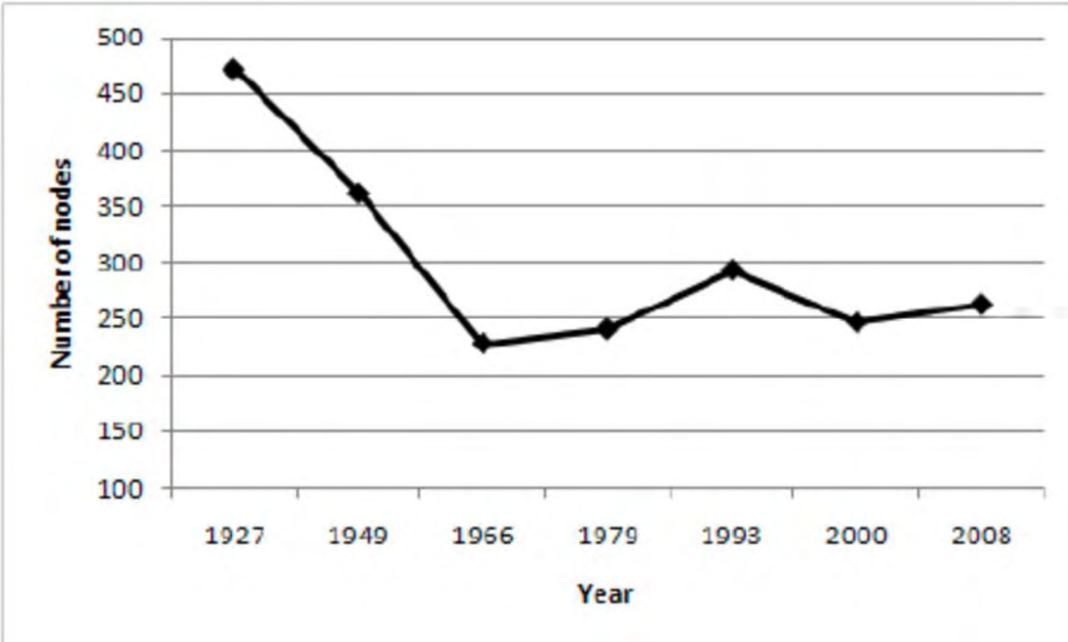

Figure 12: Inferred decrease in channel complexity of the Yakima River based on a nodal analysis as modified from Clark (2003)

3.5 Vertical Changes Due to Channelization

3.5.1 Overview of Effects

Channel straightening often leads to increase in bed gradient and local increases in sediment transport which can lead to incision and the development of uniform, trapezoidal-shaped channels devoid of typical pool-riffle sequences (USACE, 1994). When the flow is concentrated against armored levees, the uniform bankline and deeper scour often causes the channel to become attached to the bank, reducing the incidence of channel migration and often promoting deposition in the mid-channel region between the levees. Channel widening and sediment deposition zones may develop upstream and downstream of the channelized reach as illustrated in Figure 13. The upstream deposition zone occurs in response to the backwater produced by the narrowed channel. The downstream deposition zone occurs in response to excess sediment transport and scour through the channelized reach.

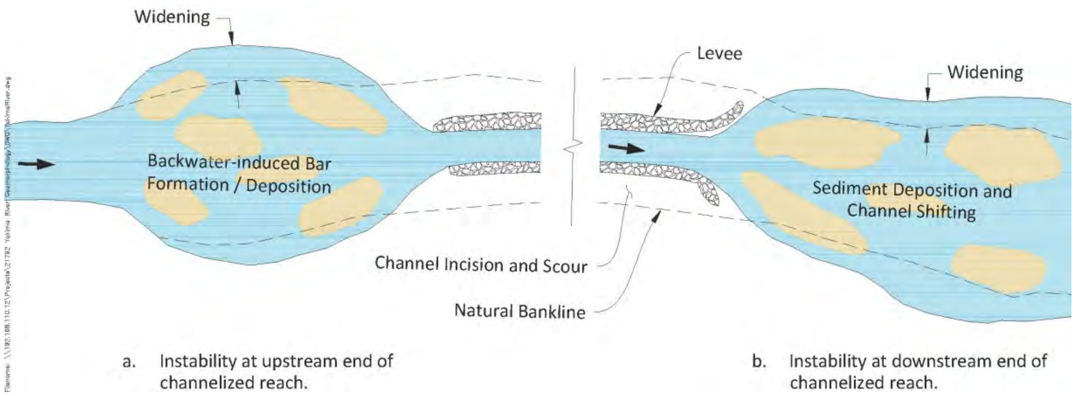

Figure 13: Local sedimentation zones upstream and downstream of a channelized reach

3.5.2 Effect of Channelization on Hydraulic Geometry

The effect of narrowing the river on channel scour was estimated using the Blench regime equation, which expresses the mean depth (h) when the discharge (Q) and channel width (B) are both imposed. The maximum depth (h_{max}) is expressed as:

$$h_{max} = Z\left[\frac{q^2}{F_b}\right]^{1/3} \text{ where} \qquad \text{Eq. 3}$$

Z is an empirical scour depth ratio that typically varies between 1.7 and 2.5 in bends;

q is the discharge intensity, expressed as discharge per unit width (Q/B);

F_b is an empirical bed factor that is related to sediment size and sediment transport rate.

In a relatively coarse-grained river such as the Yakima with a low bedload transport rate, a representative value of F_b is approximately 5.

WSAHGP (2003) provides an overview of the equation and its application to channel restoration design. Table 7 demonstrates the effect of confining a natural, single thread channel (top width of 270 feet) into 150 foot wide and 100 foot wide channelized sections. Narrowing the channel from 270 feet to 150 would increase the mean depth from 9.5 feet to 13.2 feet under a 2-year flood condition, and would increase the mean velocity from 6.3 feet/sec to 8.1 feet/sec. Narrowing the channel from 270 feet to 100 feet would increase the maximum depth from 9.5 feet to 17.3 feet and would increase the mean velocity from 6.3 feet/sec to 9.3 feet/sec.

Table 7: Effect of channelization on hydraulic geometry-case of single thread channel

Return Period	Q (cfs)	Single thread natural channel W= 270 ft V (ft/sec)	h (ft)	hmax (ft)	Confined single thread channel W = 150 ft V (ft/sec)	h (ft)	hmax (ft)	Confined single thread channel W= 100 ft V (ft/sec)	h (ft)	hmax (ft)
2-year	16150	6.3	9.5	19.0	8.1	13.2	26.5	9.3	17.3	34.7
5-year	22100	7.4	11.0	22.0	9.0	16.3	32.6	10.3	21.4	42.7
10-year	26600	7.9	12.5	24.9	9.6	18.5	36.9	11.0	24.2	48.4

Table 8 shows the comparable effects of channelizing a reach consisting of two 190 feet wide branched channels. These results demonstrate that the effect of converting a branched channel system into a narrower single channel has an even greater impact on the channel depths and velocities than the case of a single channel reach.

Table 8: Effect of channelization on hydraulic geometry-case of a natural channel in 2 branches

Return Period	Q (cfs)	Natural channel in 2 branches Total W= 380 ft V (ft/sec)	h (ft)	hmax (ft)	Confined single thread channel W = 150 ft V (ft/sec)	h (ft)	hmax (ft)	Confined single thread channel W= 100 ft V (ft/sec)	h (ft)	hmax (ft)
2-year	16150	6.0	7.1	14.2	8.1	13.2	26.5	9.3	17.3	34.7
5-year	22100	9.3	8.8	17.6	9.0	16.3	32.6	10.3	21.4	42.7
10-year	26600	9.9	9.9	19.9	9.6	18.5	36.9	11.0	24.2	48.4

These increases in velocity and channel depth also have a direct impact on the risk to infrastructure such as bridge abutments and levee revetments. The maximum depth in a channelized reach at a bend or zone of flow impingement can be over two times the mean depth (Z=2.0). In this case, the depth of scour may be increased by over 20 feet (Table 8).

3.5.3 Observed Channel Changes

A comparison of cross section surveys between 1969 and 2005 by Hilldale and Godaire (2010) showed highly variable results, which made it difficult to come to any firm conclusions about vertical changes along the river. Degradation was noted near Buchanan Lake in response to channelization works. However, changes near the SR24 Bridge were variable and not consistent.

Thalweg profile plots using data from 1954, 1969 and 2005 were also inconclusive in showing any systematic trend in aggradation or degradation over the entire Gap to Gap Reach. However, these comparisons appear to indicate systematic lowering has occurred in the reach between the SR24 Bridge and Terrace Heights Bridge (Figure 14). The distinct and localized 10 foot bed lowering approximately 7,500 feet upstream of SR24 can most likely be attributed to scour at the west bank Federal levee, when the channel became locked against the levee in the early 1970s.

It should be noted that the Corps channelized the section at the railroad bridge in 1969 through excavation of material from the channel. Also, Yakima County indicated that Washington State Department of Transportation (WSDOT) removed large quantities of material in the late 1960s immediately downstream.

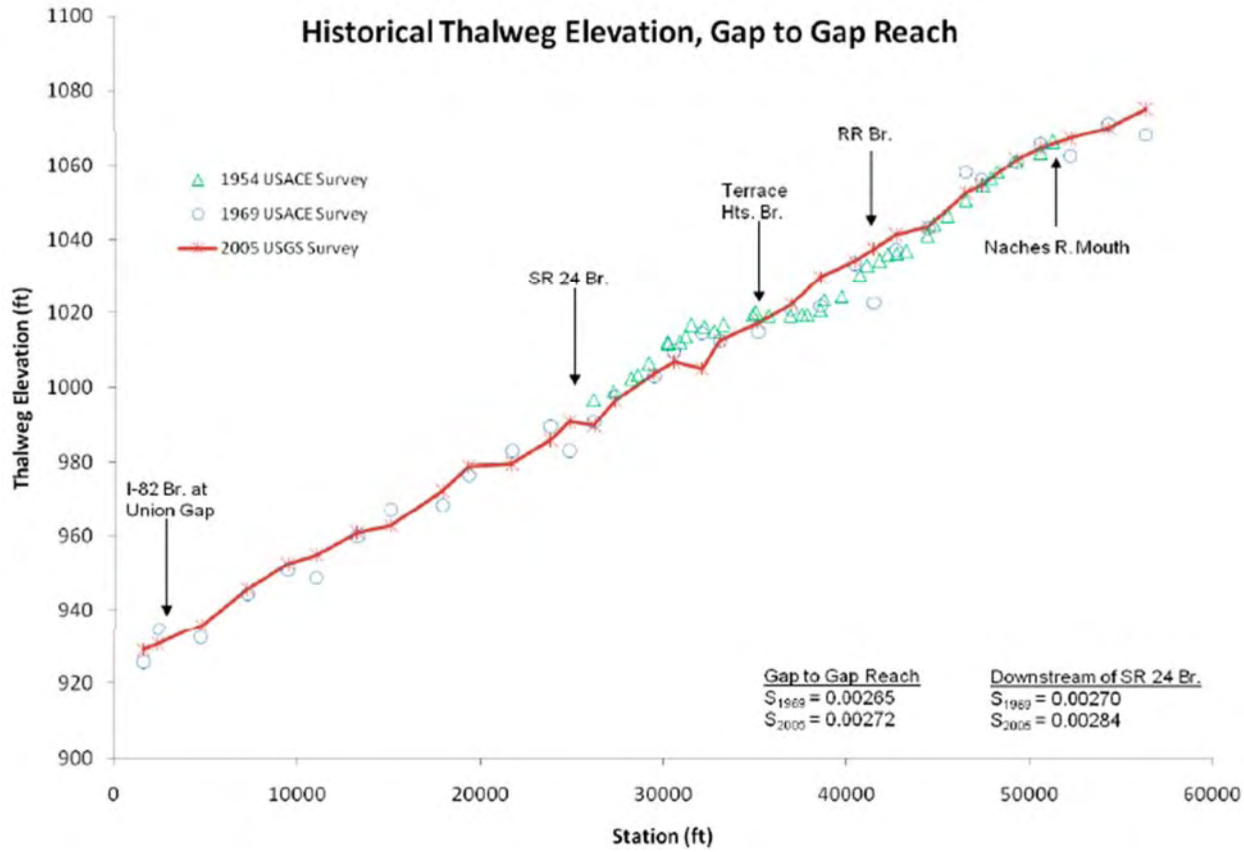

Figure 14: Longitudinal profile of Yakima River

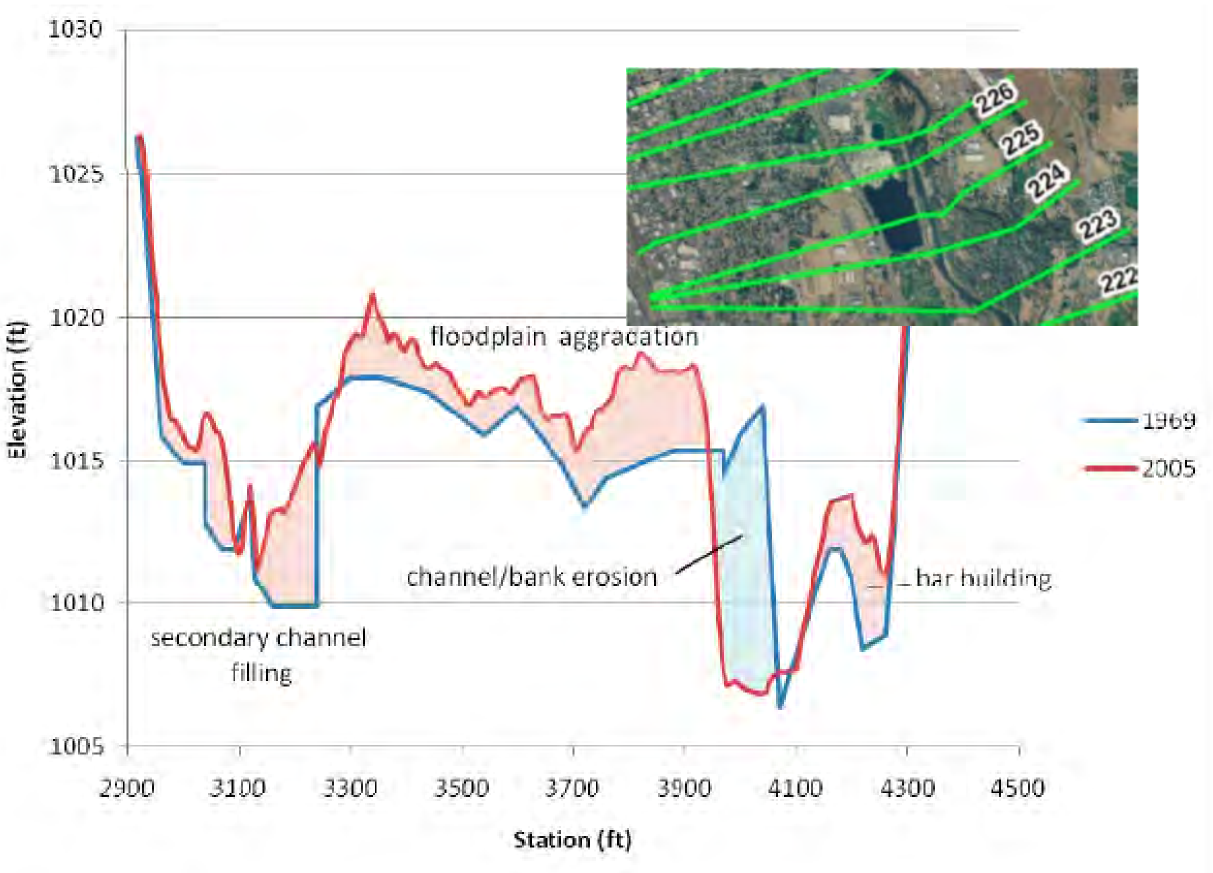

Figure 15: Cross section 225 1969 to 2005 from Hilldale and Godaire (2010).

4 EFFECT OF PIT CAPTURES ON RIVER MORPHOLOGY

4.1 Gravel Mining in Project Reach

Information on the history of mining activities is summarized in Dunne et al. (1976), Clark (2003), Washington Department of Natural Resources (2004) and Hilldale and Godaire (2010). Gravel mining has been carried out on the floodplain of the Yakima River for more than 100 years but was most intensive in the 1950s when state and interstate highways were being constructed. The effects of mining vary, depending on where the pits are located, their geometry (i.e. surface area, depth and shape), distance from the active channel, and whether structural measures are in-place to prevent the river from breaching banks and flowing into the pits. There are primarily two existing gravel mining sites that potentially affect the stability of the river in the project area:

- Newland Ponds, situated on the east bank of the river downstream of the SR24 Bridge.
- Buchanan Lake (Beech Street gravel pit) situated on the west bank of the river, upstream of the SR24 Bridge.

The Newland Ponds, developed and still under lease to Central Pre-Mix Concrete Co, are situated on the east bank of the Yakima River downstream of the SR24 Bridge. Gravel mining has been active at the site for more than 50 years (YRFMIST, 2004). The three ponds were surveyed by NHC in 2014. **Figure 17** shows the topography of the ponds as well as the adjacent floodplain and river channel. The ponds are not connected to the Yakima River and are isolated from the river by the DID #1 levee.

Table 9 summarizes the depth and volume of the ponds. The most upstream pond (Pond 1) has a maximum depth of 29.6 feet and accounts for about 12% of the total volume of the three ponds. The largest pond (Pond 3) accounts for 50% of the total volume.

Table 9: Geometry of the Newland Floodplain Ponds

Pond #	Average Depth (ft)	Maximum Depth (ft)	Invert (ft)	Area (acres)	Volume (yd^3)
1	16	29.6	959.7	6.7	176,400
2	31	59.3	931.5	11.2	558,000
3	31	72.7	933.7	14.4	721,059
			Total	32	1,455,459

The lowest (thalweg) bed levels in the Yakima River vary from elevation 985.3 feet opposite Pond 1, 977.7 feet near Pond 2 and 981.1 feet opposite Pond 3 (Figure 16). Therefore, the ponds extend from 25 to 47 feet lower than the thalweg of the river.

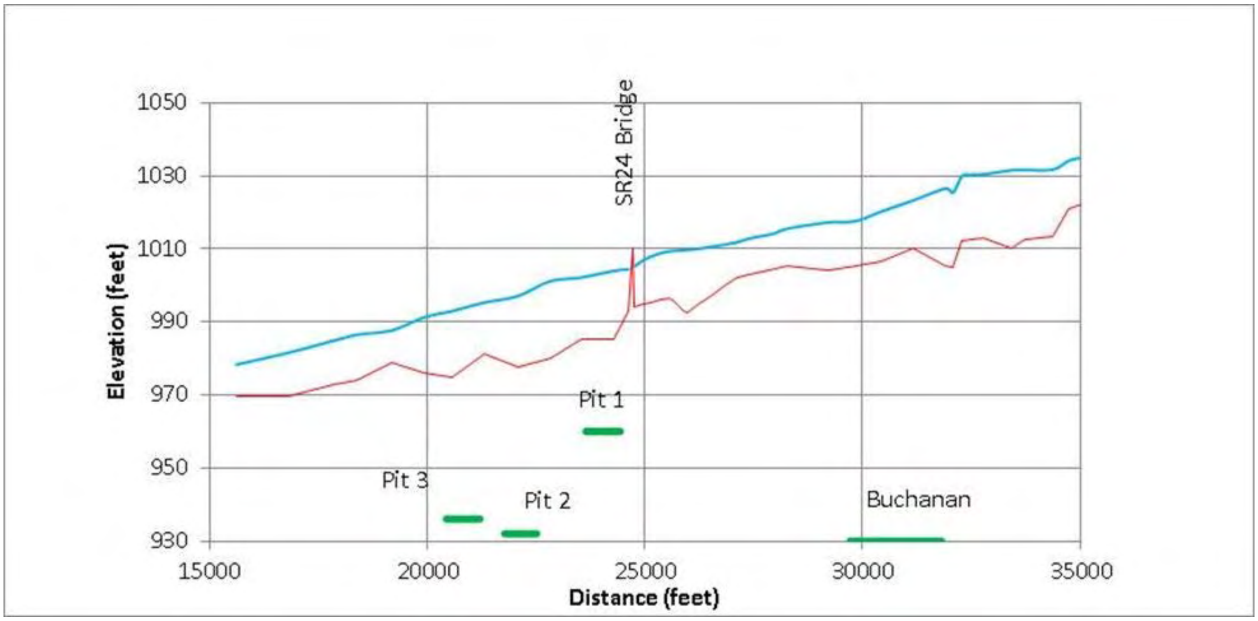

Figure 16: Comparison of pond invert levels and river bed profile

Buchanan Lake is the site of the former Beech Street gravel pit and is located 1 mile upstream of the SR24 Bridge on the west side of the Yakima River (Figure 1). The pond covers an area of approximately 60 acres and is reported to have a depth of 120 feet below the floodplain and nearly 100 feet lower than the thalweg of the adjacent river channel. The pond is separated from the Yakima River by the Federal levee.

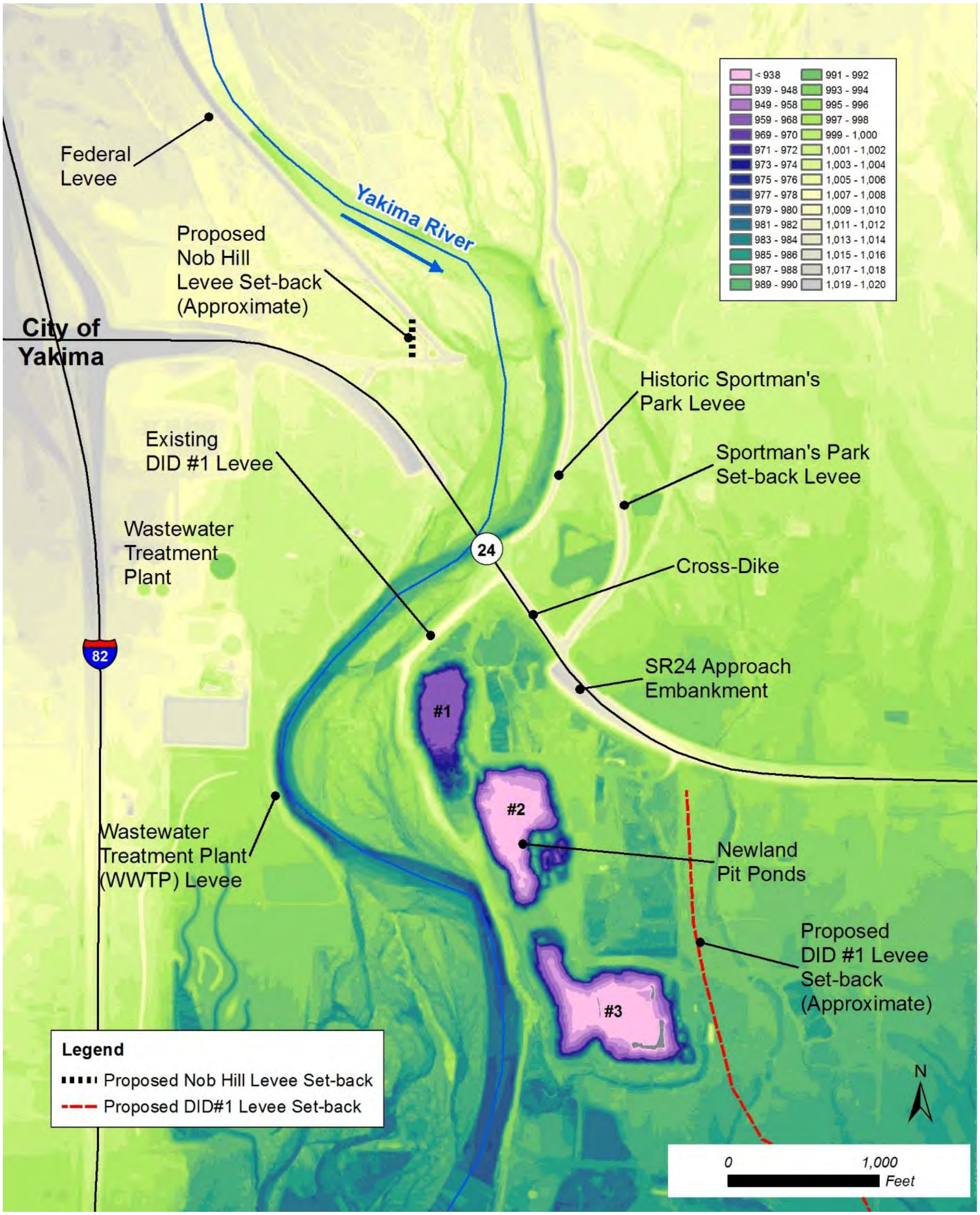

Figure 17: Gravel pits on floodplain downstream of SR24 Bridge

4.2 Review of Floodplain Gravel Mining Impacts

A substantial body of literature exists describing the effects of alluvium mining, from both the active channel and floodplain. Removal of bed material directly from an alluvial river steepens the channel upstream and reduces or removes the bedload sediment supply to the downstream channel. These initial changes commonly result in upstream incision due to headcutting, downstream incision due to bedload starvation, and coarsening of the bed material. These processes have been described in several reviews including those by, among others, Kondolf (1994b, 1997), Norman et al. (1998) and Rinaldi et al. (2005), who document over 20 individual case studies describing these effects.

Floodplain pit capture occurs when the mining has been adjacent to the river and gradual channel migration or avulsion intersects a pit, rather than the more commonly documented case of initial removal of sediment from within the active channel. The effect of floodplain pit capture is similar to that from in-channel excavation. This is because the process of capture converts the formerly off-channel pit to an in-channel pit. Numerous examples of floodplain pit capture have been documented in the literature. Selected examples are described in Appendix A.

The focus of most of these studies has been immediate (within 1 or 2 years) effects of avulsions through floodplain pits. Long term monitoring of channel response to these avulsions as well as an evaluation of floodplain pit intersections that did not cause avulsion have been relatively limited. These examples provide a dataset that will be used in later sections of this memo to develop predictive relations for channel response to floodplain pit capture and to provide a starting place for a description and evaluation of long term geomorphic response in the case studies section.

4.3 Styles of Pit Captures

There are two general types of processes responsible for gravel pit capture: avulsion and lateral connection. Avulsions occur when the channel rapidly shifts course through the floodplain pit. Avulsions occur when a shorter, steeper, less obstructed path is available for the channel than the one occupied prior to the avulsion. Avulsion can either form a new side-channel or completely abandon its previous course. The second type of capture is through lateral connection, which occurs when the channel migrates (laterally), intercepts the perimeter of a pit, and widens locally but remains with essentially the same planform alignment. Avulsions through floodplain pits typically create much greater geomorphic disturbance than lateral connection.

4.3.1 Avulsions

Documented avulsions through floodplain pits have caused major channel change, creating potential hazards to infrastructure and ecological function of the channel. These impacts are caused by accelerated lateral channel migration that can threaten levees as well as bed degradation that can enhance scour around bridge piers and at revetment toes. Deleterious habitat impacts include direct loss of existing habitat when a full avulsion occurs, changes to water temperature and chemistry, changes to aquatic species composition, and disorientation of out-migrating juvenile salmonids.

A river channel avulsion through a floodplain pit triggers a sequence of predictable processes and geomorphic responses. This sequence is reflected in many of the published case studies described in Appendix A (Table 1) and in several published documents (Bull and Scott, 1974; Dunne et al., 1980; Collins and Dunne, 1990; Kondolf, 1994a, 1994b, 1997; Norman et al., 1998; Wampler et al., 2007; Hilldale and Godaire, 2010). Findings from these studies provide the basis for the following conceptual model of geomorphic response to typical floodplain pit capture scenarios.

The first scenario focuses on an avulsion through a floodplain pit. Prior to pit capture, at time t0, the meander bend in the channel forms a relatively long path around the pit (Figure 18). A flood overflow or channel migration breaches the levee separating the upstream portion of the pit from the channel, allowing water to flow through the shorter, steeper path (Figure 18, t1), The pit, which would have a nearly flat water surface profile, causes the water surface in the upstream portion of the overflow path to become locally steeper. This over steepening of the water surface creates very high shear stress on the bed, which causes a headcut to form and the bed upstream of the pit to erode (Figure 18, t2). In a coarse gravel bed river the reach recovers (Figure 18, t3) when the upstream propagation of the headcut is halted by bed coarsening, bedload supply, or intersection with an existing grade control feature like a coarse riffle. The pit captures the supply of bedload moving downstream, forming a delta that progrades across the floodplain pit until a stable bed slope and channel cross section has reformed through the pit. At this time bedload supply is restored to the reach downstream.

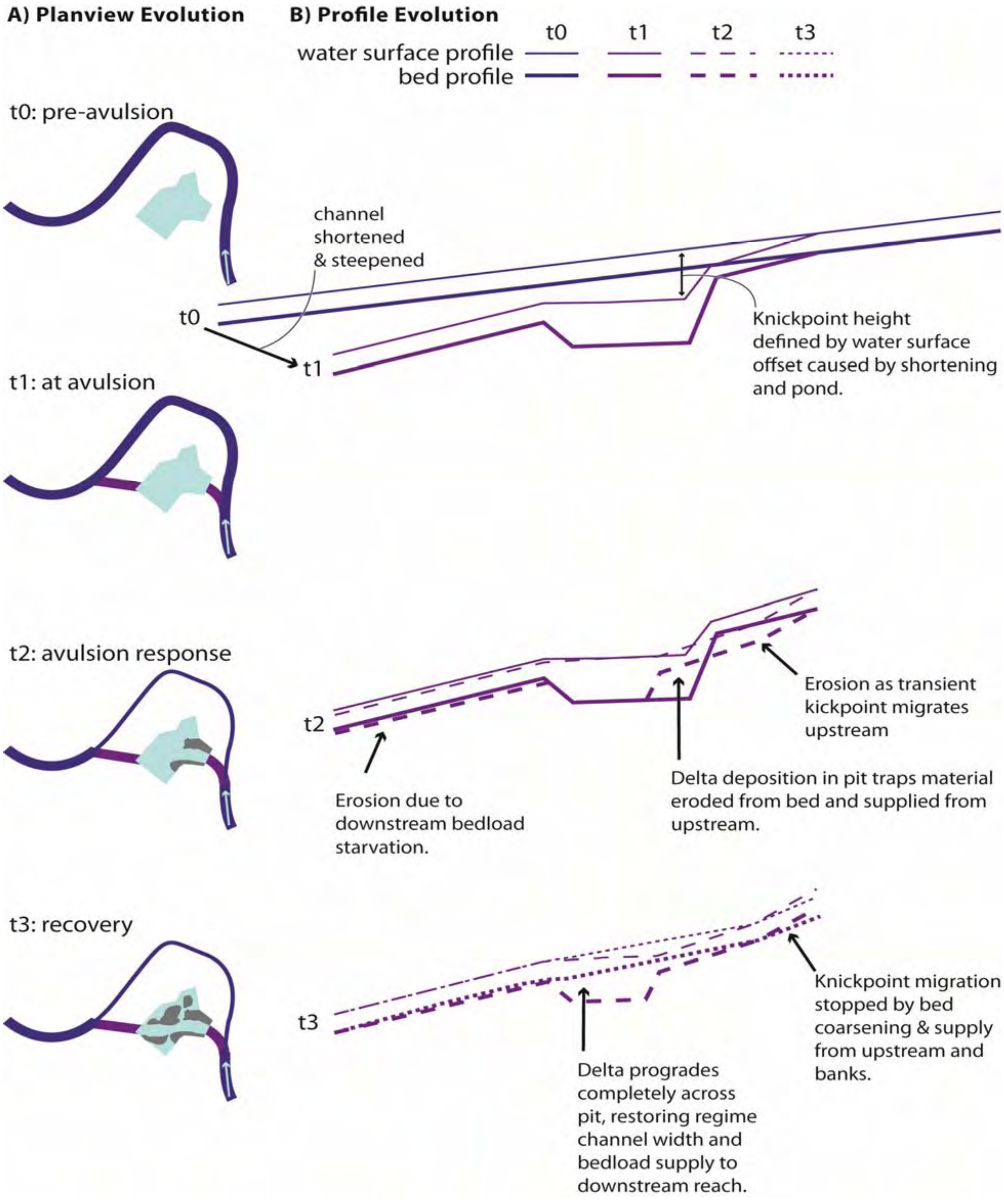

Figure 18: Typical evolution of an avulsion through a floodplain pit

4.3.2 Lateral Connection

A lateral connection forms with the intersection of a river channel and floodplain pit, but the channel does not avulse through the pit. This commonly occurs when the channel migrates laterally to form a connection on the downstream side and the new channel path through the pit is substantially longer than the existing channel path. Channel response to a lateral connection is subtle compared to response to an avulsion, and has received relatively little attention in published literature. Preliminary review of aerial photos of the Yakima and other rivers with floodplain mine pits indicates that this scenario is common, but only two of the fifteen examples in Appendix A, Table 1 represent this scenario.

In the lateral connection case, the floodplain pit is often outside of the channel's meander belt, (Figure 19, t0). Lateral migration of the channel intersects the pit (Figure 19, t1), which causes the channel to become oversized relative to a regime channel (wider, deeper, or both) and reduces the local shear stress. A large bar develops on the inside of the meander bend, directing the flow into the former pit area where suspended sediment may be deposited in slack water areas, and the channel thalweg migrates across the area of the pit (Figure 19, t2). Headcut effects and bedload starvation may both occur, but the impacts are typically much more limited than in the case of avulsion through a pit. Headcuts are limited because headcut height caused by effective channel shortening due to the length of the pond is offset by channel lengthening as a point bar grows rapidly. Bedload starvation is limited because bedload conveyance is usually partially maintained across the bar and because rapid channel migration often causes substantial floodplain erosion on the downstream side of the pond.

Many anthropogenic projects to reconnect floodplain pits represent an extreme case of the lateral connection scenario. Here, a channel is dug down valley from an existing pit to the river to allow fish access. This channel, however, acts as a tributary or side channel re-entry point and has trivial impacts on geomorphic processes in the main channel.

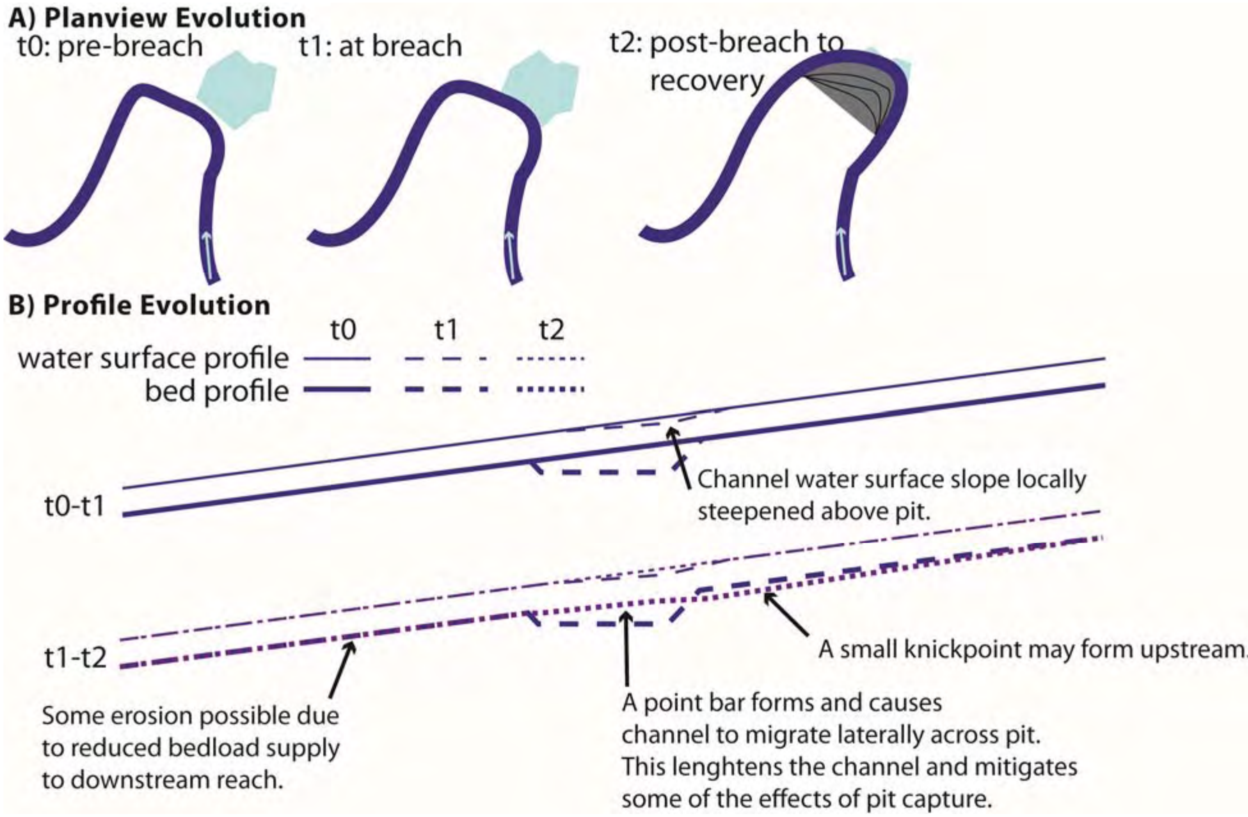

Figure 19: Typical evolution of a lateral connection to a floodplain pit

4.4 Pit Capture Hazards

Hazards to both ecosystem function and infrastructure are related to the magnitude and duration of key geomorphic processes described above. The following questions arise when attempting to define these impacts:

- What are the factors that define pit capture style, causing avulsion through floodplain pits in some cases but a lateral connection in others?
- What conditions govern the depth and upstream propagation distance for a headcut associated with pit capture?
- How do pits fill with sediment and how long is supply disrupted to the reach downstream?
- What conditions govern the depth of downstream degradation?
- What is the timescale for geomorphic recovery of the reach?

Answers to these questions require a thorough understanding of the fundamental geomorphic processes that occur when a pit is captured, which are considered below.

4.4.1 Avulsion Hazards

A broad body of literature has developed specifying the conditions in which avulsions are likely to occur. These are superelevation of the channel above the adjacent floodplain and a high ratio of the slope of a possible avulsion path to the down channel slope.

Partial avulsions can also occur when a new side channel forms but does not capture all or most of the river's flow. This occurrence is often associated with formation of large wood jams at the upstream avulsion node.

Superelevation is a parameter describing the degree to which a channel is perched above the floodplain. Absolute superelevation (S in Figure 20) is the height of the channel margin or natural levee above the floodplain. Normalized superelevation is defined as the ratio of the channel depth (D in Figure 20) to the absolute superelevation (S). When this parameter reaches values on the order of 0.5 to 1.1, avulsions typically occur, but on some low gradient rivers, such as the Mississippi, avulsions can occur at superelevation values as low as 0.19 to 0.62 (Mackey and Bridge, 1995; Mohrig et al., 2000; Jerolmack and Mohrig, 2006).

The Slope Ratio, though related to superelevation, provides another parameter that can predict the likelihood of avulsion. It is defined as the ratio of the slope of a possible avulsion path (Sa in Figure 20) to the down channel slope. Slope ratios in the range from 3 to 5 —consistent with theoretical predictions of Slingerland and Smith (1998)— are observed in naturally avulsing systems, though slope ratios an order of magnitude higher have been observed both during an active avulsion (Mohrig et al., 2000) and in some non-avulsing systems (Törnqvist and Bridge, 2002).

The superelevation and slope ratio can be calculated for a given floodplain capture site to predict the likelihood of an avulsive capture. Additional factors that influence the likelihood and scale of an avulsion include the availability of an outlet, erodibility of floodplain material, and potential for large wood jams to block the entrance to or exit from a pit.

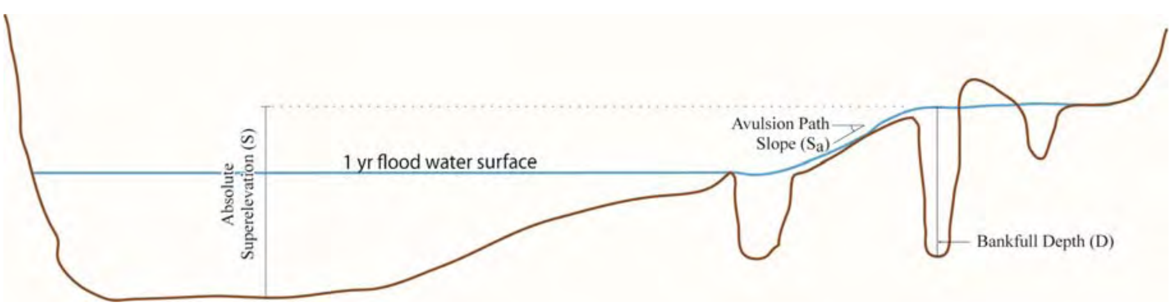

Figure 20: Geometric parameters related to avulsion probability. The figure is a hypothetical valley cross section perpendicular to the direction of flow

Table 3 in Appendix A indicates that the average slope ratio of avulsions through floodplain pits is 3.2 (± 2.5 with a strongly positively skewed distribution) while the average slope ratio for observed connections between a river and floodplain pit that did not result in avulsions is 1.5 (± 0.4), which is significantly lower (t_{21} = 2.1, p = 0.025). The minimum slope ratio where an avulsion occurred was 1.3. Results of a logistic regression (using the algorithm of Pezzullo 2014) are shown in Figure 21 (X^2 = 7.5; df = 1; p = 0.0062). This analysis indicates that an avulsion is possible at any slope ratio but that avulsion probability increases rapidly as the ratio increases from 1 to 2; avulsion has a 50% likelihood of occurring at a slope ratio of 1.7, and has more than a 70% chance of occurring once the slope ratio reaches 2.

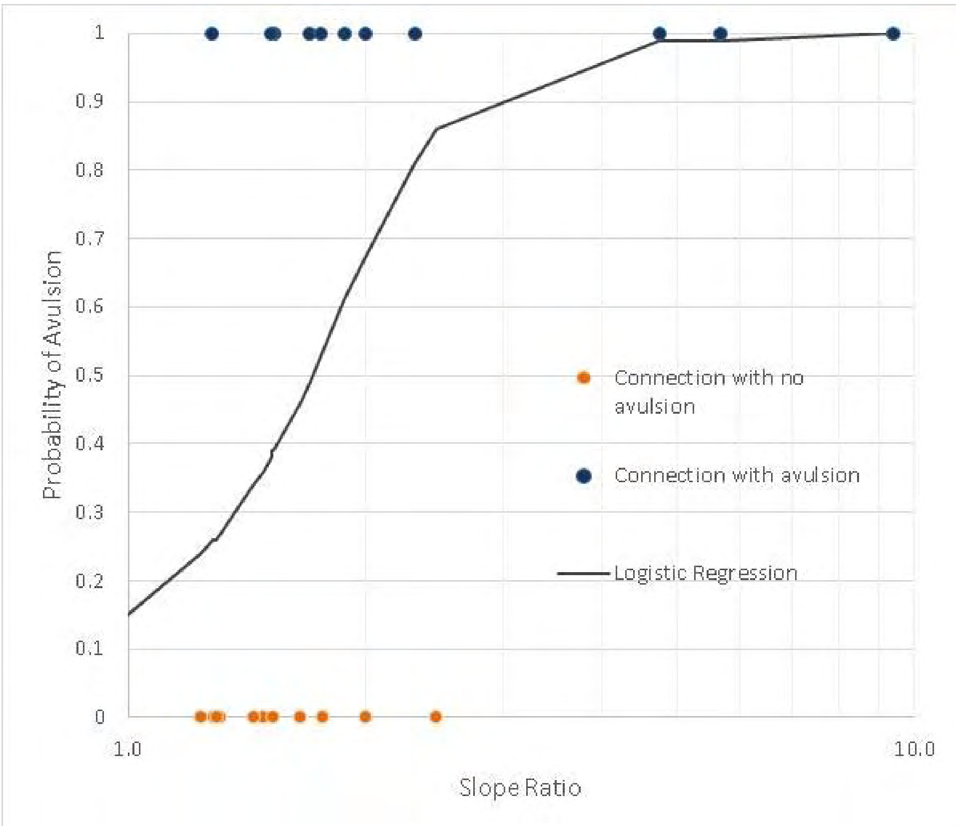

Figure 21: Logistic regression result predicting avulsion probability from slope ratio

4.4.2 Upstream Knickpoint Migration (Headcut)

A headcut is a local steep discontinuity in bed elevation. Dissipation of flow energy over the resulting drop concentrates erosion at the site of the headcut, causing it to migrate upstream. Though the term *headcut* is often used in the context of floodplain pit capture, it is best reserved for the case of a near-vertical discontinuity that forms in cohesive sediment (*sensu* Stein and Julien, 1993), where headward migration is primarily caused by erosion of a jet at the base of the drop. In the cases of floodplain pit capture considered here, where the bed is dominated by non-cohesive gravelly material, the more general term *knickpoint* is preferred. The knickpoint can be no steeper than the angle of repose for granular soil (approximately 1.5H:1V) and propagates upstream through preferential erosion by high velocity flow at the crest of the drop.

The height of a knickpoint (*z*) can be predicted based on the geometry of the avulsion pathway. The change in base level for the channel upstream defines the maximum height of the knickpoint. The new base level, in the case of an avulsion through a pond, is defined by the water surface elevation of the pond rather than the bed elevation at the bottom of the pond because the stream's energy gradient upstream will be controlled by a backwater from the pond surface as the headcut diffuses. As sketched in Figure 18 this base level change can be predicted by the slope of the channel prior to the avulsion (*Si*), the lengths of the pre avulsion path (*Li*), avulsion path (*La*), and the pond through which the avulsion occurred (*Lp*) by the relation:

$$z = S_i\left(L_i - (L_a - L_p)\right) \qquad \text{Eq.4}$$

In a lateral connection scenario pit capture results in an abrupt lengthening of the channel at the site of the pit capture. This effectively results in a negative sign for the "avulsion path" length (*La*) in the above formulation and can counter the influence of the flat water surface along the pond length.

Brush and Wolman (1960) modeled knickpoint behavior in a flume using a constant knickpoint height (*z*) under a variety of average channel slope (*Si*), discharge, and particle size distributions. They found a consistent general response, regardless of the particular parameter combination: the slope of the channel downstream of the knickpoint always decreased through time. As the knickpoint migrated upstream, the channel above the knickpoint initially narrowed and steepened, but subsequently as erosion was concentrated at the top of the knickpoint drop, the slope flattened. They found material eroded from the knickpoint formed a dune below the knickpoint that migrated downstream, causing local bank erosion.

Ultimately, Brush and Wolman (1960) found that knickpoints in homogeneous non-cohesive bed material always flatten to oblivion and the slope of the knickpoint asymptotically approaches the overall reach slope. They applied a threshold of slope difference to define the point of oblivion, which allows the runout distance to be calculated geometrically. Assuming the toe of the knickpoint does not aggrade, a knickpoint will propagate a distance of *x* feet:

$$x = \frac{z}{t\,S_i} \qquad \text{Eq. 5}$$

where z is the initial height of the knickpoint, S_i is the initial slope of the reach upstream of the knickpoint, and t is defined by the expression

$$t = \frac{S_t - S_i}{S_i} \qquad \text{Eq. 6}$$

where S_t is the slope at which the headcut propagates upstream. This formulation provides a useful parameter for estimating the length of upstream disturbance that might be expected with the capture of an individual pit. In reality, deposition of sediment at the toe of the knickpoint may reduce the length of headward migration substantially. In the case of a floodplain pit capture scenario, a large volume of sediment is rapidly deposited at the toe of the knickpoint as a delta progrades into the captured pit. Brush and Wolman (1960) suggest, somewhat arbitrarily[2], a t value of 1.2. Several factors likely govern fathe variability of t values in natural streams. These include the grain size distribution of the bed material, the ability of the bed to develop a coarse armor layer (which is controlled by partial[3] versus full mobility of the bed at the knickpoint), and existing variability in the reach longitudinal profile. The natural ratio of the slope of riffles to the reach slope might integrate these factors and serve as a starting point for prediction of the ΔS value at a given site. On the Yakima River near the project reach the ratio of riffle slopes to reach slope ranges from 2 to 4, with an average value of approximately 2.8 based on the existing thalweg profile. Given an average reach slope of 0.0026, this would equate to a range of t values between 1 and 3 with an average value of about 1.8.

The upstream progression of the headcut described by Brush and Wolman (1960) has been investigated using the basic equations of sediment transport, mass continuity and channel hydraulics by Tinney (1962), de Vries (1975, 1985), and Graf (1998). This approach leads to a more generalized expression than the empirical relation in Eq. 2 which is based on the differential equation that relates bed level changes (z) to the distance along the channel (x) and time (t) as shown in Eq. 7.

$$\frac{\partial z}{\partial t} - K\frac{\partial^2 z}{\partial x^2} = 0 \qquad \text{Eq. 7}$$

The solution of this differential equation (Eq. 8) expresses the bed level changes using the complementary error function (*erfc*) and a diffusion constant K which incorporates the bed load transport characteristics of the channel and the channel slope.

$$z(x,t) = -\Delta z \cdot erfc\left[\frac{x}{2\sqrt{Kt}}\right] \qquad \text{Eq. 8}$$

This relation is useful for understanding the physical basis of the observations made by Brush and Wolman, but is still difficult to apply in practical cases since there is considerable uncertainty in determining the diffusion constant. In Section 4.5 we have made a brief comparison of the empirical and analytical predictions for the particular case of a pit capture at the Newland Ponds.

Combining equations 4 and 5 allows prediction of the depth and length of upstream disturbance caused by an avulsion through a floodplain pit based solely on the planform geometry of the avulsion and selection of an appropriate value for the threshold slope (t). The dataset presented in Appendix A can be used to test these two empirical relations.

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[2] They do not explicitly state this in their text, but presumably this choice was related to their experience watching flume experiments evolve.

[3] Partial mobility is the condition where only some of the grains on the bed are mobile at any given time (Wilcock and McArdell, 1993).

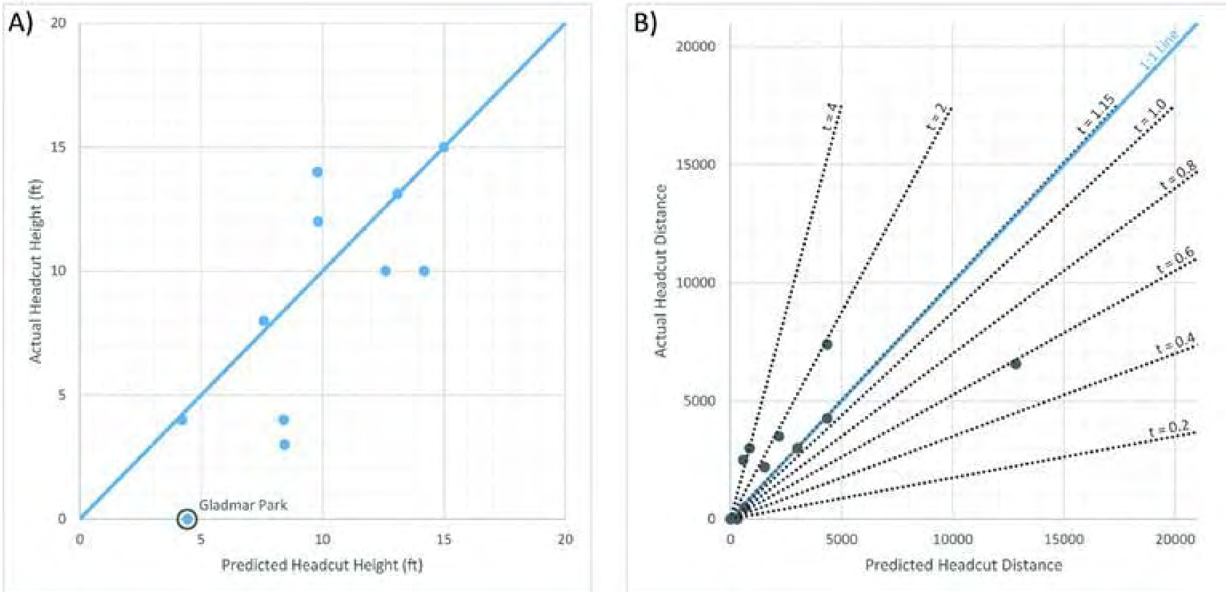

Figure 22: Headcut height predicted by Equation 4 and headcut propagation distance predicted from Equation 5. Data points are based on the calibrated t=1.15, dashed lines indicate the best fit line for *t* values as indicated

Figure 22A shows the reasonably close correspondence between known headcut heights and those predicted by Equation 2. Four out of seven examples fall directly on the 1:1 line, two headcuts were up to 40% higher than predicted by the relation, and five were lower than predicted by the relation. Note the outlier at 4.5, 0. This point is the partial avulsion at Gladmar Pond, where the headcut depth has been limited by engineering intervention and formation of a log jam spanning the avulsion channel (NHC, 2012).

Figure 22B shows the relation between known headcut propagation distances and those predicted by Equation 5. Where the headcut height (z) was known, that was used; where it was unknown, it was estimated using Equation 4. For the gravel bed rivers in the dataset, a value of t = 1.15 calibrates the regression relation. The lowest t value is 0.6 and t = 4 provides an approximate upper bound. These t values are within the range expected by the relationship between riffle slopes to reach average slope. A SRH 1-D model run of the Wapato Dam removal simulated the process of bed coarsening and knickpoint migration on a reach of the Yakima River near the project site (NHC, 2012b). This simulation predicted a knickpoint height of 10 feet on a reach with initial slope of 3%. After 25 years of simulation, the upstream extent of degradation had extended about 9,000 feet. This corresponds to a value of t = 0.4.

4.4.3 Delta Growth

Where the channel enters a captured pit, the conveyance area increases, reducing flow velocity and bed shear stress which reduces the competence of the flow and causes bedload sediment to be deposited as a Gilbert-type delta. This is a specific type of delta formed by coarse sediments where a channel enters a slack water body and foreset beds dipping downstream are deposited over finer, flatter bottomset beds deposited further downstream (Coulhoun in Goudie, 2004). The delta advances by progradation of foreset beds at the delta front. The top of the foreset beds will be slightly below the water surface of the pond, but likely above the pond tail out elevation. The slope of the delta top will be at the threshold required to transport the bedload delivered to it.

Once the delta has prograded across the pond, the channel through the pond will narrow to the point where bedload sediment will be delivered downstream, ending the period of bedload starvation for the channel downstream. The theoretical length of time required to re-establish downstream continuity of sediment transport, therefore, is equivalent to the volume of the pit divided by the bedload transport rate in the reach. In reality, the time required to re-establish downstream sediment transport will be somewhat less than this because of the large volume of sediment that will be supplied to the delta due to erosion at the headcut immediately upstream and because some volume may be transported in remaining channels outside of the pond. In addition, some volume of sand may settle from suspension and supply a portion of the pond volume, reducing the required volume of bedload to fill the void. This is illustrated by selected examples of pit filling in the Gap to Gap reach of the Yakima shown in Figure 23. This figure shows the relation between the time expected to completely fill the pit based on the reach-average bed material transport rate of 13,000 yd^3/yr (Hilldale and Godaire, 2010) and pit volumes estimated from pit areas and the average pit depth of floodplain pits along the Yakima River, which is 8.2 ±4.9 feet (± 1 σ, as determined from data published by the Yakima River Floodplain Mining Impact Study Team, 2004). This figure shows that the time required to restore downstream sediment conveyance (as indicated by planform change in aerial photos) is consistently about 40% less than the time that would be required to completely fill the pit. Another possible reason for this difference would be underestimation of the bed material load transport rate of the reach.

4.4.4 Downstream Degradation

Channel degradation downstream of a captured floodplain gravel pit may occur because the pit captures all or part of the bedload transported from upstream through the delta or point bar growth process, described above. The physical process causing degradation in this case is analogous to that occurring below dams that interrupt downstream bedload conveyance. A river channel will degrade whenever the rate of sediment supply is less than the rate at which sediment is transported away (Tinney, 1962; Kondolf, 1997). Because the presence of a pit upstream may not significantly change hydraulic or bed conditions downstream, the rate of transport out of the reach downstream will initially be the same as the pre-avulsion rate. The rate of bedload supply will be temporarily reduced to near zero after an avulsion through a pit and may be reduced substantially after a lateral connection, and so the reach downstream can be expected to degrade.

Little quantitative published information is available with which to evaluate the impact of relatively short (<5 year) interruptions to bedload transport, but given typically episodic bedload supply and based on experience observing the response of multiple systems to construction of check dams, and diversion dams for relatively small run-of-river hydropower projects on geomorphically similar systems, it appears as though sediment starvation for one to five years has limited downstream geomorphic impact. Studies of the impacts of large dams causing downstream bedload starvation have shown that the downstream channel responds through a combination of surface coarsening and channel incision (e.g. Kondolf, 1997). No simple empirical tools analogous to those developed above to predict headcut height and extent are available to quantitatively predict these impacts, and so numerical process-based modeling of the reach of interest would be required to quantify these impacts.

As described above, downstream degradation may be expected to occur for the duration of bedload trapping at an inline pit. In most past examples, however, the extent and duration of downstream degradation has been limited by bed coarsening and an increased supply of material from increased rates of bank erosion. This is not always the case as illustrated in a case study from Spain where the bed degradation due to knickpoint migration continued past the cessation of mining activities because incision caused channel narrowing and an increase in shear stress (Martín-Vide et al., 2010). Increased shear stress increased bedload transport out of the reach above conditions prior to mining and amplified effects of downstream degradation.

4.4.5 Bar Growth and Meander Migration

The style of pit capture without channel avulsion (as illustrated in Figure 19) results in a locally over-widened channel segment with low shear stress. This causes bedload transported from upstream to be deposited and form bars in the pit area, although only some fraction of the incoming bedload is likely to be captured. Deposition patterns may drive the channel thalweg migration across the pit. The primary hazard associated with this behavior is the possibility that the accelerated local channel migration may cause erosion of adjacent valuable property. Lateral intersection with a very large pit that has the potential to store many years of bedload could also lead to downstream degradation.

4.5 Geomorphic Assessment of Newland Pit Capture

4.5.1 Capture Scenarios

The Newland Ponds lie inside a meander bend just downstream of SR24. If the DID #1 levee were removed to the existing floodplain grade, overbank water flow would interact with these ponds above bankfull flow conditions If the existing revetment associated with the levee were removed, channel migration would be expected to intersect Pond 1 within a few years. Channel migration downstream would also be expected to intersect Ponds 2 and 3 very shortly after the DID #1 levee and bank riprap was removed since the two ponds are located on the outside of a meander bend where the bank is subject to high velocities and shear stresses.

Three distinct floodplain pit capture scenarios were evaluated as part of this assessment (Figure 23). Mechanisms initiating these scenarios are a function of channel migration rates, flood magnitude and flood timing, and local erodibility of the floodplain sediments. In Scenario A, an avulsion breaches all three ponds and returns to the current channel location downstream of Pond 3. This avulsion would abandon approximately 4,600 feet of channel in favour of a 3,900 foot path through 2,700 feet of ponds. The slope ratio of this avulsion path is 3.7. In Scenario B, an avulsion breaches only Ponds 1 and 2 and returns to the main channel upstream of Pond 3. Subsequent lateral migration would likely then capture Pond 3, so this scenario was not considered separately from Scenario A.

In Scenario C, lateral migration at the downstream bend occurs much more rapidly than at the bend upstream of Pond 1 resulting in a lateral connection between the channel and Ponds 2 and 3. In Scenario C, the effected segment of channel would lengthen from approximately 2,200 feet to 2,800 feet, with 1,500 feet crossing the ponds. Even though the channel is longer, the intersection with the ponds could create a headcut because of the length of channel-pond intersection. The existing DID #1 levee is in close proximity to the river bank and the edge of the three gravel pits. Key dimensions of the pits are summarized in Table 9 in Section 4.1. The maximum depth in the pits ranges from 29.6 feet to 72.7 feet. By comparison, the maximum channel depth at flood stage is typically less than 20 feet.

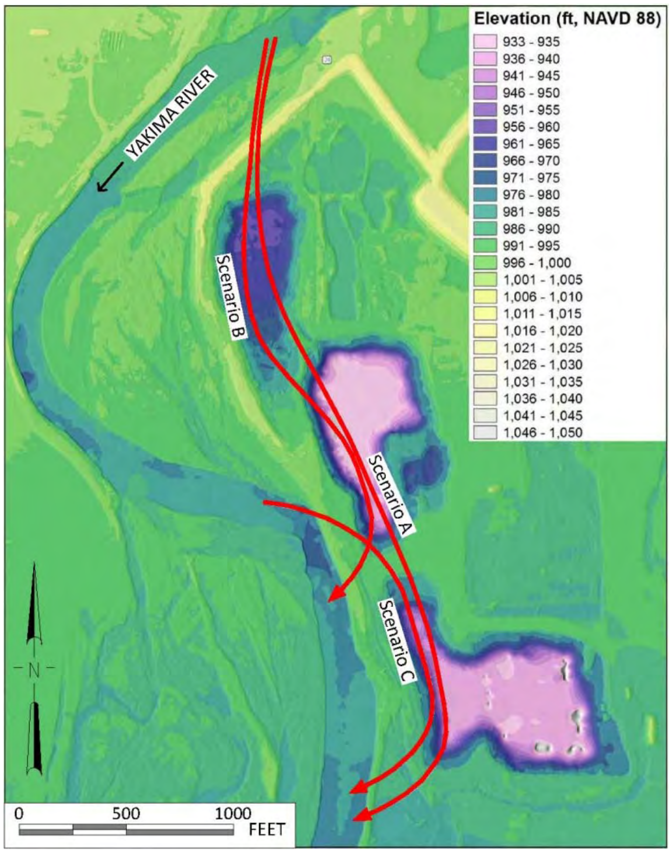

Figure 23: Possible pit capture scenarios

4.5.2 Avulsion Likelihood

The worst-case avulsion scenario (in terms of magnitude of geomorphic disturbance) would be a complete avulsion of the channel through the three ponds, Scenario A in Figure 23. Based on the slope ratio, this configuration is very likely (~95% probability) to occur if an avulsion develops an intersection between the channel and Pond 1. Given the history of dynamic channel migration in the reach and the position of the pond along the outside of a meander bend, an upstream intersection of the channel and pond is expected. The downstream ponds (2 and 3) are even closer to the channel and the geometry of that bend is such that lateral migration would be expected to begin immediately after the revetment along the levee and its revetment is removed; Scenario C is also very likely to occur and may occur prior to the complete avulsion.

4.5.3 Headcut Height and Extent

Based on Equation 4 and assuming a reach average slope of 0.0026, the predicted headcut height above the avulsion would be 9 feet for Scenario A. Given the range of scatter in the empirical data, a prediction band of ± 50% is appropriate. The furthest upstream impact of a 9 foot headcut is predicted (with a calibrated *t* value of 1.15) to be approximately 3,000 feet upstream. There is uncertainty in both the predicted headcut height and choice of *t* value. Table 10 provides a sensitivity analysis applying the range of probable values (based on the empirical dataset) for both *t* and *z.* This analysis indicates that the maximum reasonable estimate of headcut influence is approximately 13,000 feet.

If the only pit capture was to be that of Scenario C, the predicted headcut height would be 2.3 feet and the headcut would be expected to propagate between 800 and 2,200 feet upstream.

A particular concern is the possibility that a headcut caused by an avulsion at the project could destabilize the revetment protecting the former gravel pit, Buchanan Lake, which parallels the river from 5,700 to 7,800 feet upstream. The most hazardous location for a levee breach is at the upstream end of the Lake because a vegetated bar buffers the revetment from the channel and the downstream end of the Lake. Four of the sensitivity analysis scenarios result in headcut propagation distances that would reach the Lake. Given the most extreme scenario, the depth of channel downcutting at the downstream part of the Lake would be expected to be approximately 8 feet and this would taper to approximately 5 feet at the critical upstream part of the Lake. Additional detailed modeling and site-specific geomorphic analysis is required to provide final estimates of potential degradation at Buchanan Lake.

Table 10: Sensitivity Analysis for headcut propagation distance (*x*, ft), given a variety of estimates of *t* (dimensionless) and headcut height (*z*, ft).

t \ z	2.3	4.5	9	13.5
0.4	2,200	4,300	8,700	13,000
0.6	1,500	2,900	5,800	8,700
1	900	1,700	3,500	5,200
1.15	800	1,500	3,000	4,500
2	400	900	1,700	2,600
4	200	400	900	1,300

The empirical expression for headcut propagation was compared with the analytical relation in Eq. 8 using the available hydraulic and sediment transport characteristics on the Yakima River to estimate the diffusion coefficient K.

From de Vries (1985): $K \approx \frac{1}{3} n \frac{s}{i}$

Where s is the volumetric bed load transport rate per unit channel width, i is the channel slope and n is the exponent in the bed load transport relation between channel velocity and transport rate. For the purposes of these computations the following parameters were used:

- s = 13,000 cubic yards/year
- Ch. W (active) = 150 feet
- i = 0.0028
- n = 5

This resulted in a K coefficient of 1.4 x 10^6. The predicted bed level response upstream from a pit capture is shown in Figure 24. In this case it was assumed the initial bed lowering at the head of the ponds was 7 feet.

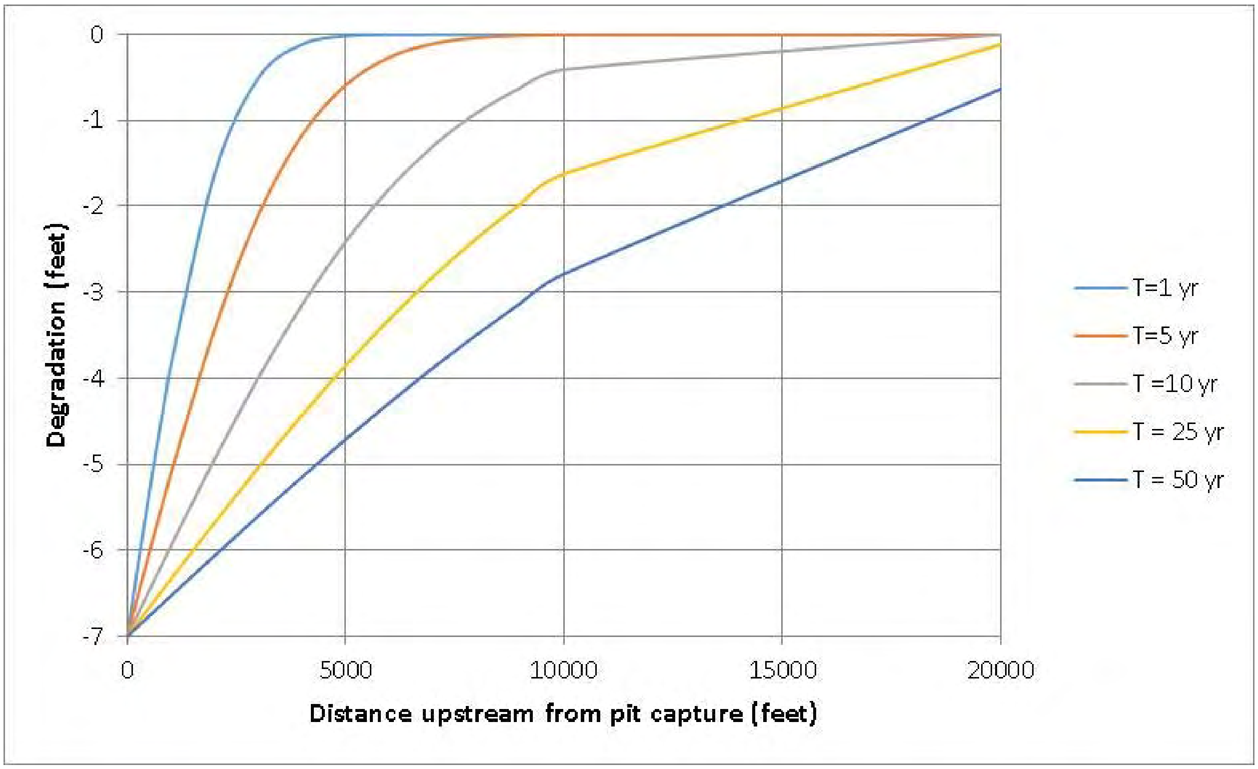

Figure 24: Predicted upstream bed level changes in response to a full pit capture at the Newland Ponds using the analytical equation by de Vries

This result shows that the bed degraded by 2 feet at a point 8,000 feet upstream from the ponds after 25 years, which is consistent with the empirical-based predictions.

4.5.4 Recovery Timescale

The duration and magnitude of geomorphic disturbance associated with pit capture is closely related to the relationship between the volume of the pit and bed material load transport capacity of the channel. The maximum period of interrupted bed material sediment supply downstream is equivalent to the pit volume divided by the average annual sediment transport rate; this same ratio provides a quantitative measure of the overall magnitude of geomorphic disturbance associated with a given pit capture.

Hilldale and Godaire (2010) estimated average annual bed material load for the reach on the basis of a one-dimensional sediment transport model and historical suspended sediment transport measurements. Their model result suggested that the river transports approximately 19,000 tons/year, and empirical observations based on suspended sediment transport between 1975 and 1993 suggest the river transports approximately 24,000 tons/year. Assuming a typical bulk density for alluvial sediment of 1.45 tons / yd^3, these values are equivalent to 13,000 and 17,000 yd^3 / year.

Based on results of floodplain pit surveys, the combined volume of the three main Newland Ponds is approximately 1.5×10^6 yd^3 with about 1.0×10^6 yd^3 of volume that would be filled along a likely avulsion path. This volume could interrupt sediment transport for a maximum period of up to half a century, which is a long enough period to cause important downstream geomorphic effects, including channel downcutting and bed armoring.

Several factors could reduce the timescale of disturbance. A regime channel often builds within mine pits that maintain significant areas of open water, allowing sediment transport continuity to be largely restored after a much shorter period than required to completely fill the pits. For example it would take less than 100,000 yd^3 to construct a pair of 100 foot wide bars along either side of the channel along the length of the avulsion, which would take on the order of half a decade to accomplish. This is generally consistent with empirical observation of the time required to restore downstream sediment transport past a captured pit. Timescale to restore sediment transport has been observed to be approximately 40% shorter than the time required to completely fill the pit. Furthermore, a large volume of material may be locally mobilized as the headcut from the pit propagates upstream, as occurred at the Selah Ponds avulsion in 1996 when 300,000 yd^3 of sediment was locally eroded during the flood that caused the breach. The best estimate for the total volume of material that may be mobilized by the headcut above a possible avulsion through the Newland Ponds is approximately 100,000 yd^3 which would reduce the time required to fill the pits by approximately 10% and may be enough material to restore downstream sediment conveyance shortly after the avulsion.

5 HYDRAULIC MODEL INVESTIGATIONS OF PIT CAPTURES

5.1 Previous Model Development

Previous morphodynamic modeling of the Yakima River in the Gap to Gap Reach was conducted by the USBR (Hilldale and Godaire, 2010). The purpose of the USBR study was to investigate the possible effects of the proposed levee set-backs, including DID #1 levee, on future condition channel morphology and hydraulics along the Yakima River. The USBR constructed a one-dimensional, morphodynamic (mobile bed) model spanning the 10.5 mile long Gap to Gap reach using their Sedimentation and River Hydraulics-One Dimension (SRH-1D, v2.3) software package. The model included 87 cross sections, with average spacing of approximately 600 feet, to represent channel and floodplain geometry along the reach. Channel bathymetry was collected by the USGS in the summers of 2004 and 2005, with small portions at the upstream and downstream limits and near the mouth of the Naches River collected in 2008 (Hilldale and Godaire, 2010). Overbank floodplain geometry was primarily represented with LiDAR topographic data collected in 2005, but was supplemented with 2002 LiDAR in some locations (Hilldale and Godaire, 2010). A key omission from the USBR study was bathymetric data of the three Newland Ponds; these data were not available at the time, and without them, a comprehensive impact assessment could not be conducted. The original hydrologic input to the SRH-1D model consisted of a continuous 25 year hydrograph of daily mean discharges spanning water years 1985 to 2009 (Hilldale and Godaire, 2010).

The base geometry of the SRH-1D model was used in subsequent models developed by the Corps using their one-dimensional HEC-RAS software package. These models were useful for prediction of flood levels along the Gap to Gap reach and assessment of various levee set-back projects. Hydrologic inputs for the HEC-RAS model were provided from previous studies and consisted of flood discharges as reported in Table 2.

The County provided NHC with the existing and proposed condition SRH-1D and HEC-RAS models, which NHC subsequently reviewed and revised as described in the following sections.

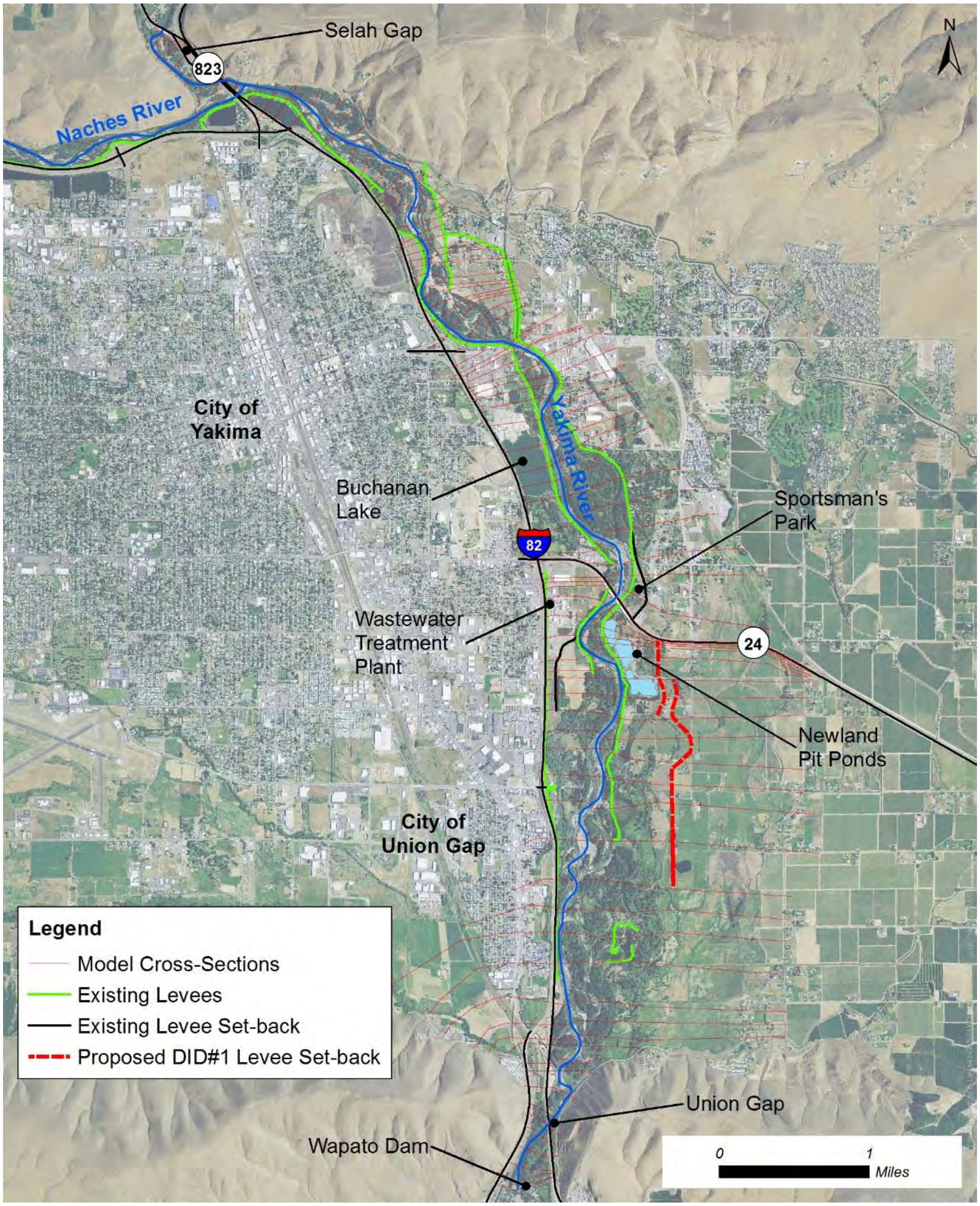

Figure 25: Extent of USBR SRH-1D model of Yakima River (Hilldale and Godaire, 2010)

5.2 SRH-1D Model of Pit Captures

5.2.1 Model Revisions

NHC reviewed the previously developed SRH-1D and HEC-RAS models and determined that minor revisions were warranted to better represent hydraulic conditions and compute flood levels in the vicinity of the SR24 Bridge crossing. For existing conditions, ineffective flow limits were added downstream of the DID #1 levee to limit flow conveyance in the far left overbank floodplain. Similarly, for the proposed DID #1 levee set-back, ineffective flow limits were added on the left overbank to account for obstruction created by the SR24 Bridge approach embankment to the east of the bridge. Figure 26 illustrates the locations where ineffective flow limits were added to limit conveyance for both the existing conditions and proposed levee set-back. In addition, NHC removed the cross-dike structure below SR24 Bridge to evaluate its impact on upstream flood levels. Review of the USBR's proposed levee set-back SRH-1D model indicated that the cross-dike was also removed for their simulation (Hilldale and Godaire, 2010). Further revision to the proposed levee set-back model included representing the topography of the Newland Ponds using NHC's 2014 surveys.

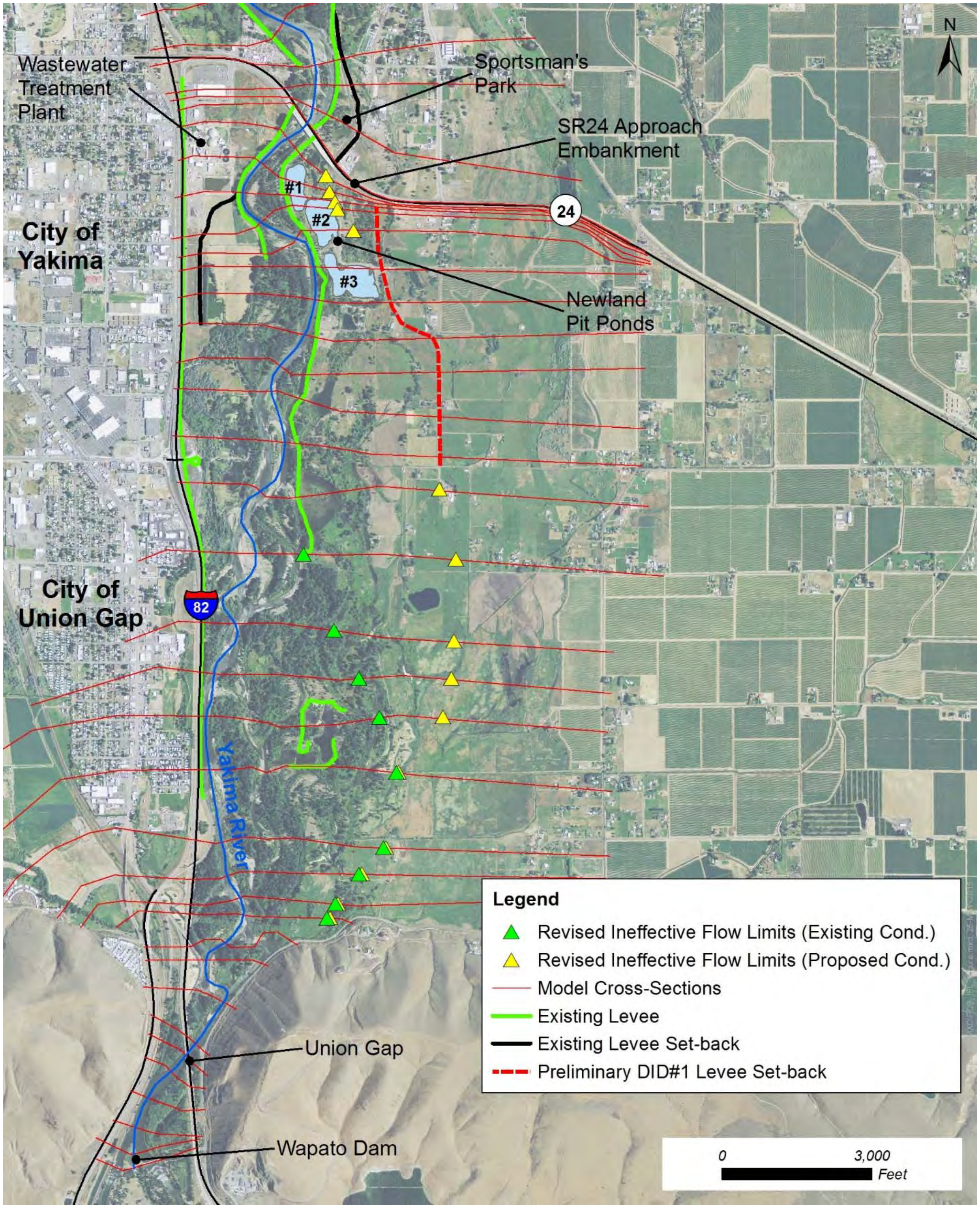

Figure 26: Modifications to model geometry

In select runs, NHC extended the simulation period from the original 25 years to 50 or 100 years by repeating the original 25 year time series used by Hilldale and Godaire (2010). The simulation period was extended to evaluate longer term patterns of sediment transport and bed morphology along the study reach.

5.2.2 Model Scenarios

Table 11 describes the scenarios that were evaluated with the SRH-1D model. The first run was made to provide a base-case comparison with the previous USBR model results in Hilldale and Godaire (2010). The results showed there was little change between the USBR and NHC models for the existing conditions. Both models indicated aggradation would occur upstream and downstream of SR24 Bridge. Hilldale and Godaire (2010) noted that downstream aggradation in the segment adjacent to the wastewater treatment plant was unrealistic and was related to the limitations of using one dimensional modeling. Field survey comparisons indicate modest degradation occurring in this location.

Table 11: Summary of SRH-1D model runs

Run	Description	Simulation Time (Years)
Exist	Existing condition model, e.g. no levee set-back, run	50
A	Levee set-back with no pit capture (just overbank flow allowed)	25
B	Full-avulsion with pit capture of all pits	50
C	Partial avulsion with capture of Pit 1 only	50
D[1.]	Run B with 120’ pilot channel added on Sportsman’s Park Island u/s of SR24 Bridge.	50
E[1.]	Run B with 220’ pilot channel added on Sportsman’s Park Island u/s of SR24 Bridge	100

1. Results summarized in Appendix B

Runs A through C were made to simulate the incremental effects of assumed levee and channel configurations.

Run A simulated set-back of the DID #1 levee assuming the main stem of the Yakima River remained in its current alignment. This run represents the increase in floodplain conveyance as a result of the levee set-back downstream of SR24 Bridge. Run A was similar to the proposed condition evaluated by Hilldale and Goldaire (2010), aside from the additional ineffective flow areas introduced by NHC.

Run B simulated a full avulsion of the Yakima River into the Newland Ponds. It was assumed that the main channel of the river would shift eastward at SR24 Bridge through all three of the Newland Ponds converging with the main stem immediately downstream of the ponds. Reach lengths were adjusted to represent the avulsion path, resulting in the overall channel shortening by approximately 400 feet. The conveyance areas within the ponds were determined iteratively and limited to the top 30 feet of the water column.

Run C simulated an avulsion of the Yakima into the upstream-most Newland Pond 1 only. Newland Ponds 2 and 3 were isolated with conveyance limited to the floodplain level, as in Run A. The avulsion path re-joined the main stem of the Yakima River immediately downstream of Pond 1.

It should be noted that the Nob Hill set-back of the west Federal levee upstream of SR24 was not considered in any of the SRH-1D model runs. The configuration of the Nob Hill set-back is located behind the existing SR24 west approach embankment, thus would not have an appreciable effect on 1D hydraulics in the reach (Figure 26).

5.2.3 Run A: Levee Setback Only with No Pit Capture

Run A simulated set-back of the DID #1 levee with accompanying increased flow conveyance downstream of SR24 Bridge, but assumed the main stem Yakima River remained in its current alignment and did not avulse into the Newland Ponds. This scenario was modeled previously by the USBR (Hilldale and Godaire, 2010). Their results indicated 1 to 3 feet of aggradation would occur downstream of SR24 Bridge and 1 to 2 feet of degradation would occur upstream to the Terrace Heights Bridge. The downstream aggradation was attributed to the sudden expansion and reduced flow energy downstream of SR24 Bridge following set-back of the DID #1 levee. However, as previously discussed, the expansion modeled by the USBR was exaggerated; i.e. it was without considering the effects of the SR24 Bridge approach embankment. These effects are considered in NHC's Run A, which showed 1 to 2.5 feet of degradation would occur downstream of the SR24 Bridge as a result of the levee set-back.

Figure 27 compares the predicted cumulative change to average channel bed elevation after a 25 year simulation from the original USBR study (Hilldale and Godaire, 2010) and subsequent runs made by NHC.

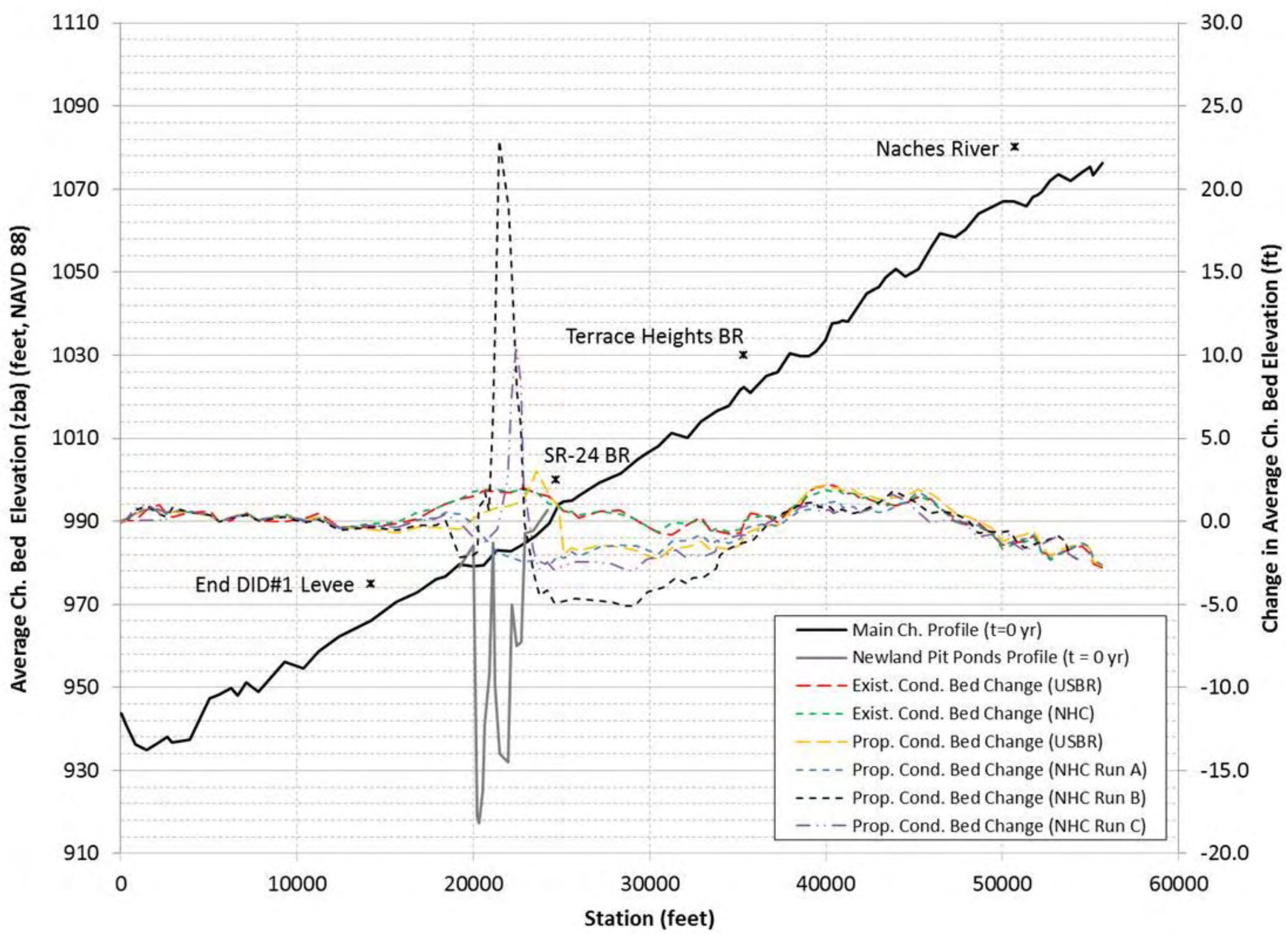

Figure 27: Change in average channel bed elevation computed by the USBR and NHC for existing and proposed conditions (25 year simulation)

Although the flow expansion past the SR24 Bridge was improved, there still appears to be a complex distribution of channel and overbank flow in the vicinity of the pits that is difficult to represent in a one-dimensional model. Upstream of the SR24 Bridge, the USBR and NHC Run A model results were similar with 1 to 2 feet of average bed degradation predicted. There is higher confidence in these results as the upstream degradation can be attributed to the locally steepened energy gradient near the SR24 Bridge following levee set-back.

Figure 28 shows thalweg profile changes for Run A over a 25 year simulation period. It should be noted that changes in thalweg elevation (as indicated in Figure 28) differ slightly from average bed levels as shown in Figure 27). This is a result of how SRH-1D represents bed level changes within the active channel. In general, changes in thalweg levels represent maximum values and are slightly greater than changes in average bed levels. Modest degradation (generally less than 2 feet), was computed in the first ten years of the simulation (*t* = 10 years). Maximum degradation of approximately 3 feet occurred between years 10 and 15, but generally remained within 1,000 to 2,000 feet upstream and downstream of the SR24 Bridge. From year 15 to 25, modest aggradation (less than 1 foot) was computed downstream of the SR24 Bridge.

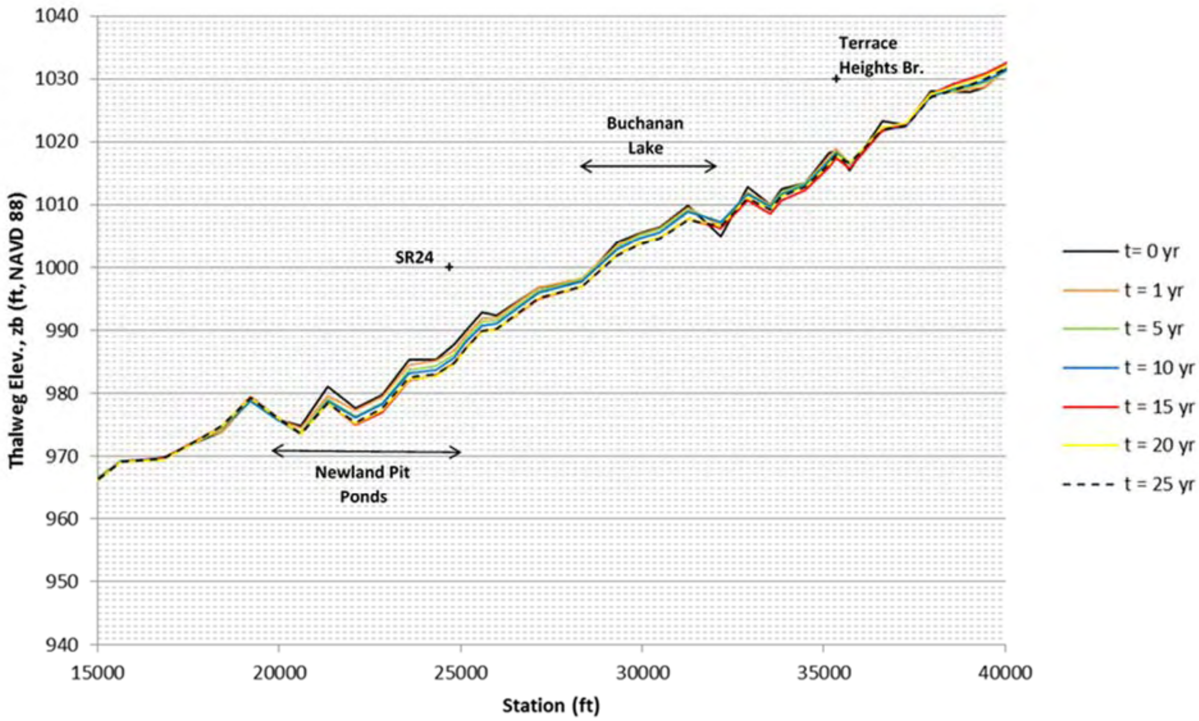

Figure 28: Run A computed thalweg long profile adjustment for 25 year simulation

5.2.4 Run B: Full Avulsion with Capture of All Newland Ponds

Run B assumed that with set back of the DID #1 levee the Yakima River would eventually avulse directly into Newland Pond 1 near SR24 Bridge, continue downstream through Ponds 2 and 3, and re-join the main channel downstream near RS 19948. It was further assumed that the current main channel adjacent to the pits would be fully abandoned. These conditions represent a ‘worst case’ scenario in terms of uncontrolled channel adjustment without the implementation of mitigation measures. The model results showed approximately 23 feet of average bed aggradation within the ponds downstream of SR24 Bridge during a 25 year time span. Further downstream, past the ponds, minor channel degradation was indicated. Upstream of SR24 Bridge, degradation more than doubles relative to Run A, with average bed elevations lowering by 3 to 5.5 feet up to the Terrace Heights Bridge.

Figure 29 shows thalweg profile changes for Run B over a 50 year simulation period. The simulation period was extended for Run B to evaluate time scales for infilling of the Newland Ponds. Within the first year (*t* = 1 year) computations showed 2 to 4 feet of degradation along the 3,000 foot reach upstream of Pond 1, over 19 feet of aggradation within the upper half of Pond 1, and 4 to 5 feet of degradation downstream of Pond 3 where the avulsion path converges with the main channel. After ten years (*t* = 10 years) Pond 1 was filled, and after 15 years (*t* = 15 years) degradation of 2 to 7 feet extended over 11,000 feet upstream of Pond 1. At the end of the 25 year simulation (*t* = 25 years) Pond 2 was filled, degradation exceeded 4 feet adjacent to Buchanan Lake, and overall degradation exceeding 2 feet extended an additional 1,500 feet toward the Terrace Heights Bridge. At the end of the 50 year simulation (*t* = 50 years), approximately two-thirds of Pond 3 was filled and degradation of 2 to 7 feet extended over 14,000 feet upstream of Pond 1 and past the Terrace Heights Bridge. Degradation downstream of the Newland Ponds during the 50 year simulation remained localized to the 1,000 foot reach immediately downstream of Pond 3.

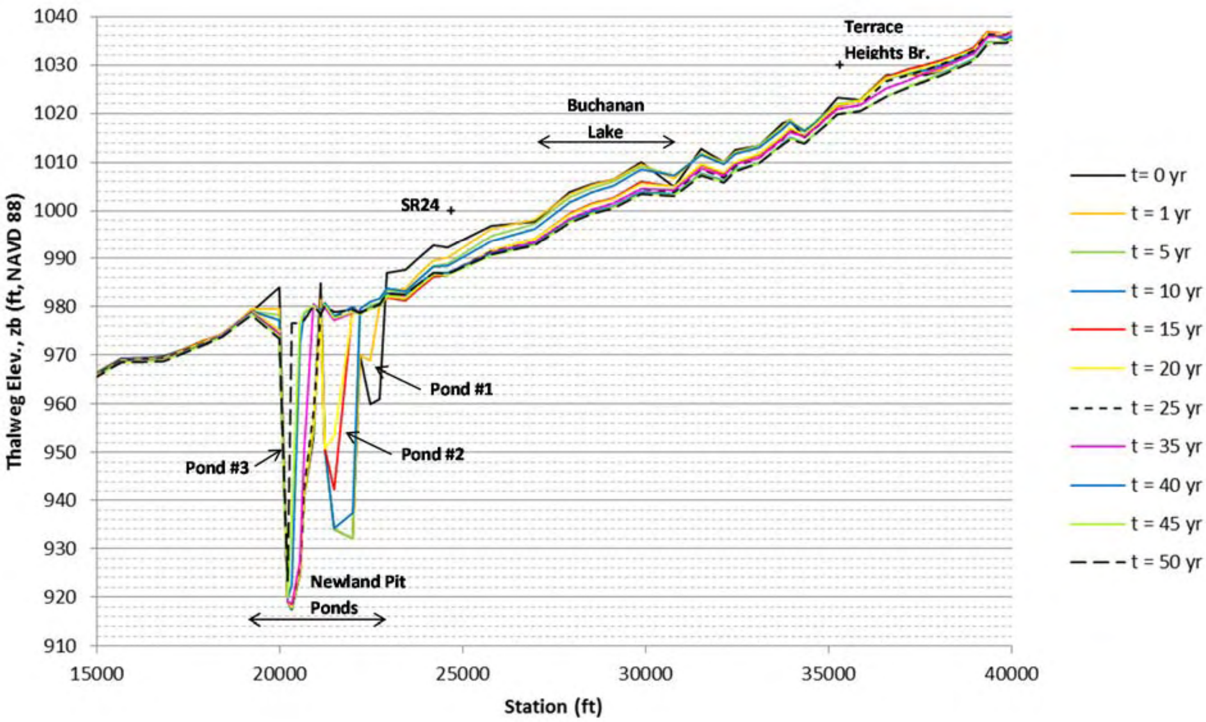

Figure 29: Run B, computed thalweg long profile adjustment for 50 year simulation

5.2.5 Run C: Capture of Newland Pond 1 Only

Run C assumed that only Newland Pond 1 was captured by a channel avulsion and represents the case where river training measures are implemented to prevent an avulsion into Ponds 2 and 3. Similar to Run B, the avulsion into Pond 1 occurred near SR24 Bridge, but the avulsion path re-joined the main channel further upstream near RS 21331. The results showed that up to 10 feet of average bed aggradation occurred within Pond 1 over 25 years. Average channel degradation of 2 to 3 feet occurred upstream and extended to Buchanan Lake.

Figure 30 shows thalweg profile development for Run C, also over a 50 year simulation period. In the first year (*t* = 1 year), degradation of 2 to 3 feet occurred along a reach extending 1,500 feet upstream of Pond 1. Pond 1 was computed to fill after 10 years (*t* = 10 years). Maximum degradation of 2 to 4 feet extending 11,000 feet upstream of Pond 1 occurred between year 10 and 15 of the simulation. After 15 years, the computations indicated that degradation ceased and the reach in the vicinity of SR24 Bridge began to recover through aggradation.

These results indicate that it would be beneficial to protect Ponds 2 and 3 from capture by the main channel. However, the magnitude and extent of the degradation is still significant.

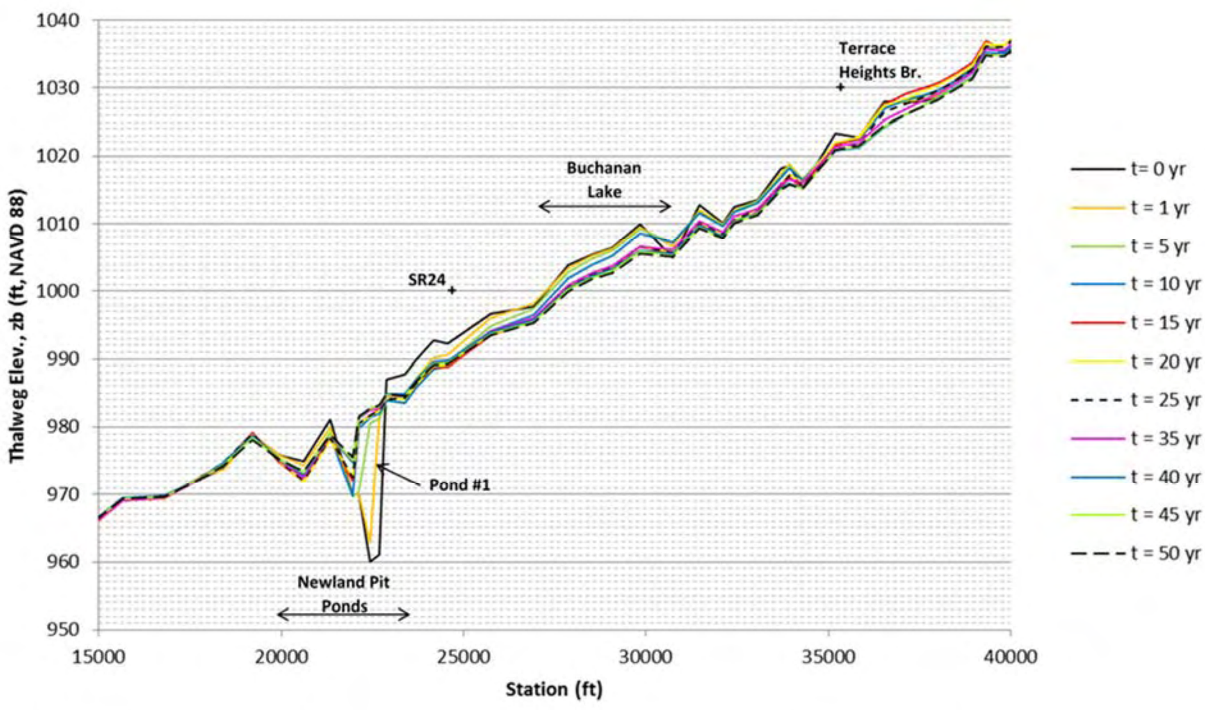

Figure 30: Run C, computed thalweg long profile adjustment for 50 year simulation

5.2.6 Bed Level Time Series at Specific Locations

Bed level changes were plotted over time at three cross section locations in order to illustrate temporal trends and the effect of peak flows on degradation rates. Hydrologic input to the SRH-1D model consisted of a continuous 25 year hydrograph spanning water years 1985 to 2009 (Hilldale and Godaire, 2010). Figure 31a-c compares changes in thalweg elevation over time for existing and proposed levee set-back conditions (Runs A, B, and C) at three cross sections located upstream of SR24 Bridge. Also shown is the continuous flow hydrograph for comparison. For reference the peaks observed during years 11 and 36 correspond to the February 2, 1996 flood of record with a discharge of 43,900 cfs.

Figure 31a illustrates channel adjustment immediately upstream of the SR24 Bridge (RS 24821.14). For existing conditions, slight degradation is computed with a trend of aggradation occurring after year 11. All proposed condition runs show more substantial degradation occurring during the first 5 years of simulation; however, only those including pit capture (Runs B and C) indicate substantial degradation greater than 3 feet during the first year. Run A shows a more gradual trend of degradation to year 11, then levels out, thus suggesting reach stabilization. Run B shows further degradation following the peak during year 11, with general reach stabilization indicated after year 25. Conversely, Run C indicates a trend of aggradation occurring after the year 11 peak.

Figure 31b and c show channel adjustment approximately 2,300 and 8,700 feet upstream of SR24 Bridge (RS 27155.89 and RS 33520.73), respectively. The same general trends as those computed downstream are observed, but simulated degradation was more gradual prior to year 11 and slightly lower in magnitude. Following year 11, only Run B showed continual degradation through the simulation period.

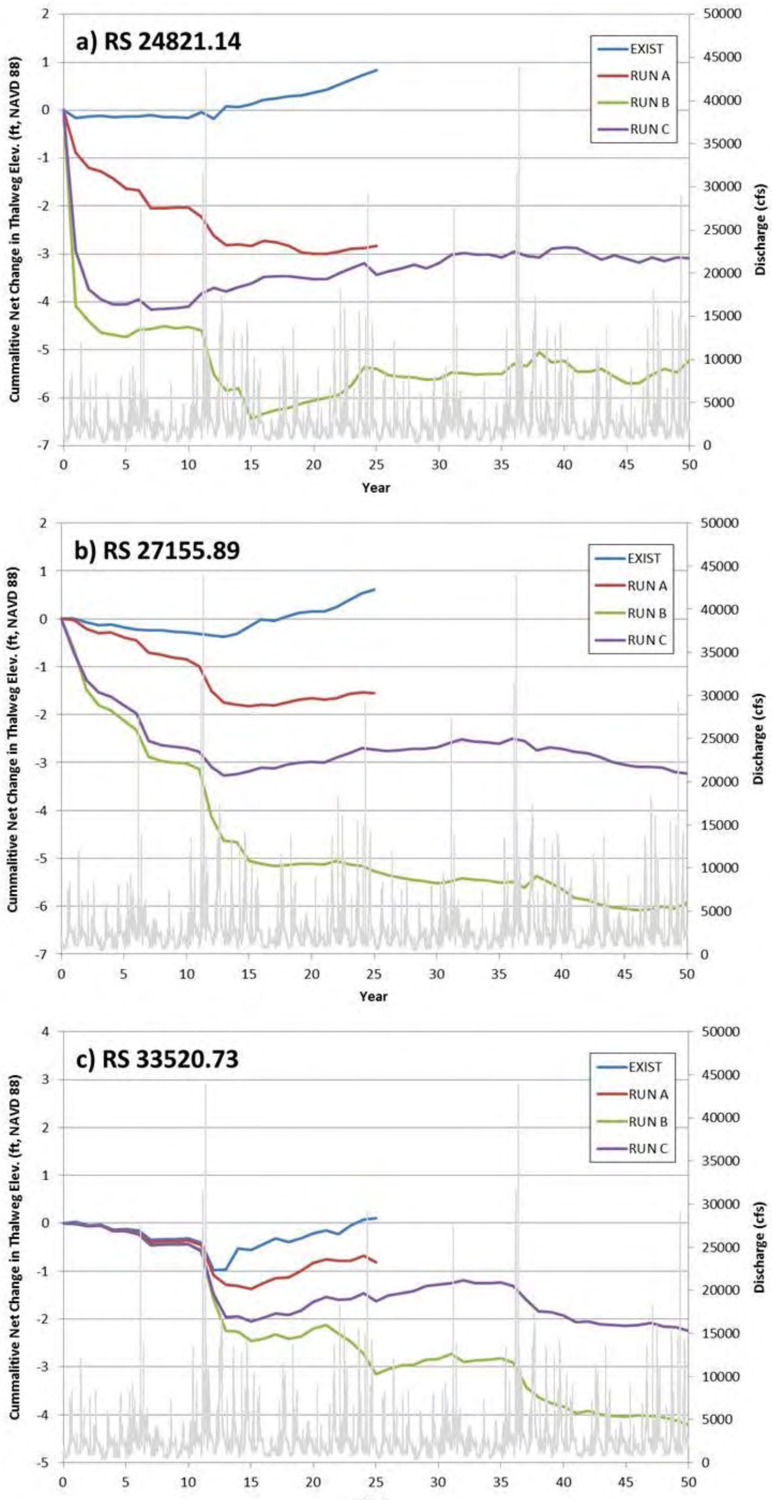

Figure 31: Time variation of computed thalweg elevation for existing and proposed conditions at three cross sections; input hydrograph shown for comparison

5.3 Assessment of Predicted Effects

This section compares the results from the geomorphic analysis (Section 4.5) with NHC's morphodynamic modeling results and previous runs by the USBR (Hilldale and Godaire, 2010) in order to reach a consensus on the magnitude of various pit capture and avulsion scenarios without mitigation. Results of the various estimates are summarized in Table 12.

The USBR study examined the hydraulic effect caused by setting back the DID #1 levee and did not consider the effect of a pit capture after the levee was removed. In effect, this scenario represents the situation of the set-back levee and the ponds being completely filled in and carrying floodplain flow (no avulsion). The NHC and USBR SRH-1D simulations both showed a levee set-back alone (without pit capture) produced 1 to 2 feet on average (maximum value of 3 feet) and extended up to 2,000 feet upstream of SR24 Bridge after 25 years. The geomorphic method (Eq. 4) can be applied to this case, by substituting the terms to represent the reduction in flow path length across the floodplain (approximately 400 feet). The resulting degradation amounts to 1.1 feet, which is in general agreement with the numerical models. This scenario is unlikely to pose a significant threat to nearby infrastructure.

The geomorphic analysis indicated a high likelihood of channel avulsion and capture of the Newland Ponds if mitigation measures are not implemented as part of the levee set-back project. The worst-case scenario involved the channel fully avulsing through all three ponds. The predicted vertical headcut was estimated to be approximately 9 feet and the resulting degradation would propagate up to 13,000 feet upstream (reaching the Federal levee near Buchanan Lake). The recovery time scale for geomorphic conditions to resume along the reach was estimated as being on the order of half a century. In comparison, NHC's SRH-1D model (Run B) predicted the average bed degradation was 3 to 5.5 feet (with maximum values of approximately 7 feet). The degradation extended upstream over 14,000 feet, (past Buchanan Lake and the Terrace Heights Bridge). The Newland Ponds had not completely filled in after a simulation time of 50 years. Again, the geomorphic method and SRH-1D model are in general agreement. Without mitigation, the degradation from this scenario could damage levees on both banks of the river upstream of SR24 Bridge and could potentially cause undermining of the Federal levee at Buchanan Lake, which would trigger another pit capture event.

The scenario of a partial avulsion into the ponds was also assessed. Using the geomorphic method, an avulsion into Pond 1 only would generate 5.8 feet of degradation. The SRH-1D model produced an average bed degradation of 2 to 3 feet, with maximum computed bed lowering of approximately 4 feet. The degradation extended approximately 11,000 feet upstream of SR24 Bridge, thus may still pose some threat to the Federal levee near Buchanan Lake. Most of the channel adjustment following pit capture, including upstream degradation and filling of Pond 1, was predicted to be completed after 15 years. However, this situation is unlikely to remain stable, since the river would quickly shift into the other ponds, triggering a full avulsion through all three ponds.

The geomorphic-based predictions and 1D model results indicated that a full avulsion and capture by the three ponds could induce significant upstream degradation and could potentially threaten the revetment protecting the Federal levee which separates the river and Buchanan Lake. Furthermore, even a partial avulsion or pit capture could result in significant bed lowering. Therefore, mitigation measures need to be incorporated into future levee set-back plans to prevent this situation from occurring.

Table 12: Summary of estimated degradation for different scenarios

Levee Set-back Scenario	Estimated Degradation	
	Geomorphic Method	SRH-1D
Ponds assumed filled-in, carrying floodplain flow	1.1 feet	1 to 2 feet average, 3 feet maximum
Full pit capture of all 3 ponds	9 feet	3 to 5 feet average, 7 feet maximum
Pit capture of Pond 1	5.8 feet	2 to 3 feet average, 4 feet maximum
Pit capture of Pond 3 only	2.3 feet	

6 FUTURE WITHOUT PROJECT

6.1 Purpose

Chapter 3 described the historical changes and physical impacts to the Yakima River due to channelization from past flood control projects. Chapter 4 and Chapter 5 reviewed the history of floodplain gravel mining on the river, developed a general process-response model to assess these effects and then quantified the potential effects of pit captures at the Newland Ponds for a number of scenarios without mitigation.

This chapter summarizes the channel stability problems along the three mile reach of the Yakima River from downstream of the Newland Ponds up to near the Terrace Heights Bridge with the present levee configuration and with no additional mitigation measures installed. This situation represents a “future without project” scenario and is intended to provide a basis for assessing the benefits and justification for conducting a comprehensive restoration and mitigation project on the river.

6.2 Recent Channel Changes

Moderate channel changes occurred in the vicinity of SR24 Bridge between 2004/2005 and 2014 (Figure 11). The most prominent change occurred upstream of the SR24 Bridge where the formerly anabranching channel shifted approximately 200 to 300 feet to the left (east), forming a single thread channel fixed against the existing Sportsman’s Park Levee. In the 6,000 foot reach downstream of SR24 Bridge adjacent to the Newland Ponds, the channel has maintained a single thread pattern fixed against the wastewater treatment plant revetment on the right (west) bank.

Figure 32 compares thalweg profiles from the 2004/2005 and 2014 bathymetric surveys. The reference channel distances are shown on the accompanying air photo. Continuous degradation of 1 to 3 feet has occurred approximately 2,000 feet upstream along the Sportsman’s Park Levee. Further upstream, at the limit of the 2014 bathymetric survey (River Station [RS] 28351), degradation of approximately 2 feet occurred. The maximum degradation (approximately 5 feet) occurred in the vicinity of the bridge and is attributed to increased scour along the Sportsman’s Park levee as the channel has recently migrated eastward.

The existing SR24 Bridge was constructed in 2005/2006. Review of as-built plans provided by the Washington State Department of Transportation indicate the base of the bridge pilings are approximately 50 to 55 feet below the current channel bed elevation and are not vulnerable to scour. Downstream of SR24 Bridge, minor degradation occurred along the 4,000 foot sub-reach adjacent to the wastewater treatment plant and Newland Ponds; however, the tailout of this reach appears to have degraded by up to 2 feet (RS 21331) since 2005. Further downstream, aggradation of up to 2 feet was observed.

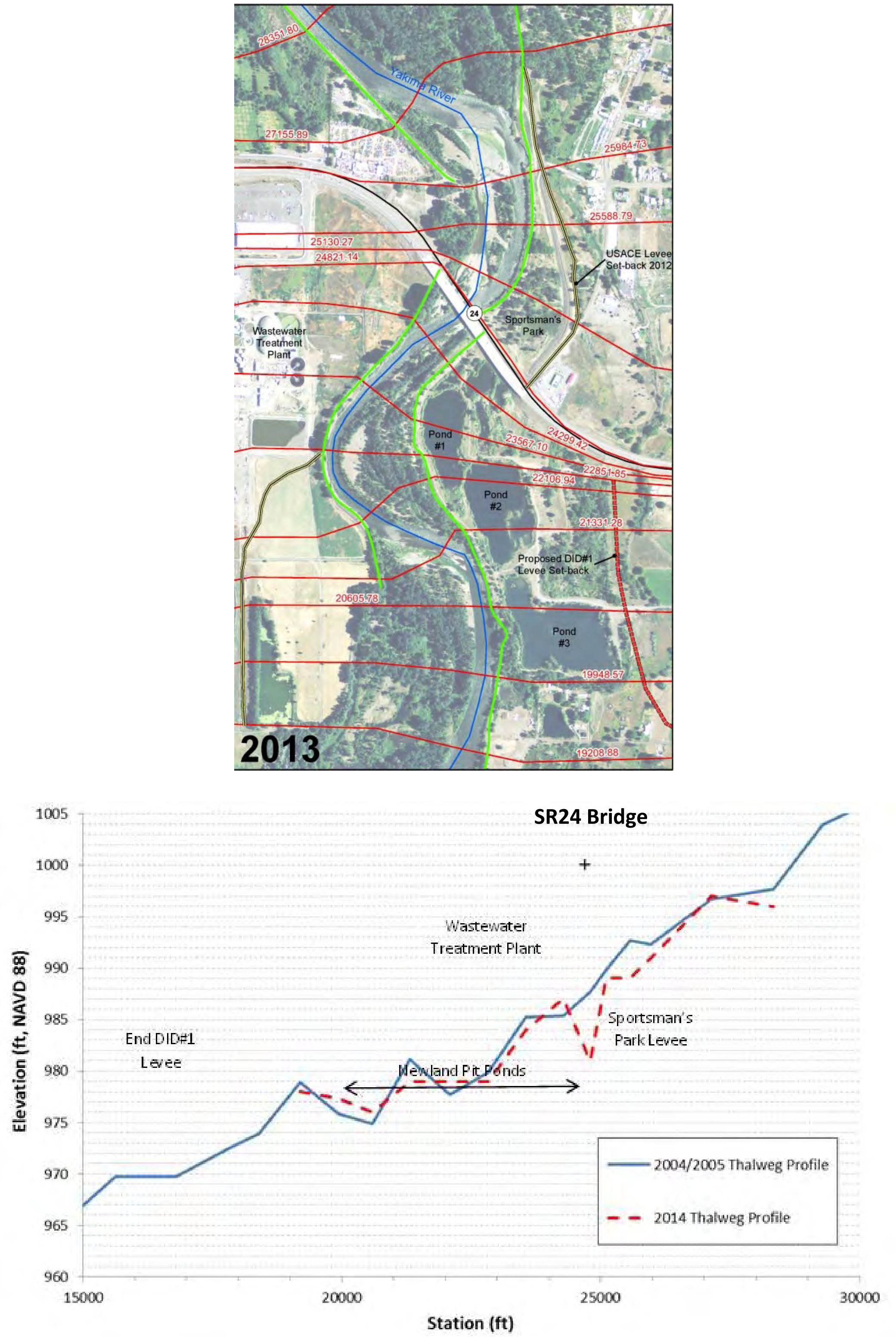

Figure 32: Comparison of channel thalweg profile from 2004/2005 and 2014 bathymetric surveys

Figure 33 compares cross sections from 2004/05 with the surveys from 2014. The location of the cross sections is shown in Figure 32. A general trend of channel degradation at the toes of fixed levees or revetments such as the wastewater treatment plant (Figure 33e) and along the Sportsman's Park Levee (Figure 33f-h) is illustrated. In contrast, along reaches lacking bank protection, lateral channel migration is apparent. On the left bank just downstream of the wastewater treatment plant (Figure 33c) approximately 20 feet of eastward bank erosion is shown.

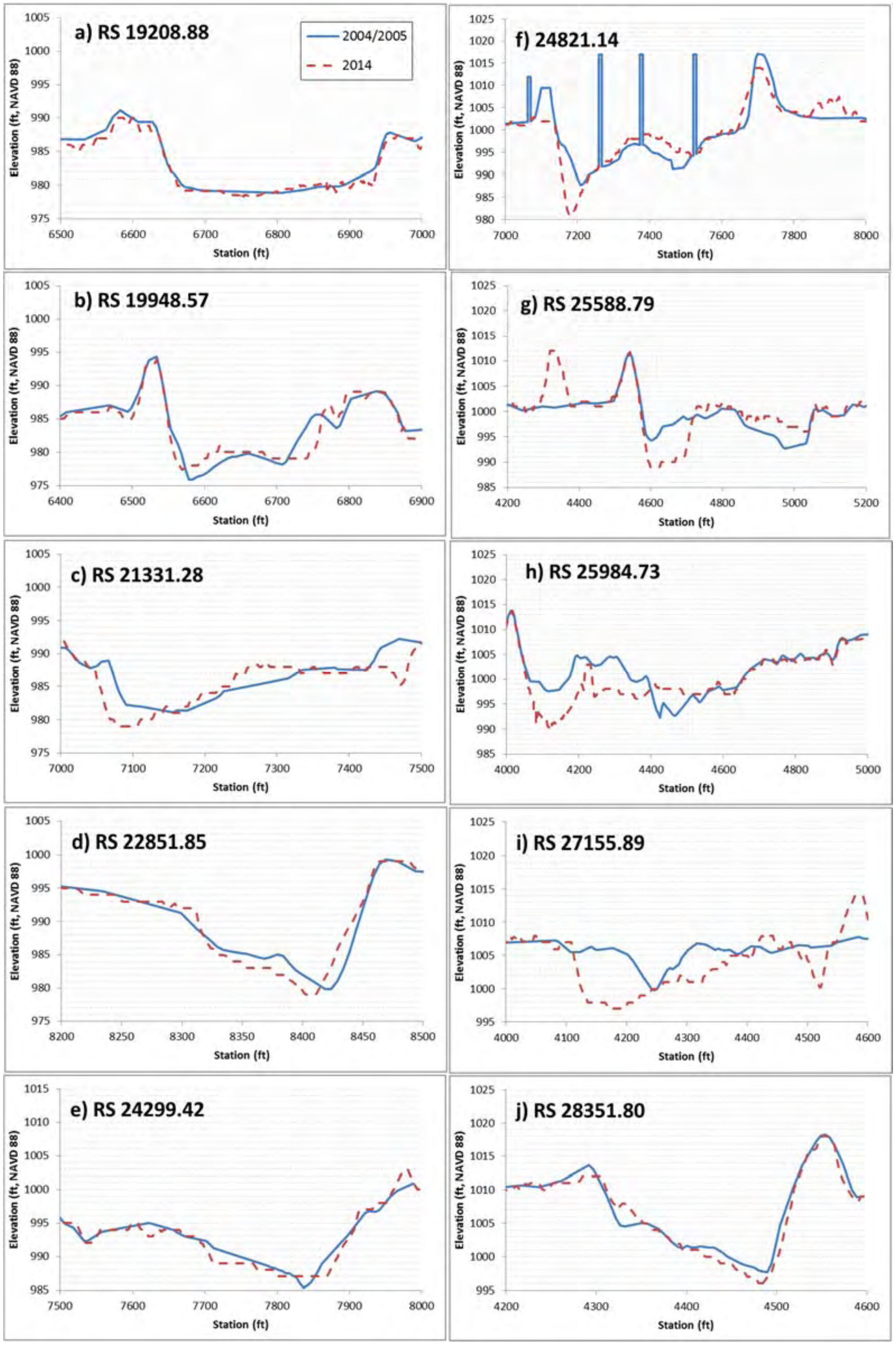

Figure 33: Cross section comparisons in project reach between 2004/05 and 2014

6.3 Future Without Project Mitigation Measures

Potential recurring channel instability problems are expected at a number of sites for the case of the "future without project" scenario. These hazards exist with the existing levee configuration and if the levees were set back without constructing any mitigation measures. Figure 34 shows the location of the sites that are discussed in this section.

6.3.1 Federal Levee near Buchanan Lake

The sharp bend and narrow, channelized section in this location (Site 1 on Figure 34) will generate deep scour along the toe of the levee during extreme floods. A breach of the levee here would cause the river to flow into Buchanan Lake, forming a lateral pit capture.

Two methods were used to evaluate scour along the right bank of the Federal Levee upstream of Buchanan Lake. The first method employs the Thorne Equation for bend scour (ISPG, 2002):

$$\frac{d}{y_1} = 1.07 - log\left[\frac{R_c}{W} - 2\right] \qquad for\ 2 < \frac{R_c}{W} < 22 \qquad \text{Eq. 9}$$

Here, d is the maximum depth of scour below the local bed elevation, y_1 is the average flow depth directly upstream of the bend, W is the width of flow, and R_c is the radius of curvature along the stream centerline (ISPG, 2002). An active channel width of 250 feet was assumed and the radius of curvature was estimated at 1,350 feet (R_c/W = 5.4). Flow depths were calculated by the HEC-RAS hydraulic model and measured immediately upstream of the bend at RS 32890.

The second method used to evaluate scour at the Federal Levee employed the Blench Equation (Eq. 3) introduced in Section 3.5.2. Scour depths estimated using the Blench equation assumed a value of 5 for F_b and a Z factor of 2. Table 13 compares scour depths computed by the Thorne and Blench Equations for the 10- and 100-year events.

Table 13: Scour Depths Calculated for Existing Conditions at the Federal Levee near Buchanan Lake

	Max. Depth of Scour Below Local Bed Elevation, *d* (feet)	
Method	10-year	100-year
Thorne Equation	8	11
Blench Equation	11	16

Both methods yield comparable results and indicate there is substantial scour potential along the Federal levee just upstream of Buchanan Lake. The condition of the riprap protection in this reach is unknown and should be checked to determine whether it is adequate to protect the levee from scour.

The geomorphic investigations and SRH-1D model showed that this section is vulnerable to additional bed lowering caused by degradation in the event of a pit capture downstream at the Newland Ponds. The amount of potential degradation was estimated to reach up to 9 feet.

6.3.2 Federal Levee Downstream of Buchanan Lake

The narrow, channelized section of the river continues downstream of Buchanan Lake for a distance of approximately 0.5 miles (Site 2 on Figure 34). The channel turns eastward, forming a moderately curved bend as it approaches Nob Hill. The bend will generate additional scour along the outer (concave) bank which forms the toe of the Federal levee. During high floods, the high velocities and scour in this channelized reach could cause undermining and breaching of the Federal levee as described above in Section 6.3.1. This section is also vulnerable to additional bed lowering caused by a pit capture at the Newland Ponds. The only difference between this site and the site in Section 6.3.1 is that failure of the levee here would not trigger a pit capture at Buchanan Lake.

6.3.3 Old Sportsman's Park Levee

Portions of the old levee are actively failing, which will allow spills to flow over the floodplain between remnants of the old levee and the new set-back levee. The reach is labelled as Site 3 on Figure 34. Photo 1 shows a location of active erosion at the old SR24 Bridge abutment which is now isolated in the channel with strong secondary currents flowing behind the structure, washing out a section of the old levee. During a flood, the resulting spill onto the floodplain will produce overbank flows between the old levee and new levee. The flow path will be unpredictable, depending on whether water breaches the old levee from the backside or is turned back into the main channel at the cross-dike. This hazard should diminish after the DID #1 levee set-back project is completed and the cross-dike is removed.

Another longer term issue is whether the meander bend upstream of SR24 will continue to shift eastwards due to growth of the point bar on the west bank. This would cause the river to flow against the new Sportsman's Park levee and become "attached" to the levee in the same manner that the channel flows against the Federal levee on the west side further upstream. This situation is illustrated hypothetically in Figure 35, which shows an eastward shift upstream and downstream of SR24 Bridge.

6.3.4 Newland Ponds

This reach is labelled as Site 4 on Figure 34. The existing DID #1 levee is exposed to direct river attack and could fail by scour or erosion during a major flood. A dike breach could allow the river to avulse into the ponds, inducing significant impacts (up to 9 feet of degradation) to the channel upstream and downstream of the ponds. Chapters 4 and 5 provide a detailed assessment of the potential channel changes that could occur from a pit capture. Figure 35 shows a hypothetical avulsion and eastward shift of the river. The magnitude of these effects is essentially the same whether the avulsion was triggered by erosion of the existing DID #1 levee or by setting back the DID #1 levee without providing suitable mitigation measures.

Figure 34: Potential channel instability hazard sites in project reach

6.3.5 Bend near WWTP Levee

This location is identified as Site 5 on Figure 34 and is along the outside of the meander bend downstream of the SR24 Bridge adjacent to the WWTP levee. Deep scour occurs along the outer (concave) portion of the bend at present due to the high curvature of the bend and the narrow channelized channel section through this reach due to the restricted opening under the SR24 Bridge.

6.3.6 Yakima Greenway

This location (Site 6 on Figure 34) is located in a reach that has experienced channel shifting in the past in response to previous gravel pit captures. The site is downstream of the Newland Ponds and would be affected by channel changes associated with a pit capture of these ponds. The magnitude of downstream degradation was estimated to be relatively low (Chapter 5). The main impact would be increased channel shifting induced from upstream changes in channel alignment. This could induce new pit captures at existing ponds in the Greenway reach.

6.3.7 Summary

Table 14 provides an overview of the hazards and the potential consequences under a "future without project" scenario. The greatest risk to future channel stability in the reach is if the main channel avulsed into the Newland Ponds which then subsequently initiated a second levee breach and pit capture at Buchanan Lake. This risk exists regardless of whether DID #1 levee is set back or not.

Table 14: Sites at risk of future channel instability

Site	Feature	Hazard	Potential Consequence
1	Buchanan Lake/Federal Levee	Scour at sharp bend in channelized reach, triggering a pit capture at Buchanan Lake. Degradation from a pit capture at Newland Ponds would exacerbate the risk of failure.	Failure of Federal levee, breach into Buchanan Lake
2	Federal Levee	Scour at outside of bend in narrow, channelized reach. Degradation from a pit capture at Newland Ponds would exacerbate the risk of failure at the upstream site.	Failure of levee from undermining
3	Old Sportsman's Park Levee	Local bank erosion induces spill and channel formation between old levee and new set-back levee.	Channel develops along new set-back levee, erosion and scour at SR24 Bridge cross-dike
4	Newland Ponds	Existing DID #1 levee between river and gravel pits could be undermined by scour and river attack.	Avulsion into pits, causing channel instability and degradation extending up to Terrace Heights Bridge
5	WWTP levee	Erosion and scour around outside of bend.	Westward migration of channel
6	Yakima Greenway	Channel instability could increase in future due to upstream changes.	Potential channel shift into ponds if river becomes de-stabilized in future

Figure 35: Future eastward shift in river alignment if no mitigation is carried out

7 MITIGATION ISSUES AND CONCEPTS

7.1 Problems and Issues

7.1.1 Background

The present Section 1135 Ecosystem Restoration Feasibility Study is intended to mitigate environmental impacts resulting from the Yakima Federally Authorized Flood Control Project (YFAFCP) and the Drainage Improvement District # 1 levee (DID #1). The project entails the setback of the DID #1 levee south of the SR24 Bridge, any other YFAFCP set-backs, and restoration both upstream and downstream of the SR24 Bridge. The goal is to restore, as much as possible, the natural river processes to reduce risk to infrastructure and fish.

Restoration plans for the river are complicated by the presence of the former gravel pits (Newland Ponds and Buchanan Lake) on the floodplain. These features have adversely impacted the natural channel and floodplain processes in the past (as documented in Chapter 3 and Chapter 4). Under existing conditions, there is a high risk that the river could breach its existing levees during a major flood event, resulting in a pit-capture event. Therefore, the ponds pose a significant risk of causing further channel incision and interrupting bed load transport processes throughout the system even if the existing levees were not set back. With the levees set back, the risk of adverse morphological impacts from pit captures would be increased. The ecosystem restoration project needs to mitigate the effects of past channelization measures as well as mitigate the additional potential effects due to floodplain gravel mining. Fortunately, mitigating the effects of past channelization works and gravel pit construction can be combined into a well-integrated approach that addresses both problems. This is because the types of impacts generated by a pit capture (channel incision, interruption of sediment transport processes) are similar to many of the processes that result from channelization.

7.1.2 Objectives of Mitigation Measures

The overarching goals of the project are twofold:

1. Allow the river to develop a more natural alluvial channel pattern that will improve channel habitat.

2. Reduce flood and erosion hazard risks to infrastructure.

Specific project objectives include:

1. Restore the channel pattern to a more natural state that existed prior to major channelization works from past flood control projects.

2. Limit continuous flow against planned or existing levees.

3. Increase the floodplain conveyance by setting back the levees while limiting any adverse morphological impacts from past gravel mining operations on the floodplain.

7.1.3 Method of Assessment

The "process-response model" developed during the geomorphic investigations in Chapter 3 provides a good basis for defining the requirements for some of the mitigation concepts that have been proposed in this chapter. Additional 1D morphodynamic modeling (SRH-1D) and 2D hydrodynamic modeling (SRH-2D) was also carried out to assess the effectiveness of specific mitigation alternatives. The model runs were conducted primarily to test and compare different alternatives and were not intended to provide final designs of selected alternatives. The model results are summarized in Appendix B at the end of this report. Additional 2D morphodynamic modeling should be carried out before detailed designs of mitigation measures are prepared.

The mitigation alternatives were compared using a multi-criteria analysis by using a number of hydraulic and geomorphic criteria that were assessed qualitatively. This method provided a basis for ranking and comparing the alternatives. Other biological and social-economic criteria may also be applied to finalize the alternatives.

7.1.4 Spatial Extent

The engineering mitigation measures should extend at least 1 mile downstream of SR24 Bridge and at least 2 miles upstream of SR24 Bridge to the Terrace Heights Bridge. The mitigation measures have been described below in three sub-reaches:

- Reach A: Newland Ponds to SR24 Bridge
- Reach B: Sportsman's Park levee
- Reach C: Upstream of Sportsman's Park to Terrance Heights Bridge

7.2 Reach A: Newland Ponds

7.2.1 Overview of Present Situation

A pit capture and avulsion of the river into the Newland Ponds would trigger up to 9 feet of degradation upstream of SR24, creating additional channel incision upstream to the Terrace Heights Bridge, increasing the risk of a failure of the Federal levee and a breach into Buchanan Lake. Due to the large volume of the Newland Ponds, the effect of a pit capture on river morphology would persist for several decades.

7.2.2 Purpose of Mitigation Measures

The mitigation measures are intended to restore and re-connect the floodplain downstream of the SR24 Bridge with the active channel of the Yakima River. In order to accomplish this, the adverse impacts associated with a pit capture (upstream degradation and channel incision, downstream reduction in sediment supply) also need to be mitigated.

7.2.3 Mitigation Alternatives

The investigations described in Chapter 4 and Chapter 5 indicate that the potential upstream effects of a pit capture require mitigation measures to be implemented. Therefore, simply setting back the levees without mitigation measures is not considered a feasible alternative. However, this scenario is referenced in this section in order to gage the effectiveness of the proposed mitigation alternatives.

The potential mitigation alternatives in Reach A that were considered include:

- Alternative A1: Fill in the ponds to the thalweg level of the main channel and allow the river to naturally re-occupy the floodplain.
- Alternative A2: Fill-in the ponds to the adjacent floodplain level to restore east bank floodplain conveyance.
- Alternative A3: Install a grade control structure on the floodplain upstream of SR24 Bridge (near the present cross-dike) in conjunction with Alternative A2. The grade control sill would be set below the existing floodplain level in order to prevent any flow obstruction. However, it would be designed to arrest development of a new low-flow channel.

In addition, other complementary measures have been defined that would be carried out in combination with all of the alternatives. These measures include:

- Retaining portions of the riprap on the DID #1 levee near Pond 2 and Pond 3 (A4).
- Excavating the point bar on the west bank opposite Pond 3 (A5).

The purpose of these complementary measures is to reduce the attack on the east bank.

The alternatives are illustrated in Figure 36. All of these alternatives assume that the temporary cross-dike on the upstream side of the SR24 Bridge is removed.

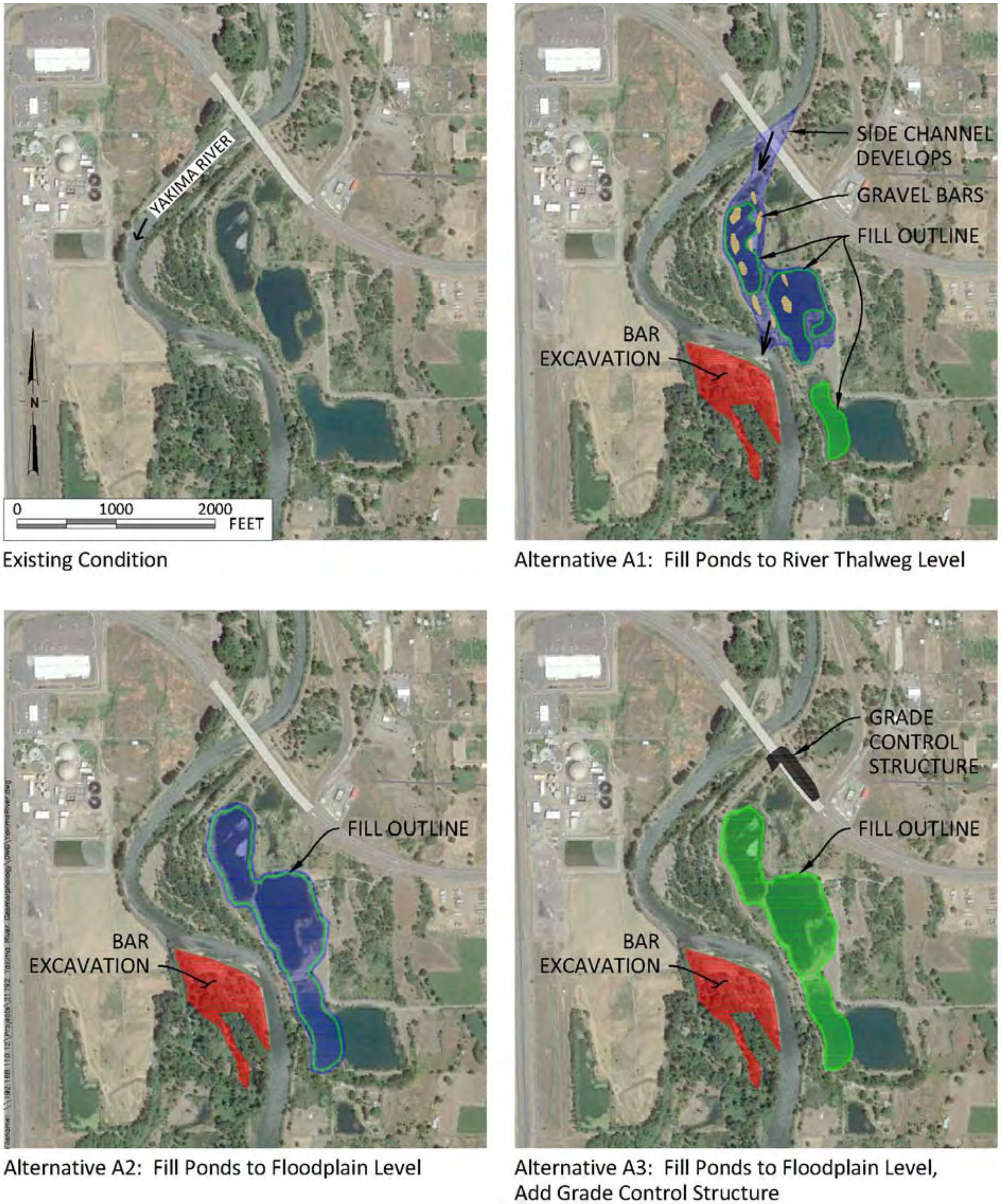

Figure 36: Mitigation alternative on Reach A, Newland Ponds

7.2.4 Alternative A1: Fill Ponds to Thalweg Level

This alternative involves filling-in Pond 1 and Pond 2 and the western portion of Pond 3 using material salvaged from the old levees, from other channel excavations and by re-grading portions of the adjacent floodplain. Figure 37 shows the proposed extent and filling level. The quantity of material was estimated by the County and is summarized in Table 15.

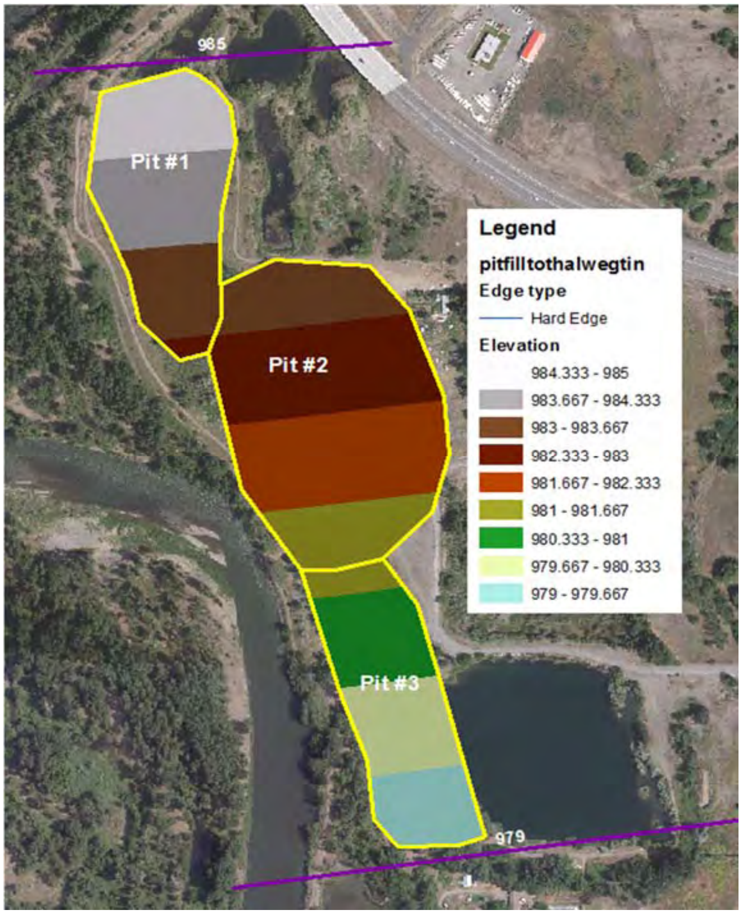

Figure 37: Proposed pond filling elevations – Alternative A1

Table 15: Proposed infilling of Newland Ponds – Alternative A1

Pond	Infill Volume (cubic yards)	Fill Elevation (feet)
1	175,376	987 to 990
2	503,211	986 to 988
3	187,770	984 to 986
Total	866,357	

Note: quantities provided by Yakima County

With the ponds filled to the thalweg level of the channel, we expect that an active side channel would develop across the floodplain in a relatively short time-frame (one to three years, depending on the magnitude of the flood flows after the cross-dike and DID #1 levee are removed). The location of the inlet and outlet of this channel would depend on the local bank topography and erodibility of the bank sediments. The locations sketched on Figure 36 are approximate. The path length for flow across the partially filled floodplain is considerably shorter than the length around the existing bend, so that the longitudinal slope across the floodplain will be steeper than in the main channel:

- Main channel slope: 0.0026
- Slope across filled-in ponds: 0.0038

Once opened up, the side channel would convey a significant portion of the main river's flow and sediment load. The process of side channel formation would be analogous to a partial chute cutoff that develops across a meander bend. Figure 38 shows an example. Preliminary 1D hydraulic model runs indicated the side channel would convey approximately 60% of the main channel flow. Under these conditions the side channel would evolve to having a stable top width of approximately 200 feet, a mean depth of 8 feet and a mean velocity of 6.5 feet/second (based on Eq. 1, Parker regime equation). The initially steeper slope across the floodplain would trigger a corresponding channel response at the upstream inlet, with the water surface and bed level lowering to re-establish the initial slope. This would generate approximately 3.5 feet of degradation upstream of SR24 Bridge.

The channel would transport gravel and sand bed load from upstream and would shift laterally across the floodplain, re-working the floodplain sediments which would maximize re-connection of the floodplain into the active channel zone. The existing channel on the west side of the river, downstream of the SR24 Bridge will become shallower in response to the decreased velocity and flows. This is likely to initiate a more complex pattern of bars through this branch.

Figure 38: Example of a chute cutoff side channel development at a bend

7.2.5 Alternative A2: Fill Ponds to Floodplain Level

The alternative would involve filling Ponds 1 and 2 to the adjacent floodplain level. The amount of infilling would be approximately 8 feet higher than in Alternative 1. The corresponding total fill volume would be approximately 1,021,000 cubic yards. Four different floodplain pond filling scenarios were investigated in the SRH-2D model (refer to Appendix B for details). The amount of flow on the floodplain ranged up to 23% during a 10-year flood and up to 50% during a 100-year flood. The overbank velocities were found to be very high (up to 14 feet/second), particularly along the bank at the point of spilling from the main channel (Figure 39).

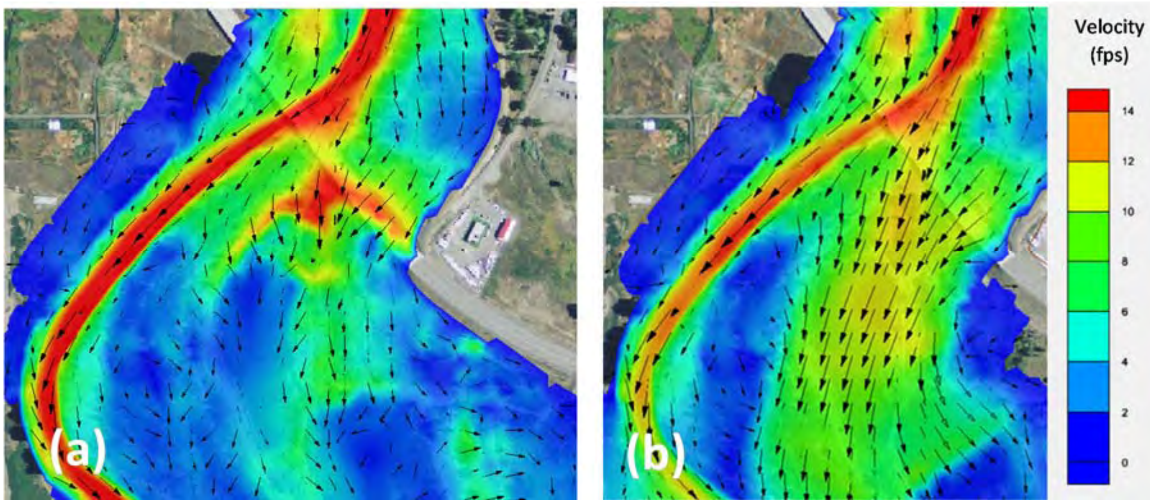

Figure 39: Velocity field on floodplain after pond filling: 10-year flood condition

The bed shear stress was also computed and was found to be sufficient to erode the gravel and sand sized sediments on the river bank. Given sufficient time, a side channel would develop across the floodplain, similar to the situation with Alternative A1. The time frame for this to occur would depend on the occurrence of high floods (10-year return period or greater). During the initial stage, the increased floodplain conveyance would generate approximately 2 feet of degradation upstream of SR24 Bridge. However, over time, as the side channel developed the bed lowering would be similar to the situation in Alternative A1 (3.5 feet). Therefore, this alternative would not have any significant advantage over Alternative A1 and would be considerably more expensive since it would require approximately 150,000 cubic yards more material.

7.2.6 Alternative A3: Grade Control with Pond Infilling to Floodplain Level

This alternative is intended to ensure that the restored floodplain can convey overbank flows but will not form a major active side channel, at least in the medium term (several years). The 2D hydraulic modeling (Appendix B) showed that a grade control structure would be required to limit the overbank flow through the filled-in ponds. In order to accomplish this, the cross-dike would be replaced with a buried riprap ground sill that would limit the amount of flow through the left overbank. The structure would allow floodplain flows under the bridge but would prevent a permanent side channel from developing across the floodplain. The crest elevation of the sill would be set at elevation 994 feet, which is below the existing floodplain level upstream of the existing cross-dike. The sill would start at the bridge abutment/new levee and would extend 600 feet to the upstream termination of the existing DID #1 levee. The sill would then turn and follow the alignment of the old levee for a further 460 feet. An intake was included to provide inflows to enter into the restored and re-graded floodplain downstream of the bridge. The notch elevation was tentatively set at elevation 988 feet.

The sill would have a thickness of 5 feet and will need to be constructed of heavy riprap. We have tentatively proposed a D_{50} rock size of 2 feet. A conceptual layout of the sill is shown in Figure 40.

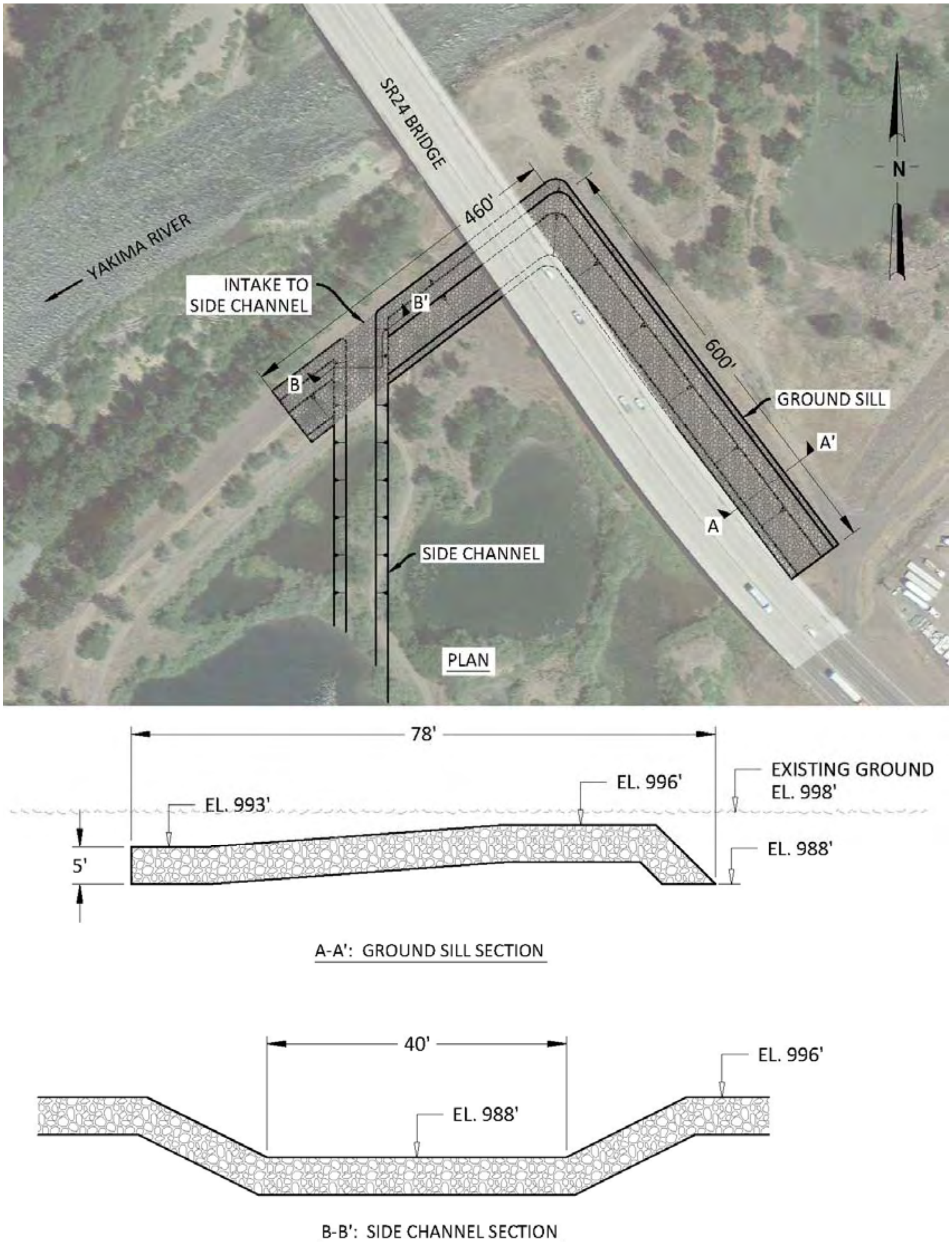

Figure 40: Concept of a ground sill to limit degradation on the east floodplain upstream of SR24 Bridge

This alternative would achieve the goal of restoring floodplain conveyance, but would maintain the river in a relatively narrow single-thread channel through this reach. The sill would also add a significant cost component and would require periodic maintenance after high floods.

7.2.7 Other Complementary Measures

Two complementary measures have been included in this reach (identified as A4 and A5). Alternative A4 involves retaining the existing riprap along 1,200 feet of the DID #1 levee near Pond 3 to the top of bank level. The retained revetment will help to prevent future eastward channel migration towards Pond 3. Alternative A5 involves excavating the point bar (including a pilot channel) on the west bank to reduce flow impingement against the bank near Pond 3. The proposed excavation level on the bar surface was tentatively set to 5 feet below the 2 year flood level. The total volume of excavation was estimated by Yakima County to be 71,400 cubic yards.

Figure 41: Alternative A4: Excavation on west bank point bar

It is expected that the gravel point bar would eventually aggrade with gravel and sand over a period of years. The measure would need to be carried out in combination with the other alternatives. The rate of infilling would be greatest if it were implemented with Alternative A2, since the river would continue to flow in a single thread channel under the bridge. If the measure were combined with Alternative A1, the rate of infilling would be reduced and it is likely that the curvature of the bend would be reduced, resulting in a long-term reduction in the flow attack against the east bank near Pond 3. Therefore, Alternative A4 would be most effective if it were combined with Alternative A1.

7.2.8 Assessment of Alternatives

The effect of the mitigation measures was assessed considering the following criteria:

- Limit upstream degradation
- Limit downstream impacts
- Restore natural channel characteristics
- Restore channel/floodplain connectivity
- Restore sediment balance in reach
- Limit future maintenance requirements

Table 16 summarizes the expected channel response on the floodplain and the estimated degradation associated with each alternative. The table also describes the effects that could occur without any mitigation as a comparison.

Table 16: Upstream degradation with mitigation alternatives at Newland Ponds

Mitigation Alternative	Hydraulic/morphologic response at ponds	Channel response upstream of SR24 Bridge
Alternative A1	Re-activation of floodplain, side channel development, reduced scour in main channel under SR24 Bridge	3.5 feet of bed lowering in response to lower flood levels
Alternative A2	Increased floodplain conveyance, re-activation of floodplain after high flood spills incise channel through floodplain	2 to 3.5 feet of bed lowering in response to lower flood levels
Alternative A3	Increased floodplain conveyance	2 feet of bed lowering in response to lower flood levels
No mitigation	Avulsion through Newland Ponds	Up to 9 feet (avulsion through 3 ponds)

Alternative A1 (pond filling to thalweg level) is the better alternative in terms of achieving the project objectives of restoring the natural channel/floodplain features and would also be lower in cost than Alternatives A2 or A3, since the volume of filling is less and it avoids placing an armored ground sill. The difference in potential upstream degradation between Alternatives A1 and A2 is minor since it is expected that a new side channel will eventually develop through the floodplain in either case.

Therefore, the adopted engineering mitigation measures in this reach include A1 along with A4 and A5. These measures should be implemented before the cross-dike is removed. There would be an advantage to leaving a portion of the buried section of the cross-dike near the SR24 Bridge abutment in-place to further minimize the risk of scour at the abutment. Further scour protection investigations and measures to incorporate sections of the existing structure should be carried out during final design.

7.3 Reach B: Upstream of SR24 Bridge – Old Sportsman's Park Levee

7.3.1 Overview of Present Situation

The old Sportsman's Park levee is situated along the outer (concave) bank of a meander bend and confines the river in a narrow, channelized section for a distance of 2,200 feet. The channel width through this reach is approximately half of its unconfined regime width. The new armored set-back levee which was constructed in 2010 eliminates the need to maintain the old structure. Removing the old levee will allow the river to widen and to re-establish natural bend processes in the reach. The driving forces that govern channel processes in this reach include:

- Accretion on the point bar on the west side of the river which deflects the flow towards the east bank (Figure 42).
- The alignment of the single-thread channelized reach upstream, which encourages meander development and directs the flow towards the east bank (Figure 42).
- The narrow channelized section through the reach which promotes scour along the outer bank leading to rapid, unpredictable bank erosion.

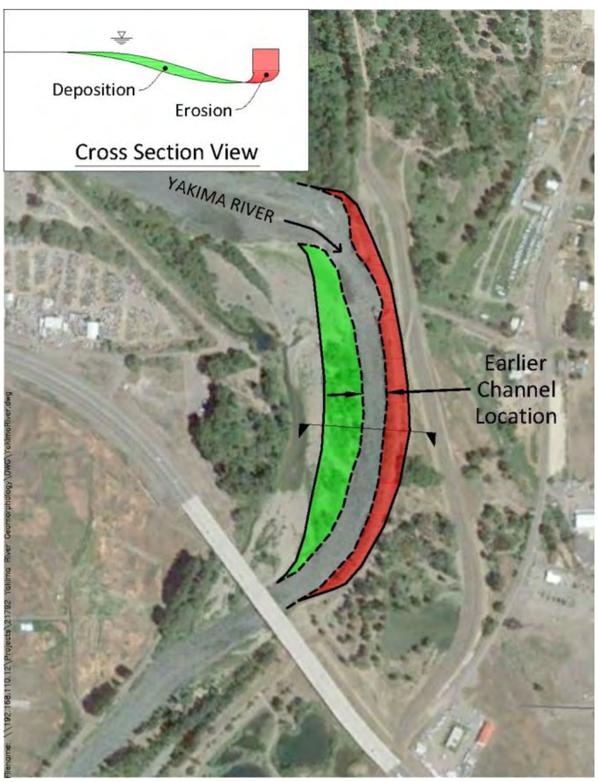

Figure 42: Effect of channel alignment and point bar deposition on erosion processes upstream of SR24 Bridge

However, if the old levee and the cross-dike are removed without other mitigation measures, it is likely that the river would shift towards the new levee and eventually flow directly against it for a considerable distance (more than 1,000 feet). The river could also be directed against the SR24 Bridge abutment. Once the channel flows against a levee, additional local scour occurs and the channel tends to persist against the structure. This would result in the river once again being continuously confined by a hard structure, similar to the present situation.

7.3.2 Purpose of Mitigation Measures

The purpose of the additional engineering measures is to minimize the risk of the channel shifting against the new levee after the old Sportsman's Park levee is removed, while restoring the bank and allowing the river to re-establish natural channel processes through the bend.

7.3.3 Mitigation Alternatives

Figure 43 shows sketches of the alternatives that were considered for this reach. The alternatives include:

- Alternative B1: Remove old levee and existing bank protection.
- Alternative B2: Remove old levee, leave existing bank protection in-place up to mean annual water level.
- Alternative B3: Remove old levee, maintain existing bank alignment using intermittent training measures.
- Alternative B4: Remove old levee, add structures on new levee to keep river away from new levee.

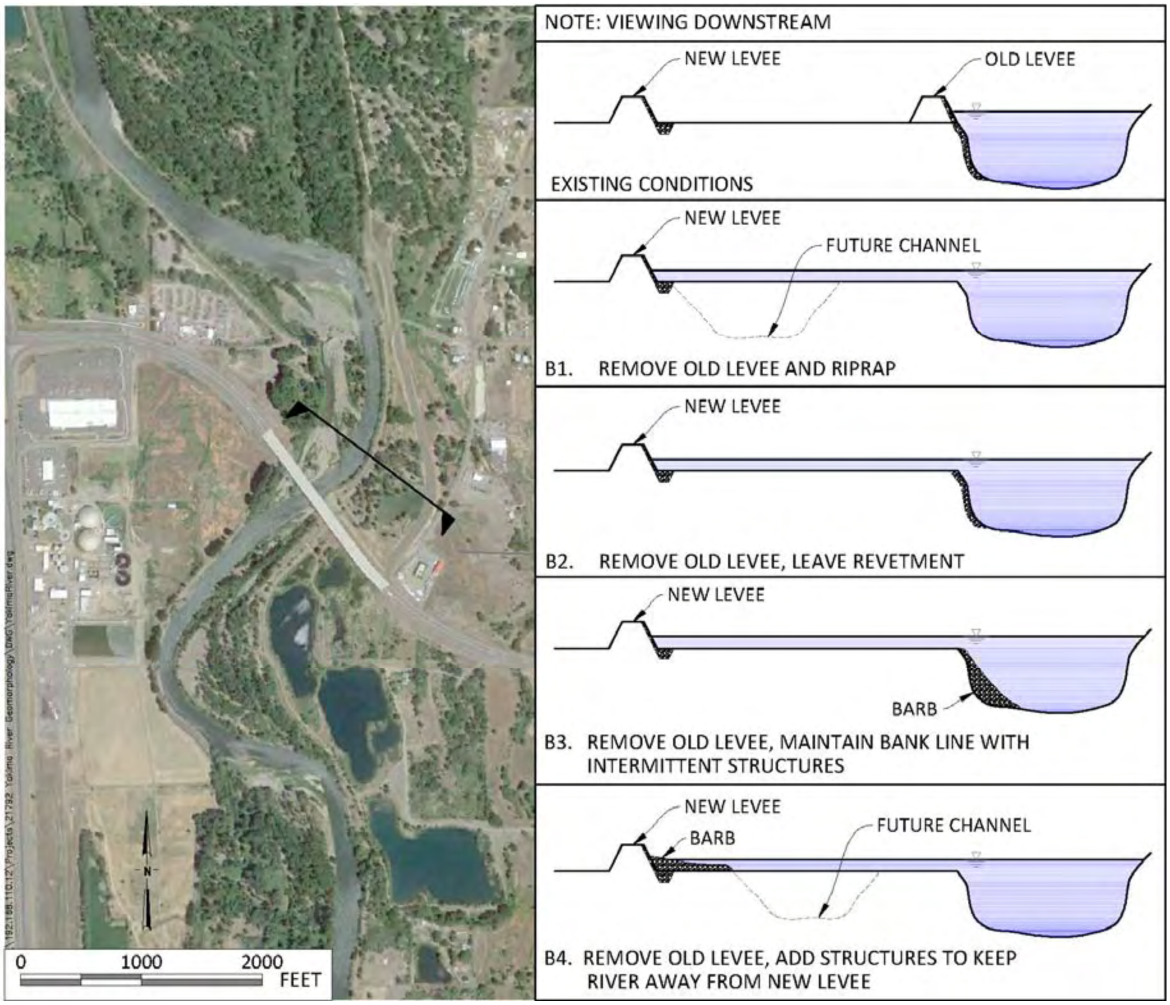

Figure 43: Mitigation alternatives on Reach B, old Sportsman's Park levee

7.3.4 Alternative B1: Remove Old Levee and Existing Bank Riprap

In this case, the existing riprap (including material presently underwater) and the levee material would be removed from the active channel zone. The underlying fine-grained bank material would be exposed to the flow, causing the banks to undergo relatively rapid retreat. This is already happening near the upstream end of the structure where local scour at the old SR24 Bridge abutment has triggered local failures. Without other mitigation measures, it is likely the river would eventually shift eastwards and flow against the new levee over a length of at least 900 feet.

7.3.5 Alternative B2: Retain a Portion of the Bank Riprap

In Alternative B2, the bank protection below the mean annual water level would be left in-place. This would restore the floodplain conveyance and would restore the upper bank to allow for future channel adjustment. However, retaining the lower portion of the riprap would limit the initial rate of bank retreat. This would avoid a sudden shock to the system resulting from complete removal of protection. Over time, the natural bend processes would dominate, allowing the river bank to migrate eastward.

7.3.6 Alternative B3: Maintain Existing Bank Alignment with Intermittent Structures

This would involve removing the existing bank protection as described in Alternative B2 but would include installing a number of spurs or barbs along the bank to maintain the overall bank alignment. Bank erosion and bank retreat in between the structures would occur, creating a more natural bank than under existing conditions. However, the river would still be confined in a narrow, single thread channel.

7.3.7 Alternative 4: Add Intermittent Structures to New Levee

This would involve removing the existing riprap along the bankline and installing a series of spurs or barbs on the new levee to prevent the river from flowing directly against the structure. This would allow the river to develop its full regime width and re-establish the natural bend processes in the reach. The structures on the new levee could be implemented at a time after the old levee is removed on the basis of monitoring the rate of bank retreat.

7.3.8 Complementary Measures

In addition, three complementary measures are included in this reach. Alternative B5 involves removal of the lower 350 feet of the Federal levee on the west bank near the Nob Hill Wrecking Yard and replacement with a set-back levee (**Figure 44**). Removal of the Nob Hill levee segment will reduce the existing geomorphic constriction, allow for more lateral channel migration, and help restore natural channel processes.

Alternative B6 involves excavating gravel from the point bar on the west bank to reduce flow impingement against east bank. The point bar on the west side of the river has grown substantially since 2006, causing the river to shift eastward towards the Sportsman's Park levee. The entrance to the existing side channel on the inner (west) side of the bar complex is presently obstructed by gravel deposits. Part of measure B5 involves excavating the entrance to the side channel to re-open the flow path. The 2-year flood elevation is estimated to be at elevation 1003.3 feet near the site (RS 25985). The present bar surface varies in elevation between 998 and 1006 feet. It is proposed to excavate the bar down to elevation 999 feet (4 feet below the 2-year flood). The total volume of excavation was estimated by Yakima County to be 25,500 cubic yards. Given the meander pattern, it is expected that the bar will re-form over time. Therefore, in order to be effective, this excavation will have to be repeated periodically. There may also be a need to remove gravel from the downstream end of the side channel to ensure the outlet remains un-obstructed.

The old SR24 Bridge abutment on the east bank is generating local bank erosion and scour which is accelerating channel shifting to the east and could deflect the flow towards the new levee (Photo 1). Alternative B7 involves removing the above-water portion of this structure.

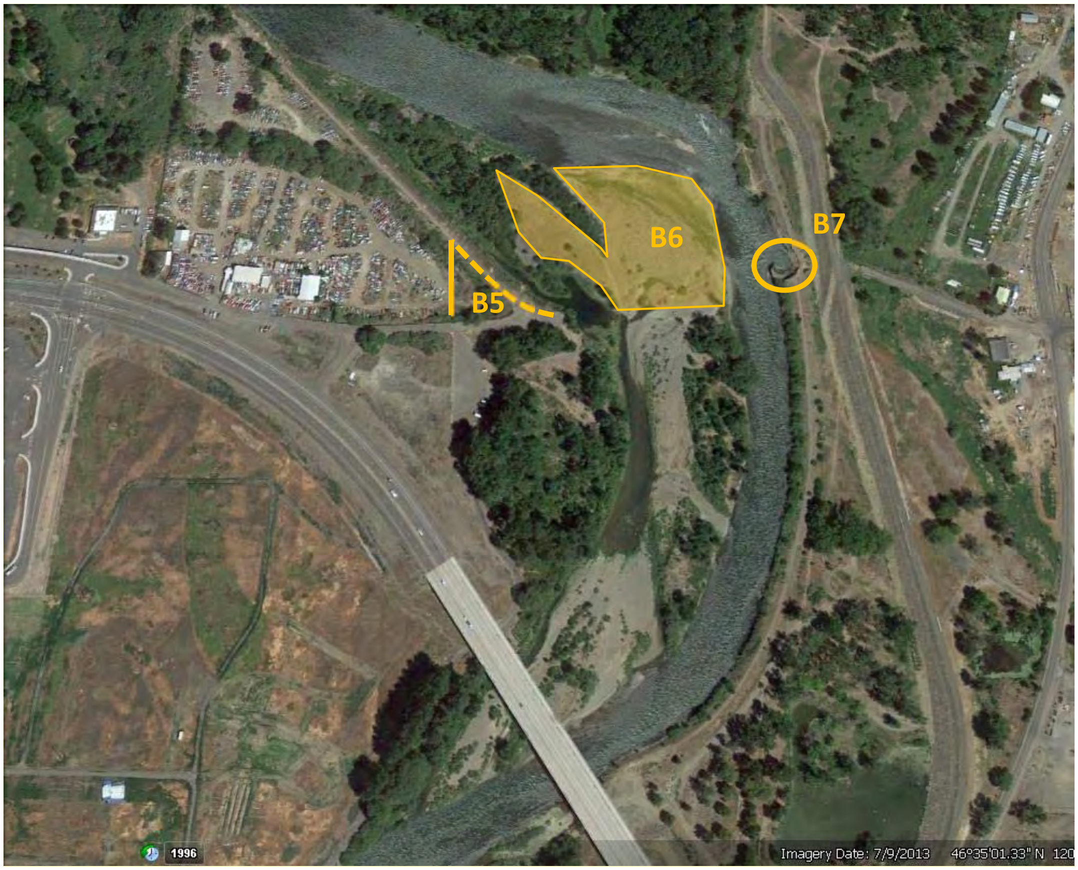

Figure 44: Alternatives (B5) Removal of Nob Hill levee segment, (B6) gravel bar excavation on west bank point bar, and (B7) removal of old SR24 abutment.

7.3.9 Assessment of Alternatives

The alternative measures were assessed by considering the following criteria:

- Restore natural bend/floodplain processes.
- Reduce the chance of the river shifting against the new levee.
- Promote formation of a natural unconfined channel.
- Improve flow approach to SR24 Bridge.

Alternative B1 meets the main criterion of restoring a natural bank and river bend but has a high likelihood of the river migrating relatively quickly against the new set-back levee in the near future. Alternative B2 would also meet these criteria but is intended to provide a more attenuated response than Alternative B1. Alternative B3 would still confine the river into a narrow channel and would not improve the flow conditions at SR24 Bridge and does not meet the mitigation criteria. Therefore, we propose utilizing Alternative B2 for this reach in the intermediate term. In the longer term, it may be necessary to install a series of spurs along the new levee. These measures could be implemented at a later date if monitoring indicates the rate of channel migration is accelerating. However, removing the Nob Hill levee segment as well as gravel from the point bar on the west bank (Alternatives B5 and B6) should reduce the rate of eastward channel migration at least in the short-term, until accretion on the point bar deflects the flow eastward. The upstream channel re-alignment alternatives described in Reach C below will also help to reduce the eastward migration towards the levee.

7.4 Reach C: Sportsman's Park to Terrace Heights Bridge

7.4.1 Overview of Present Situation

The river flows continuously against the Federal levee in a single, narrow channel. Deep scour occurs along the base of the levee threatening to create a breach and lateral pit capture into Buchanan Lake. Floodplain sedimentation on the island on the east side of the river also contributes to maintaining the present channel alignment.

7.4.2 Purpose of Mitigation Measures

The purpose of these additional mitigation measures is to reduce the impact of levee confinement in order to promote a more natural, complex channel to develop through the reach. The objectives include:

- Restore the river to an anabranched network, similar to its pattern prior to levee confinement and channelization (e.g. Figure 9).
- Mitigate the high velocities and scour that presently occurs along the base of the Federal levee.
- Mitigate against potential downstream degradation effects.

7.4.3 Mitigation Alternatives

The primary mitigation measure in Reach C involves excavating a new 5,000 foot long channel through the island complex. The alignment of the channel is shown in Figure 45. The downstream end of the channel follows an existing silted-in side channel. The new channel then turns across the central portion of the island and then splits into two branches near the head of the island. This alignment dissects the island and was intended to minimize the risk of the channel shifting against the existing levee along the east bank. Other possible alignments are also feasible. Anchored large woody debris would be placed along the new channel at strategic locations. Local excavation would also be necessary at the entrances to distributary branch channels along the route. Additional excavation is proposed at the head of the island to ensure there is a large conveyance section to capture flow from the main channel.

Figure 45: Side channel alternatives in Reach C

The proposed channel has an overall bed slope of 0.0028. Table 17 provides tentative invert levels for the downstream and upstream ends of the channel.

Table 17: Profile of side channel

Location	River Station (feet)	Invert Elevation (feet)
Downstream end	27155.9	1001
Upstream end	32154.4	1015

Two alternatives were assessed for this conceptual design phase:

- Alternative C1: 120 feet base width x 6 feet depth at bankfull stage. The total volume of material is approximately 135,000 cubic yards.
- Alternative C2: 220 feet base width x 6 feet depth at bankfull stage. The total volume of material excavated from the channel was estimated by Yakima County to be 220,000 cubic yards.

In addition, it is proposed to install a series of low spurs along the existing Federal levee adjacent to Buchanan Lake. This additional measure is labelled C3 in Figure 45. The purpose of these structures is twofold:

- To assist in directing flow into the new side channels excavated through the island.
- To deflect the thalweg of the main channel away from the base of the Federal levee.

The spurs are tentatively located at two sites (six at the upstream site and three at the downstream site). The spacing between the spurs is 120 feet. Each structure is approximately 40 feet long, with the top of the spur set near the 2-year flood stage. Figure 46 illustrates the spur concept. The design velocity along the toe of the levee was estimated from the HEC-RAS model to be approximately 15.5 feet/second. The spurs would need to be constructed of large riprap; for initial planning purposes, we have adopted a tentative D_{50} size of 3.3 feet (1.4 ton).

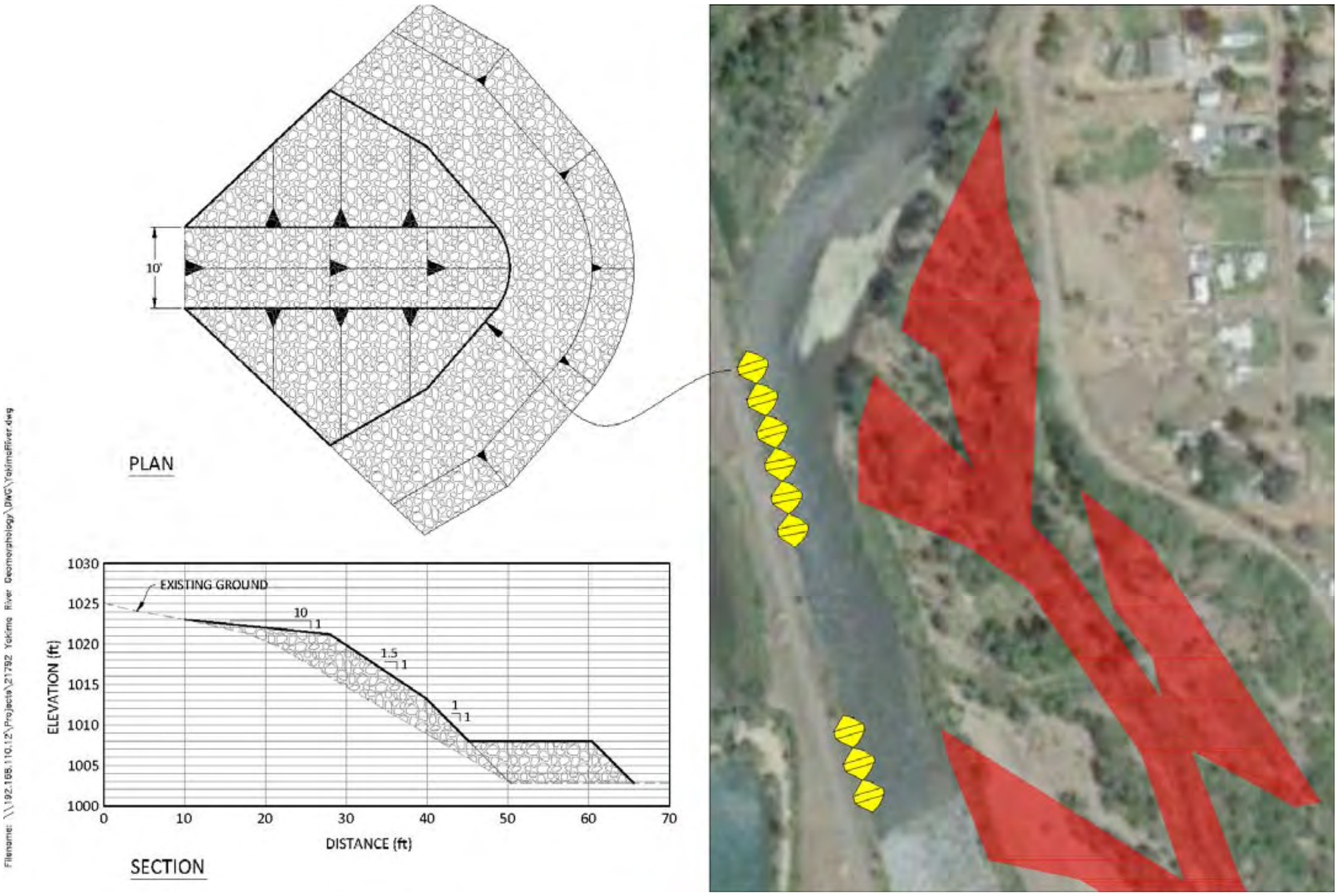

Figure 46: Conceptual design of spurs along Federal levee near Buchanan Lake

7.4.4 Assessment of Side Channel Alternatives

The side channel alternatives were compared in terms of:

- Effectiveness in reducing scour at the federal levee
- Effectiveness in attenuating upstream degradation originating at the Newland Ponds.

Table 18 summarizes computed scour depths using the Blench Equation (Eq. 3 introduced in Section 3.5.2) for the 100-year event assuming a value of 5 for F_b (cobble-sized bed material) and a Z factor of 2. Maximum scour depths (d) were estimated by subtracting computed values of y_r from average 100-year flow depth in more uniform reaches, or approximately 20 feet.

Table 18: Scour Depths Calculated Using the Blench Equation (ATC, 2004)

Condition	Q channel (cfs)	q (ft^2/s)	y_r (feet)	d (feet)
Existing	43000	172	36	16
C1: 120-ft Side Channel	37044	148	33	13
C2: 220-ft Side Channel	31118	124	29	9

The Blench equation shows that the 120 foot side channel (Alternative C1) would reduce the scour depth by 3 feet, while the 220 foot side channel (Alternative C2) would reduce the scour depth by 7 feet.

Effects of the side channels on upstream degradation were evaluated with two additional SRH-1D model simulations. The two simulations (Run D and Run E) consisted of adding a simplified, trapezoidal channel to the left bank between RS 27155 and 32154 to the full avulsion scenario where no mitigation is implemented and full avulsion into the Newland Ponds occurs (Run B, as described in Chapter 5). In Run D, the channel was assumed to have a top width of 120 feet and a depth of 6 feet. In Run E, the channel width was increased to 220 feet with the depth remaining at 6 feet. Details of this analysis are discussed in Appendix B.

Figure 47 compares changes in the thalweg elevation over time at three locations adjacent to the Sportsman's Park Island for the full avulsion case without mitigation (Run B) and with excavated 120 foot (Run D) and 220 foot (Run E) wide side channels added. Results indicate that the excavated side channels will slow the upstream progression of the headcut initiated by the pit capture at the Newland Ponds but does not completely eliminate it. Over a longer term period (greater than 50 years), the model indicates degradation would continue; however, long-term predictions from a 1D model in what would become a dynamic two-dimensional reach are uncertain. Empirical hydraulic geometry equations indicate that with a 220 foot side channel the river would be able to maintain two stable approximately equal branch channels, rather than flow in the single channel that presently exists, thus attenuate degradation.

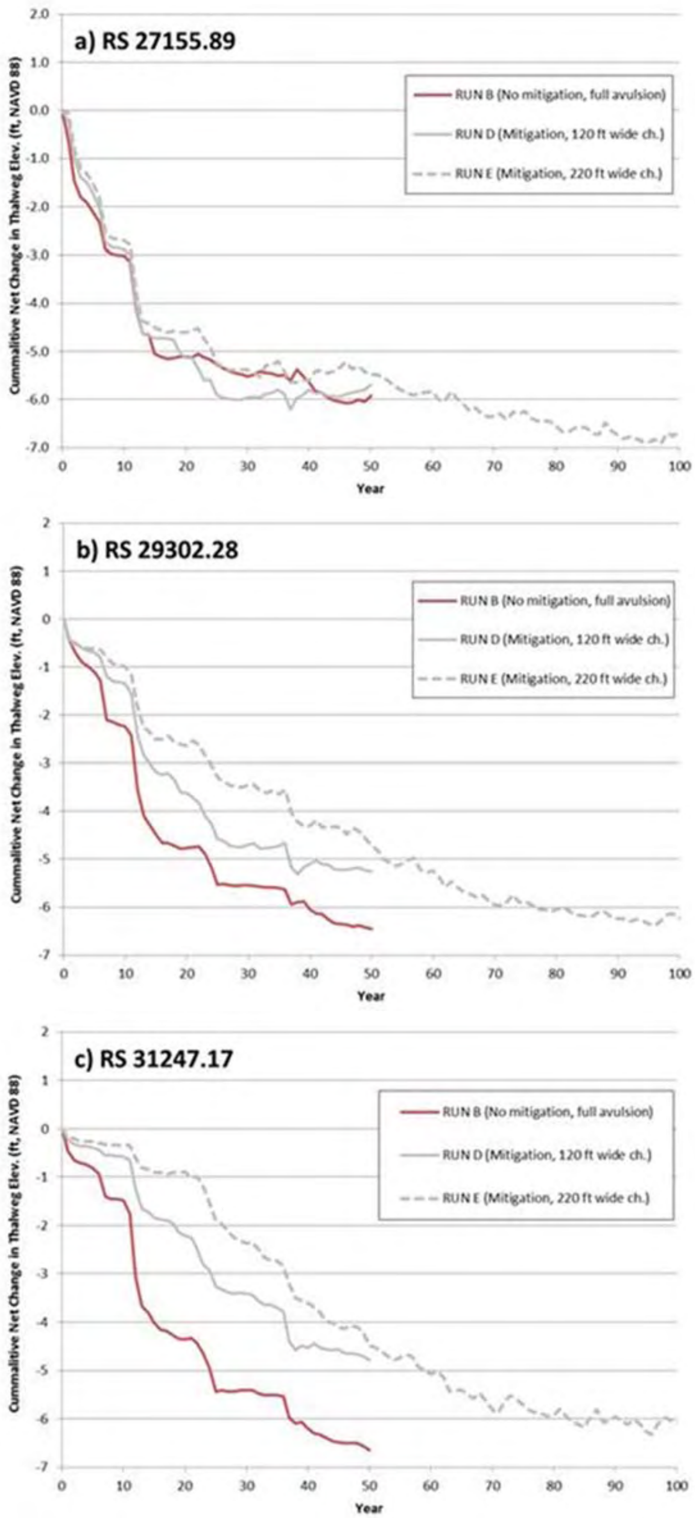

Figure 47: Net change in thalweg elevation at three cross sections near Sportsman's Park Island

Based on these criteria, the 220 foot side channel is more effective at reducing scour at the Federal levee and in attenuating downstream degradation impacts. For preliminary planning and costing purposes, we propose adopting the 220 foot channel (Alternative C2) as the mitigation alternative. During the detailed design stage, additional morphodynamic modeling and geomorphic assessment is required to define and optimize the preferred alternative.

7.5 Summary of Preferred Mitigation Measures

The following section provides a brief summary of the preferred alternative measures introduced above. Table 19 summarizes the mitigation measures and their function. The measures are listed from furthest downstream to furthest upstream and not by their priority or ranking. The location of the measures is shown on Figure 48 and Figure 49.

Table 19: Proposed mitigation measures

ID	Measure	Function
A1	Fill-in Newland Ponds 1 and 2 and part of Pond 3 to thalweg level	Mitigate pit capture, restore channel/floodplain connection
A4	Retain riprap on bank near Pond 3	Reduce risk of lateral shift into Pond 3
A5	Excavate point bar on west bank opposite Pond 2	Reduce risk of lateral shift into Pond 3
B2	Remove old levee and a portion of the existing riprap	Restore natural channel/floodplain
B5	Remove Nob Hill segment of Federal levee	Restore natural morphologic and sediment transport processes
B6	Excavate point bar on west bank upstream of SR24 Bridge	Reduce risk of channel flowing against new levee on east bank
C1	Excavate side channel through island complex	Restore channel conveyance, reduce scour along levee near Buchanan Lake
C3	Construct spurs along Federal levee near Buchanan Lake	Reduce attack on toe of levee, re-direct flow into side channel

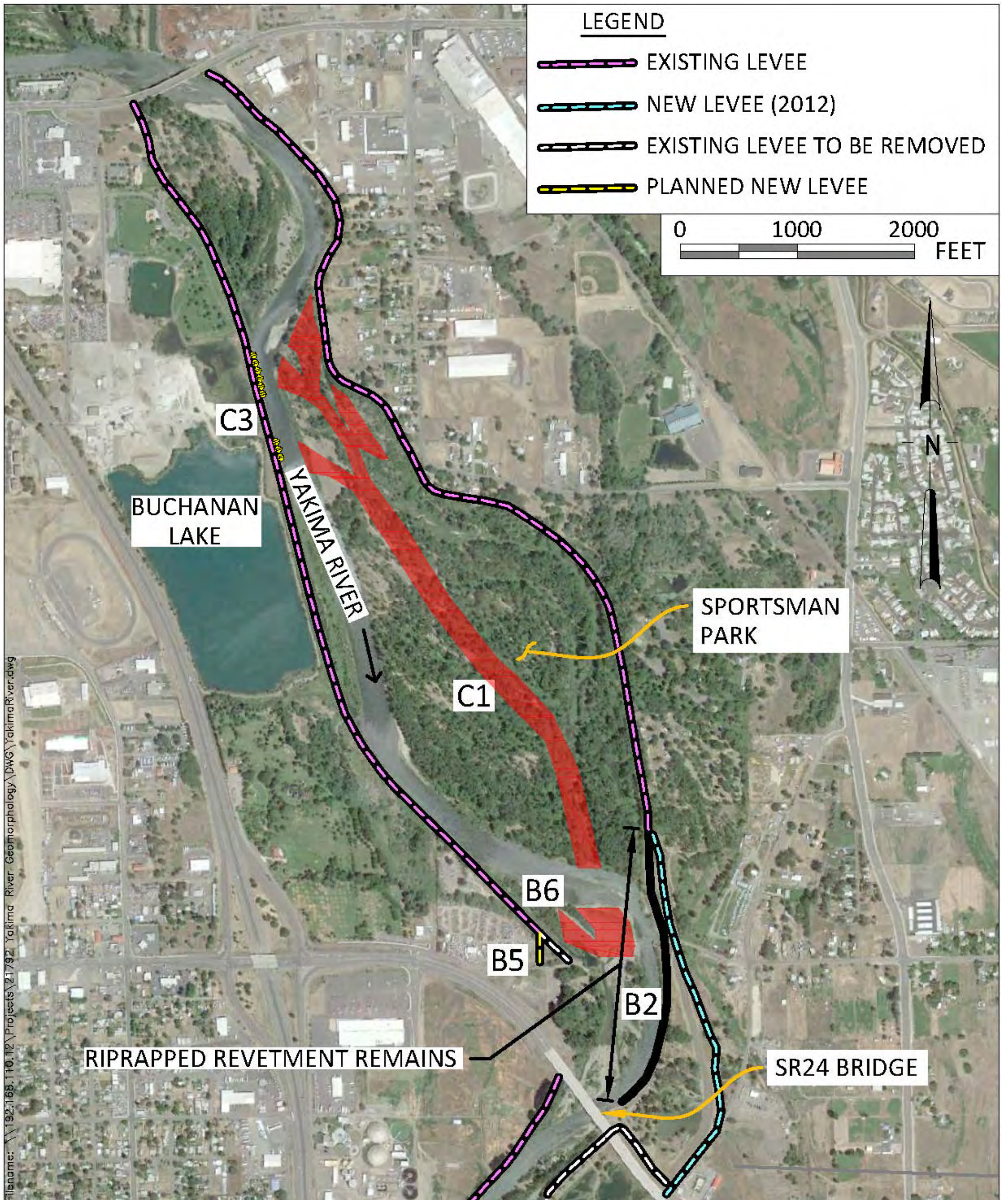

Figure 48: Mitigation measures in northern reach

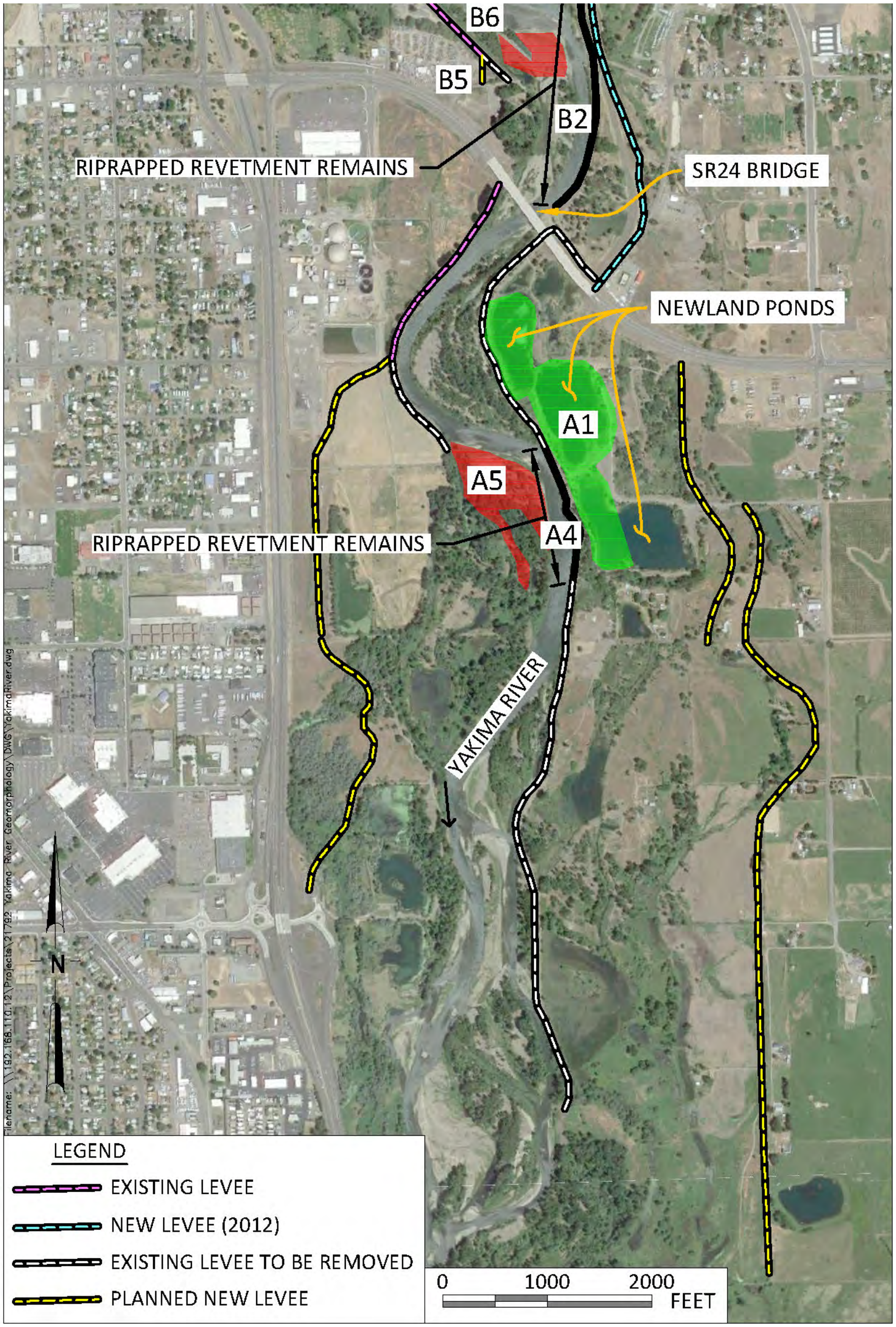

Figure 49: Mitigation measures in southern reach

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

1) A channel avulsion of the Yakima River into the Newland Ponds would generate significant hydraulic and morphological impacts upstream and downstream of the site. Estimates of the maximum upstream degradation range from between 7 and 9 feet. The degradation would extend upstream past Buchanan Lake to the Terrace Heights Bridge. The morphological effects would persist for a period of at least 50 years. This hazard exists under the present DID #1 levee in-place and as well as for future set-back levee alignments.

2) Setting back the DID #1 levee and removing the existing bank protection will increase the risk of an avulsion. Therefore, mitigation measures need to be incorporated into the set-back plans.

3) It is technically feasible to mitigate the risk of an avulsion and to restore the channel to a more natural system, while still improving the overall stability of the reach and lowering flood levels by setting back the levees.

4) The potential effects of an avulsion at the Newland Ponds can be mitigated but require a broad range of measures. These include: infilling a substantial portion of the ponds, installing grade control along the upstream side of the SR24 Bridge, retaining portions of existing bank protection revetments along sections of the banklines, excavating gravel at selected bars to re-direct flow attack, excavating a major new side channel through the island upstream of the SR24 Bridge and installing a series of river training spurs along the Federal levee adjacent to Buchanan Lake.

5) Ongoing monitoring of the reach and an adaptive channel maintenance program will be required during the initial period after the mitigation measures are implemented and the DID #1 levee is set back.

8.2 Recommendations

1) Additional hydraulic investigations should be carried out prior to finalizing the layout and design of the mitigation concepts. Consideration should be given to conducting 2D morphodynamic modeling to develop a detailed layout for re-grading the floodplain at the three ponds and for designing the grade control structure near the SR24 Bridge.

2) The condition of existing riprap scour protection along the Federal levee adjacent to Buchanan Lake should be confirmed prior to finalizing the mitigation plans.

3) The effects of the channel re-alignment and potential future scour levels at the east abutment of the SR24 Bridge should be confirmed prior to finalizing the mitigation plans.

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PHOTOS

Photo 1: Erosion and undermining at old levee on left bank upstream of SR24 Bridge, viewing downstream.

Photo 2: From old levee showing right abutment of SR24 Bridge, viewing downstream.

Photo 3: Near left bank at SR24 Bridge showing cross-dike blocking floodplain.

Photo 4: Existing DID #1 levee downstream of SR24 Bridge, viewing upstream.

Photo 5: From DID #1 levee viewing towards Pond 1, showing SR24 Bridge in background, viewing upstream.

Photo 6: Coarse gravel point bar deposits on left bank, downstream of SR24 Bridge.

Photo 7: Existing DID #1 levee exposed to direct river attack near Ponds 2 and 3, viewing upstream.

Photo 8: Existing DID #1 levee exposed to direct river attack near Ponds 2 and 3, viewing downstream.

APPENDIX A

Data Compiled on Geomorphic Response of Pit Captures

APPENDIX A:
DATA COMPILED ON GEOMORPHIC RESPONSE OF PIT CAPTURES

Numerous examples of floodplain pit capture have been documented in the literature. Selected examples are described in Table A-1 and details of the geometry of each are listed in Table A-2 (which also includes the geometry from geomorphically similar meander cutoffs and inline pit excavations).

Figure A1 shows the locations of pit captures from within Washington and Oregon that are included in Table A-1. The focus of most of these studies has been immediate (within 1 or 2 years) effects of avulsions through floodplain pits. Long term monitoring of channel response to these avulsions as well as an evaluation of floodplain pit intersections that did not cause avulsion have been relatively limited. These examples provide a dataset that will be used in later sections of this memo to develop predictive relations for channel response to floodplain pit capture and to provide a starting place for a description and evaluation of long term geomorphic response in the case studies section.

An overlapping was collected to evaluate condition conducive to avulsions through floodplain pits. Considered locations and geometric data for these sites are included in Table A-3.

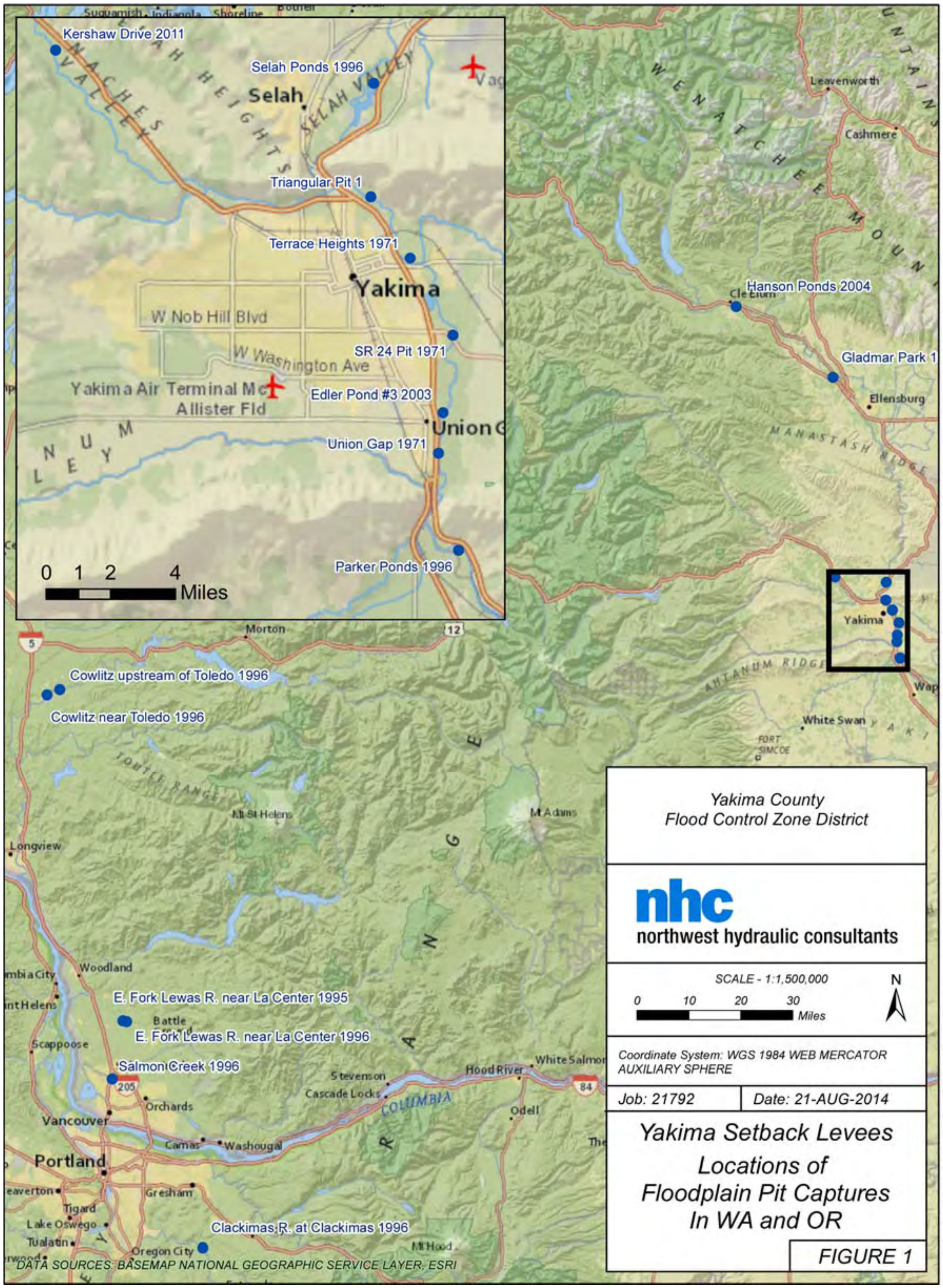

Figure A1: Locations of floodplain pit captures in WA and OR

Table A-1: Published examples of floodplain pit capture.

Location (date)	Description	References
On Yakima River		
Terrace Heights (1971)	Downstream connection between a 22 acre pit and the channel occurred in late 1960s; upstream connection and avulsion in 1971. A delta prograded into the pit over the subsequent two decades forming dozens of small vegetated bars and side channels: by 1981, an 8 acre delta had formed and the delta had completely prograded across the pit by 1996, restoring downstream sediment transport. The estimated* pit volume was 290,000 ± 170,000 yd^3. No upstream or downstream impacts are documented.	(Hilldale and Godaire, 2010; Kelly, 2003; Yakima River Floodplain Mining Impact Study Team, 2004)
Union Gap (1971)	Approximately 1 mile of channel avulsed 2000-3000 feet across the floodplain. The upstream avulsion node occurred at a breach in the levee surrounding a floodplain pit; subsequent flood flows scoured a new channel connecting the upstream pit to another 4,000 feet downstream. Kelly (2003) notes that the captured pits had mostly filled with sediment by 1979; aerial photos indicate that downstream sediment conveyance was restored by 1974. The upstream pit area was 18.7 acres and the downstream pit 15.8. Upstream and downstream pit volumes were 240,000 ± 140,000 and 206,000 ± 122,000 yd^3, respectively. No upstream or downstream impacts are documented.	(Dunne et al., 1980; Hilldale and Godaire, 2010; Kelly, 2003; Norman et al., 1998)
Triangular Pit (Between 1979 and 1992)	Partial avulsion into 8.2 acre pit from an upstream breach. Gravel delta prograded between 1992 and 1996 across upper 2/3 of pit until the downstream part of a bar isolated the remainder of the pit from high velocity flow. The downstream portion of the pit has persisted as a backwater channel with little change since 1996. Estimated* total pit volume is 110,000 ± 64,000 yd^3 and the estimated volume of the gravel deposit is ~71,000 ± 42,000 yd^3.	(Hilldale and Godaire, 2010)
SR 24 pit (~1971)	Partial avulsion through a 4 acre pit with estimated* volume of 52,000 ± 31,000 yd^3. Pit was completely filled by 1974.	(Hilldale and Godaire, 2010)

Location (date)	Description	References
Gladmar Park (1996)	Lateral migration breached the embankment separating a gravel pit from the main channel, and likely would have resulted in a full avulsion if the breach had not been restricted with riprap and a natural large wood jam. A delta-like splay fan deposit formed in the gravel pit below the notch. The volume of the splay deposit in 2011 was approximately 20,000 yd^3, which is substantially more than the 3,000 yd^3 yr^{-1} estimated average annual bedload transport into the reach. Because the elevation of the breach was controlled by riprap and growth of the splay deposit, channel degradation has been minimal upstream of the avulsion. Downstream effects have not been documented.	(Yakima River Floodplain Mining Impact Study Team, 2004; NHC, 2012)
Selah Ponds (Feb 1996)	The Yakima River upstream of the Selah gap breached the embankment that isolated a set of ~15 foot deep gravel pits occupying ~250 acres. The channel avulsed through the ponds and abandoned 1.5 miles of channel. Six to eight feet of degradation occurred in the channel immediately upstream of the breach. A knickpoint, indicated by a standing wave, migrated upstream eroding at least 300,000 yd^3 from the channel bed. This material was deposited in a splay fan across 33 acres of the upper pit. A dike was reconstructed isolating the ponds during the summer after the event, limiting long-term evolution of the avulsion and adjacent channel.	(Norman et al., 1998; Yakima River Floodplain Mining Impact Study Team, 2004)
Parker Ponds (Feb 1996)	A 100 foot wide breach in the embankment separating a set of shallow (~10 ft average maximum depth) floodplain pits opened resulting in a partial avulsion of the channel. Between 1996 and 1998, a splay delta formed in the upstream pond filling it to a depth of approximately 4 feet.	(Norman et al., 1998)
Edler Gravel Pond #3 (2003)	Floodplain (no embankment) separating a 1.9 acre pond from the channel eroded and connected the pond to the margin of the outside of a meander bend. A large recirculating eddy formed in the pond. Sediment conveyance past the pond does not appear to have been blocked. The pond was completely filled with sediment by 2013. No upstream or downstream effects recorded, and no long-term documentation of changes at the site.	(Hilldale and Godaire, 2010; Kelly, 2003; Yakima River Floodplain Mining Impact Study Team, 2004)
Hanson Ponds (2004)	In a controlled reconnection effort, breaches were made at both upstream and downstream parts of embankments separating the ponds from the channel. The upper breech is protected with riprap. No results of long-term monitoring have been located.	(Yakima River Floodplain Mining Impact Study Team, 2004)
Other Examples		

Location (date)	Description	References
Tujunga Wash, CA (1969)	Channel migration on an ephemeral alluvial fan intersected a 50-75 foot deep gravel pit. Scour to depths greater than 14' occurred and extended 3,000 feet upstream. Two-to three million tons of bedload accumulated in the pit and the channel downstream aggraded up to 12 ft and migrated laterally.	(Scott, 1973; Bull and Scott, 1974; Collins and Dunne, 1990; Kondolf, 1994b)
Cowlitz upstream of Toledo, WA (1996)	An avulsion containing ~¼ of the river's flow crossed 3,200 ft of agricultural land before entering a floodplain pit downstream. The avulsion channel scoured and a knickpoint indicated by a migrating standing wave propagated upstream. No scour was noted in the main channel upstream or downstream of the avulsion. The revetment that had failed upstream was replaced in the summer following the event.	(Norman et al., 1998)
Cowlitz near Toledo, WA (1996)	A dike constructed of sand and gravel separating a gravel pit from the main channel was breached and some portion of the flow traveled through the pit. Effects on the main channel were not measured, but appear minimal based on aerial photos. The dike was repaired immediately after the flood.	(Norman et al., 1998)
Salmon Creek, Vancouver, WA (Feb 1996)	Avulsion into a gravel pit of unspecified depth caused a knickpoint to migrate upstream, causing a 4 ft of degradation ¼ mile upstream. The avulsion caused 1,700 feet of channel to be abandoned in favor of a 700 foot long path across the floodplain pit.	(Norman et al., 1998)
Clackamas River at Clackamas, OR (Feb 1996)	The Clackamas River avulsed across a set of floodplain pits. 140,000 yd^3 of material eroded from the bed upstream, causing 6 feet of channel incision about 3,000 feet upstream of the avulsion node. By 2003, the knickpoint had propagated 1.4 miles upstream. Long term channel response has not been documented.	(Kondolf, 1997; Wampler et al., 2007)
East fork Lewis near La Center, WA (1995)	~2,000 feet of a meander bend were cut off when the river avulsed along a 1,700 foot cutoff through a floodplain pit. Upstream and downstream impacts of this event were not documented.	(Norman et al., 1998)
East fork Lewis near La Center, WA (1996)	The river avulsed through six floodplain pits, abandoning ~4,900 feet of channel for a ~4,000 foot long path. This caused about 10 feet of downcutting for an unspecified distance upstream. The combined pit volume was estimated* to be approximately $2X10^6$ yd^3.	(Norman et al., 1998)
Tangipahoa River LA (1980-2004)	Six pit captures occurred between 1980 and 2004 along a 12 mile long reach. Pit depths were typically on the order of 50 feet. Overall, the channel reach degraded approximately 20 feet.	(Mossa and Marks, 2011)

Location (date)	Description	References
Naches R., S. Kershaw Drive (2011)	Lateral channel migration breached the berm separating a ~15 ft deep floodplain pit from the main channel. After the breach, the point bar opposite the eroding bend expanded substantially. Large wood has accumulated along the downstream margin of the pit, potentially blocking a full avulsion through the pit. A small (2-3 foot high) knickpoint has formed and propagated approximately 200 feet upstream of the pit. Avulsion has not yet occurred. The preferred avulsion path through the pond is approximately 1,600 ft with 400 ft of pond length, while the length of the channel that would be abandoned is approximately 1,900 ft.	Original observations.

Note: All descriptions based on information in citations and aerial photos reviewed as a part of this study.
*Reported pond volumes estimated from pond area based on the average pond depth of 8.5 ± 4.9 ft reported by the Yakima River Floodplain Mining Impact Study Team (2004).

Table A-2: Geometric Attributes of Documented Floodplain Pit Avulsions, Inline Excavations, and Meander Cutoffs.

Site	Avulsion Path Length (ft)	Avulsion Path-Pond intersect (ft)	Abandoned Channel Length (ft)	Pit depth (ft)	Headcut height at avulsion node (z, ft)	Headcut propagation distance (x, ft)	Pre-avulsion slope (S_i, ft/ft)
Floodplain Pit Avulsions							
Union Gap[1]	8,600	1,300	11,200	unk.	unk.	unk.	0.002
Gladmar[1]	3,400	1,100	3,500	10	0	0	0.004
Selah[1]	7,000	2,300*	8,000	25	8	3,000	0.002
Parker[1]	1,400	710	1,700	10	small	unk.	0.003
Edler[1]	350	350	350	20	small	not detectable	0.003
Tjunga Wash, CA[1]	3,200	700	3,200	75	14	3,000	0.014
Salmon Ck. [1]	700	400	1,700	unk.	> 4	> 1,300	0.006
Clackimas R., OR[1]	4,900	3,000	9,000	25	10	7,400	0.002
Lewis R. ('95) [1]	1,700	700	2,000	30	unk.	unk.	0.003
Lewis R. ('96) [1]	4,000	3,500	4,700	30	10	unk.	0.003
Inline Excavations							
Cache Creek, CA[2]	unk.	0	unk.	13	10	4,260	0.002
Cedar R. [3]	5,000	5,000	5,000	6	3	2,200	0.0017
San Juan Creek, CA[4]	4,000	4,000	4,000	55	30	3,500	0.012
Meander Cutoffs							
Puyallup River[5,6]	48,000	0	64,000	0	12	21,000	0.001
Fossil Creek[7]	250	0	1,000	0	15	500	0.02
Lillooet River[8]	64,000	0	81,000	0	13	unk.	0.0008
Mississippi hill streams[4]	15,000	0	24,000	0	4	69,000	0.0005
Dam Removals							
Marmot Dam[9, 10]	0	0	0	0	49.2	6,560	0.003

* Length of the upstream pond. Total pond length was 5,500 ft.
Sources: 1: as reported in Table A-1, 2:NHC (1995), 3: NHC (2014a), 4: Galay (1983), 5: NHC (2014b), 6: Czuba et al. (2011),7: NHC (2005), 8 Weatherly and Jakob (2014), 9: Major et al. (2012), 10: Cui et al. (2014).

Table A-3: Slope Ratios for floodplain-pit intersections that did not result in avulsions and that did result in avulsions.

Location	Slope Ratio at Avulsion Node*	Source
Connections with no avulsion		
Parker	2.5	1
Edler	1.3	1
Just us of Ellensburg KOA	1.7	2
River Bottom Rd. Ellensburg	1.2	2
Tjossem Rd. Ellensburg	1.5	2
Yakima RB 4 km ds of Rotary Lake	1.3	2
Yakima RB 1.6 km N of Edler Ponds	1.5	2
Union Gap LB	1.3	2
Union Gap 3000 ft us of diversion dam	2.0	2
Pond 5	1.0	2
Wyanochee Pond 3 mi N. of Wyanochee Wiska Rd. Bridge	1.8	3
Humtulips Ponds	1.4	3
Connections with avulsion		
Union Gap	1.5	1
Gladmar	1.5	1
Selah	1.7	1
Tjunga Wash	1.3	1
Salmon Creek	5.7	1
Clackimas	4.7	1
Lewis (95)	2.0	1
Lewis (96)	9.4	1
Terrace heights	1.9	2
US Toledo	1.8	3
Toledo	2.3	3

* The slope ratio is adjusted for ponds by subtracting pond length from avulsion path length.
All data from citations supplemented by aerial photos reviewed as a part of this study.
Data Sources 1: Table A-2, 2: Floodplain Mining Impact Study Team (2004), 3: Norman et al. (1998).

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APPENDIX B

Hydraulic Modeling of Mitigation Concepts

1 HYDRAULIC MODELING OF MITIGATION CONCEPTS

Northwest Hydraulic Consultants Inc. (NHC) conducted preliminary two-dimensional (2D) hydraulic modeling to assess the DID #1 levee set-back and measures to mitigate for the potential channel avulsion into the Newland Ponds. Mitigation measures evaluated were conceptual-level and included partial pond filling, overbank channel construction, and floodplain re-grading. Hydraulics in the vicinity of the SR24 Bridge and the Newland Ponds, where flow will expand into the left overbank following set back of the DID #1 levee, are complex, thus require 2D hydrodynamic modeling. The 2D model was used to evaluate spatial variation in water surface elevation, flow depths, and velocity flow fields in the vicinity of the SR24 Bridge and Newland Ponds. Although only fixed bed conditions were considered, the results were used to identify areas of likely geomorphic instability, as well as evaluate the effect of possible mitigation measures on floodplain hydraulics.

Two additional one-dimensional morphodynamic model runs were also made to assess the effectiveness of potential mitigation measures implemented upstream of the SR24 Bridge, on the Sportsman's Park island complex. These measures include re-opening and excavating side channels through the island to arrest potential upstream degradation following the DID #1 levee set-back and capture of the Newland Ponds.

1.1 2D Model Description

The 2D model was developed using the U.S. Bureau of Reclamation's (USBR) SRH-2D software package. This model solves the depth-averaged St. Venant (2D dynamic wave) equations over a computational mesh representation of the topographic surface (Lai, 2008). The mesh was generated using the Surface-water Modeling System (SMS) software developed by Aquaveo. The topographic surface was constructed using LiDAR data collected by the U.S. Army Corps of Engineers (Corps) in 2013, as well as channel and pond bathymetry data collected by NHC in the summer of 2014. The 2D model domain and existing condition topography is shown in Figure 1.

Surface roughness was defined by land cover as identified on recent aerial photographs. Seven land cover classes were established and delineated in ArcGIS and roughness coefficients (Manning's 'n') were assigned to each class. Coefficients were initially based on values used in one-dimensional models developed for the reach (e.g. SRH-1D and HEC-RAS) provided by the County. The values were reduced by approximately 25% because the form drag component of Manning's 'n' values are implicitly considered in 2D models. A surface roughness coverage was created in ArcGIS and used as 2D model input.

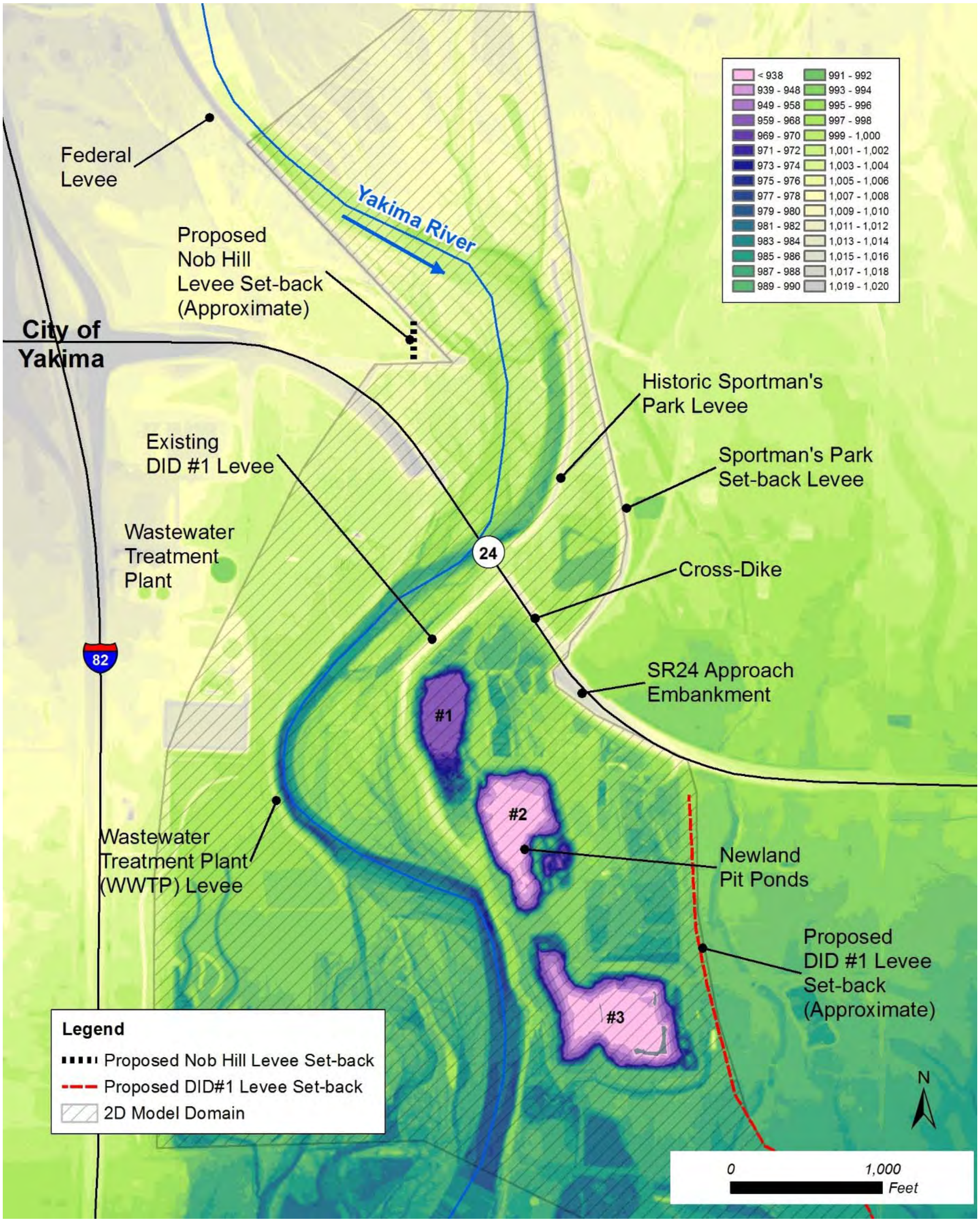

Figure 1: Model domain for the SRH-2D model and existing condition topography

The 2D model was run steady-state with a fixed bed for the 10- and 100-year instantaneous peak discharges of 26,600 and 57,300 cfs, respectively. Test runs were also conducted using the 2-year instantaneous peak discharge (16,150 cfs) but results showed limited overbank flow into the Newland Pond area. The 10- and 100-year discharges were input to the model at the upstream boundary. The downstream boundary condition was based on estimated starting water surface elevation computed by a previously developed one-dimensional (HEC-RAS) model. The 2D model was run for 48 hours or until hydraulic computations converged to a steady-state. The topographic surface constructed with available LiDAR and bathymetry data represents existing conditions, thus modifications were required to model set-back levee conditions, as well as the mitigation options. These modifications were conducted using tools available in ArcGIS.

1.2 Modeled Scenarios

A total of four scenarios were evaluated. The first (Scenario 1) consisted of simply removing existing levees flanking the site and setting back the DID #1 levee. The remaining three scenarios included conceptual mitigation options initially developed by NHC and County staff. Refinement to the mitigation options were then made based on subsequent 2D model results. Figure 2 compares the modified topographic surfaces for each scenario, while Figure 3 compares the relative changes in ground topography between the scenarios. The following sections describe how the surfaces were modified and specific mitigation features.

1.2.1 Scenario 1 – Levee Set-back

Scenario 1 included removal of the DID #1 levee downstream of SR24 Bridge, as well as the cross-dike and historic Sportsman's Park levee upstream of the SR24 Bridge. In addition, Scenario 1 included the proposed DID #1 set-back levee downstream of SR24 Bridge. The set-back alignment was constructed using an approximate alignment provided by the County and estimated elevations capable of containing the 100-year flow. Scenario 1 functioned as the baseline condition to evaluate overbank flow patterns in the vicinity of the SR24 Bridge and the Newland Ponds. It also served as the base geometry for the following three scenarios.

1.2.2 Scenario 2 – Pond 1 Fill and Downstream Swale

Scenario 2 included filling Newland Pond 1, excavating a swale feature connecting Pond 1 to the main channel downstream, and removing the downstream-most 700 foot segment of the wastewater treatment plant (WWTP) levee. The ground near the cross-dike was lowered to approximately elevation 997 feet, NAVD 88, which is close to the adjacent floodplain level immediately upstream. Filling of Pond 1 and excavation of the downstream swale was intended to direct flow back toward the river and away from Ponds 2 and 3 downstream. Pond 1 was filled to an elevation of approximately 991 feet, NAVD 88, or about 5 feet higher than the surveyed water level. The small ponds immediately north of Pond 1 were not filled. The swale feature was approximately 600 feet wide by 700 feet long and would be formed by excavating 6 to 7 feet off the downstream quarter of the existing gravel bar. The swale is intended to direct flow back toward the river and away from Ponds 2 and 3 downstream. Removal of the lowermost segment of the WWTP levee consisted of lowering the structure 4 to 5 feet. This was intended to evaluate the effect of allowing more flow into the right floodplain away from the main channel and Ponds 2 and 3.

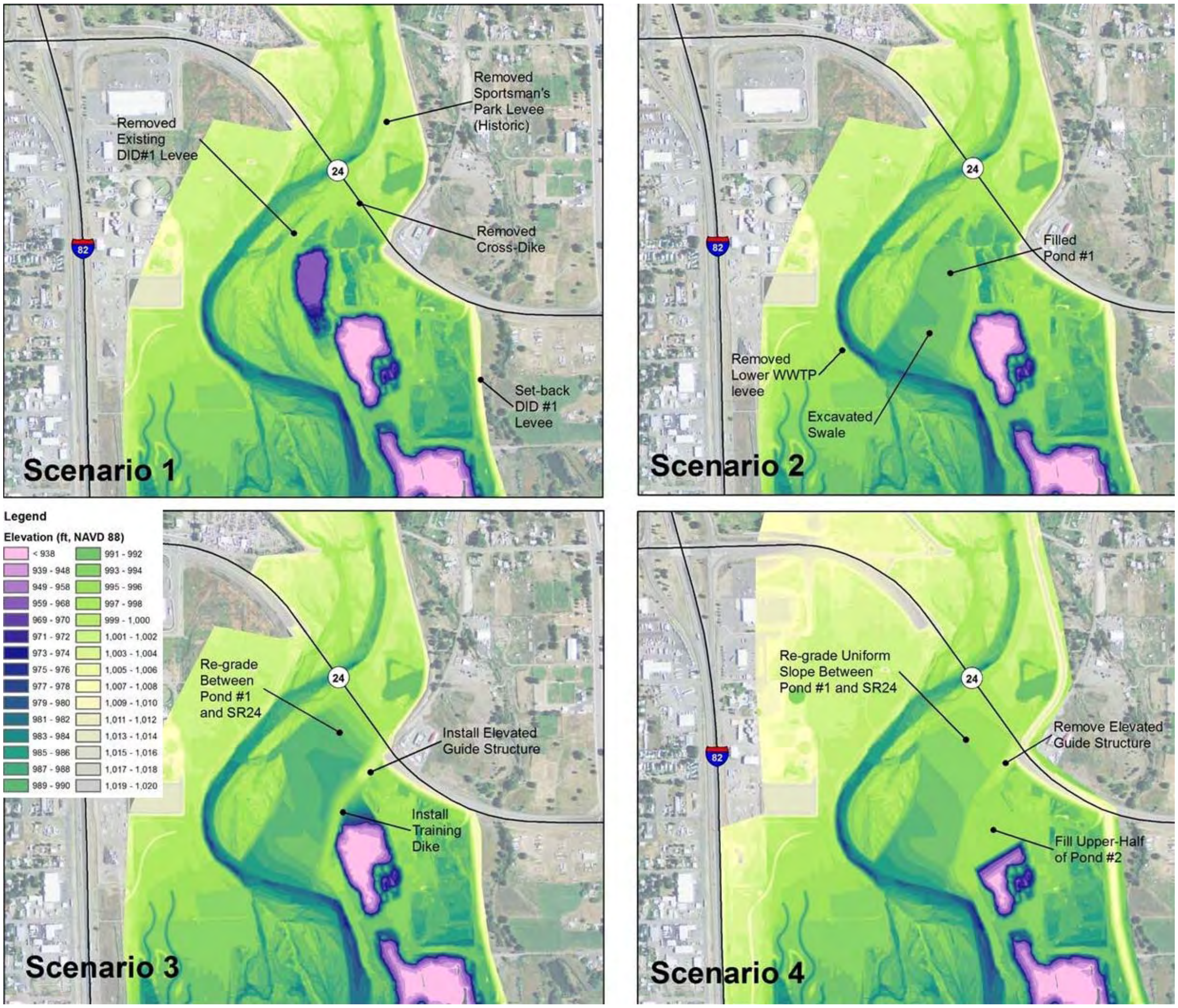

Figure 2: Two dimensional model surfaces

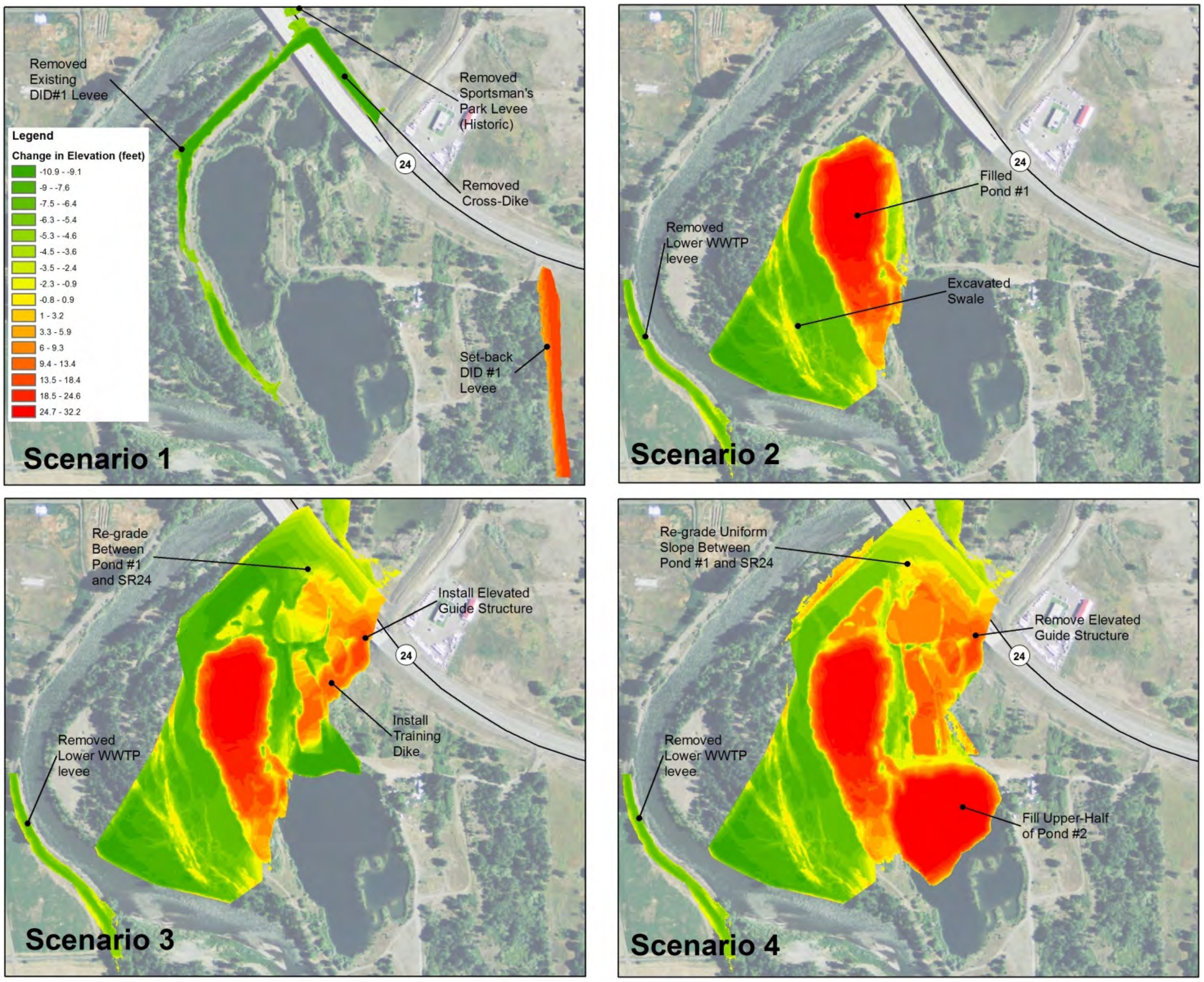

Figure 3: Relative elevation changes to floodplain topography

1.2.3 Scenario 3 – Left Bank Guide Structure and Re-grading Under SR24 Bridge

Scenario 3 included the addition of a raised guide structure extending downstream of the left SR24 Bridge abutment and re-grading the left overbank in the vicinity of the removed cross-dike just upstream of the SR24 Bridge. The ground near the existing cross-dike was lowered approximately 3 feet below the level in Scenario 2. The guide structure extended 275 feet downstream of the existing left abutment and was oriented perpendicular to the bridge and parallel to the river. The elevation, set at approximately 1000 feet, NAVD 88, is capable of containing the 100-year discharge. It was assumed a low-lying training dike would extend downstream of the guide structure an additional 1,000 feet to isolate flow into Pond 2. Re-grading below the SR24 Bridge was intended to further reduce concentrated acceleration over the 8 to 10 foot grade drop. Even with re-grading, the left overbank slope remained relatively steep, at about 3%.

1.2.4 Scenario 4 – Uniform Grading of Swale

Scenario 4 included removal of the SR24 Bridge guide structure (Scenario 3) and additional re-grading under the SR24 Bridge and the Pond 1 area to create a more uniformly sloped swale. The ground near the existing cross-dike was lowered approximately 3 feet below the level in Scenario 2. The resulting re-graded slope was approximately 0.6% and it was intended to reduce flow acceleration just downstream of the SR24 Bridge. In addition, the upstream half of Pond 2 was filled to reduce local velocities and the likelihood of pond capture.

1.3 Simulation Results

Results from the SRH-2D model simulations, including velocity fields, water surface elevations, and flow depths computed for the 10- and 100-year discharges are presented at the end of this appendix. Key observations from the 2D simulations are summarized below.

The majority of overbank flow entering the floodplain and Newland Ponds is concentrated on the left bank just downstream of the SR24 Bridge. Here, acceleration occurs over the existing grade drop between the removed cross-dike and the small ponds to the north of Pond 1. Peak velocities computed in this vicinity ranged from 10 to 12 fps for the 10-year discharge, and greater than 14 fps for the 100-year discharge in Scenarios 1 to 3. Filling of Pond 1 (Scenario 2) did not eliminate the acceleration immediately upstream but reduced the magnitude of the floodplain spill. Re-grading the floodplain topography in Scenario 3 showed no benefit over Scenario 2. Re-grading to a more uniform slope of 0.6% (Scenario 4) shows some improvement with peak velocities reduced to 8 to 10 fps for the 10-year discharge, and 10 to 14 fps for the 100-year discharge. Figure 4 illustrates the velocity reduction by comparing Scenarios 1 and 4 at the 100-year discharge. Regardless of minor reductions in velocity, significant erosion could occur upstream of the Newland Ponds under these scenarios if measures are not implemented to supress degradation.

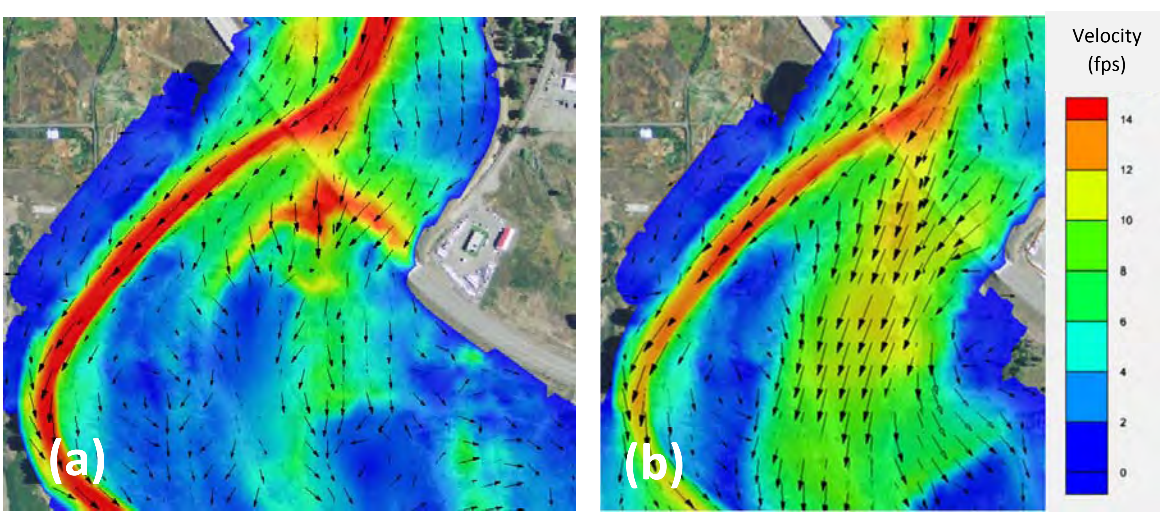

Figure 4: Comparison of computed 10-year velocity field for (a) Scenario 1 and (b) Scenario 4

Water surface elevations illustrate how the flow transition downstream of the SR24 Bridge varies between modeled scenarios. Scenarios 1 to 3 show a similar pattern of super-elevated water surface levels in the main channel relative to levels in the Newland Pond area on the left overbank. The difference in water surface elevations ranges from 2 to 6 feet for both the 10- and 100-year discharges. With Scenario 4, water surface elevation becomes more uniform and evenly distributed across the channel and overbank as indicated by the increased space between contours. Figure 5 compares 100-year computed water surface elevations and contours for Scenario 1 and 4.

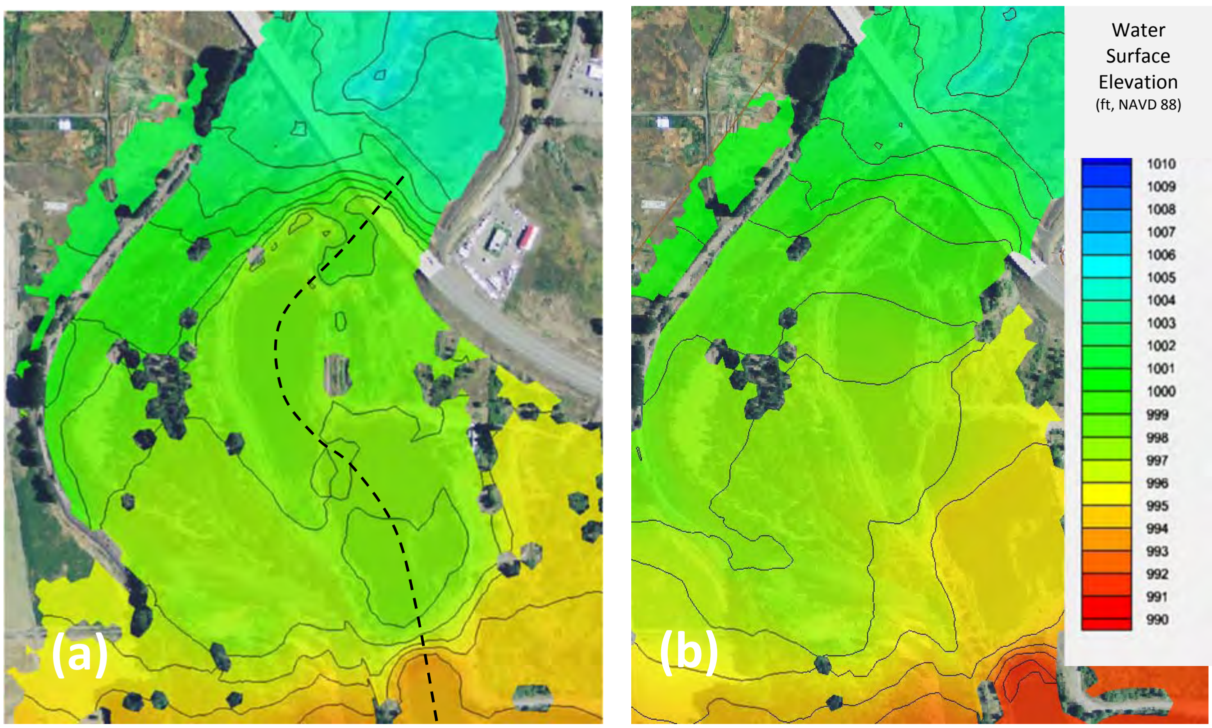

Figure 5: Comparison of computed 100-year water surface elevation for (a) Scenario 1 and (b) Scenario 4

Profiles of computed 10- and 100-year water surface profiles, extracted from the 2D results of levee set-back only (Scenario 1) condition, show locations of abrupt head loss and backwater control through the Newland Ponds (Figure 6). The profile alignment is shown in Figure 5a. A 4 to 5 foot head drop occurred immediately downstream of the SR24 Bridge as overbank flow spills into the small ponds to the north of Pond 1. Downstream, Ponds 1 and 2 effectively act as a single waterbody with water surface elevations controlled by the thin strip of land dividing Ponds 2 and 3. The computed head drop between Ponds 2 and 3 was 3 to 6 feet; thus, breaching and erosion of this feature would result in a cumulative head drop of 7 to 11 feet though the combined Newland Ponds.

Table 1 summarizes the proportion of flow remaining within the main channel to that overtopping into the left overbank near the Newland Ponds. In Scenario 1 (levee set-back only), 16% of the 10-year discharge and 48% of the 100-year discharge spilled overbank. Filling of Pond 1 (Scenario 2) resulted in a reduction in the flow spilling overbank in comparison to Scenario 1. The reduction in spill was 3% (from 16% to 13%) at the 10-year flood and up to 14% (48% to 34%) at the 100-year flood. Scenarios 3 and 4 involved additional re-grading of the floodplain upstream and downstream of the ponds. It appears that lowering the floodplain along the upstream side of the SR24 Bridge had a significant impact on the spill magnitude. For example, the overbank spill was significantly higher than in Scenario 2 and was virtually the same as in Scenario 1. Thus a small lowering of the entrance to the spill negated the effect of infilling Pond 1. These results illustrate that the magnitude of the spill is very sensitive to the floodplain elevation.

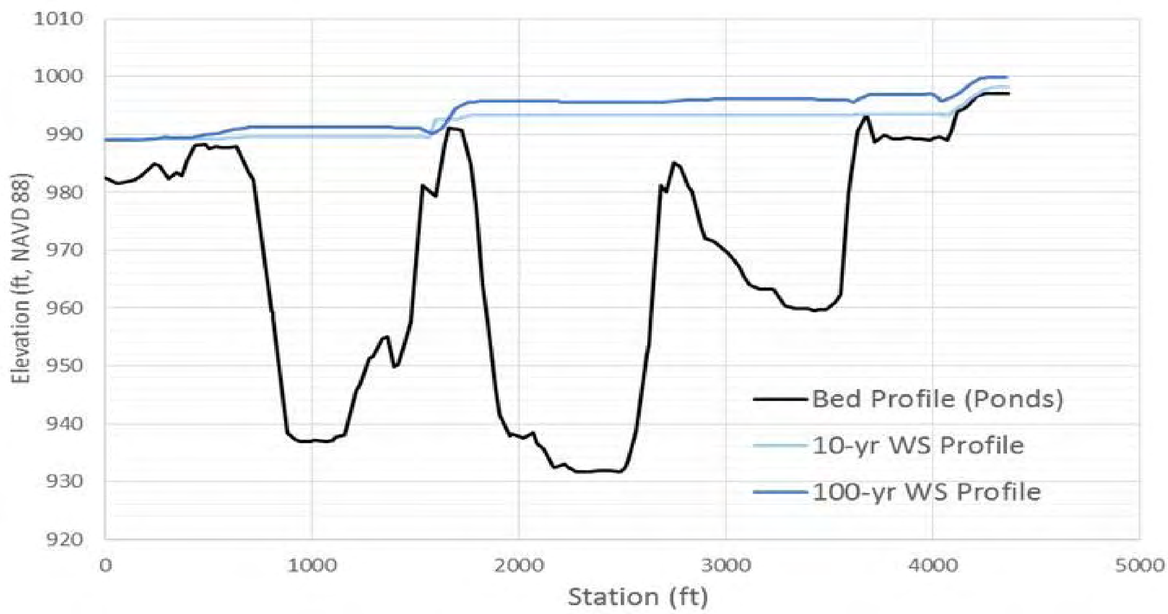

Figure 6: Computed bed water surface profiles through the Newland Ponds extracted from 2D model results (Scenario 1)

Table 1: Computed flow distributions in channel and left overbank

Scenario	Main Channel Flow (cfs)	Left Overbank Flow (cfs)	Main Channel Flow (%)	Left Overbank Flow (%)
10-year Discharge				
Scenario 1	23,237	4,344	84%	16%
Scenario 2	22,743	3,496	87%	13%
Scenario 3	20,694	6,039	77%	23%
Scenario 4	21,259	4,414	83%	17%
100-year Discharge				
Scenario 1	28,605	25,956	52%	48%
Scenario 2	37,080	19,282	66%	34%
Scenario 3	29,806	26,804	53%	47%
Scenario 4	28,711	28,711	50%	50%

1.4 Assessment of 2D Model Results

The hydraulic model confirmed that following the proposed DID #1 levee set-back and removal of the historic Sportsman's Park and cross-dike levee structures, the left overbank between SR24 Bridge and Pond 1 will be subjected to significant hydraulic forces during floods. The model results show that velocities at both the 10-year and 100-year discharges will be focused at the upstream end of the Newland Ponds and competent to erode the native floodplain soils. These conditions represent a 'snap shot' of the initiation of an avulsion of the Yakima River through the Newland Ponds. Therefore, further investigation of possible mitigation measures is suggested to prevent an uncontrolled pit capture at the three ponds.

Comparison of Scenario 1 (set-back levee) and Scenario 2 (set-back levee and filling Pond 1) showed that infilling the ponds can reduce the magnitude of the floodplain spill. Scenario 3 and Scenario 4 showed that the benefits of filling in the pond can be greatly reduced if the ground level near the entrance to the spill is lowered below the general floodplain grade.

Other measures such as the guide bank on the left bank at SR24 Bridge and removal of the lower portion of the WWTP levee showed limited effect. Removal of the WWTP levee results in a 2-6% increase in overbank flow to the right floodplain at the 100-year discharge.

Proposed set-back of the Nob Hill segment of the Federal levee (Figure 1), was not explicitly evaluated in this analysis because it is unlikely to influence conditions downstream of the SR24 Bridge and the Newland Ponds. The existing alignment of the river puts this levee segment on the inside of a meander bend where a large gravel bar has formed. Review of 2D hydraulic patterns in this vicinity indicates that there is currently limited flow on the right floodplain, adjacent to the levee, as the majority is conveyed through the main channel. Fixed bed, 2D hydrodynamic analysis would show little change following levee set-back under these conditions. Furthermore, even with set-back of the Nob Hill levee, it is still located in the upstream shadow of the west SR24 Bridge approach embankment, thus resides within a hydraulically ineffective area. As such, the primary benefit of the Nob Hill set-back will not be hydraulic, but rather, geomorphic. Removal of this levee segment will reduce the existing geomorphic constriction, allow for more lateral channel migration, and help restore natural channel processes, particularly if measures are implemented upstream, as discussed below.

1.5 Side Channel Restoration at Sportsman's Park Island

1.5.1 Description of Concept

The objective of this mitigation measure was three-fold:

1) Restore the river from its present channelized state into an anabranched system.
2) Reduce the potential rate and upstream extent of degradation resulting from a pit capture.
3) Reduce the potential deep scour that occurs along the Federal levee due to past channelization.

This concept involves re-opening and excavating a 5,400 foot long side channel through the island complex that has formed upstream of the SR24 Bridge (Figure 7). The river is confined in a straight, single channel in this reach and flows against the Federal levee on the west bank. The island is a formerly active bar complex that has stabilized and become forested over the past few decades (since 1971 based on aerial photos). The stabilization of the island is partly due to the river becoming entrenched next to the Federal levee adjacent to Buchanan Lake as well as accretion on the island itself.

The mitigation measures consist of excavating the upstream end of the island to create a new entrance, opening up existing channel sections through the island that have filled-in, opening the entrances to distributary channels that have been blocked and constructing new sections of channel through portions of the island. The tentative dimensions of the side channel were based on the bankfull hydraulic geometry relations for a branched channel (USACE, 1994; ASCE, 2008). Based on this information the tentative top width of the channel was varied from 120 feet to 220 feet.

1.5.2 Model Representation

Side channel restoration was evaluated using a Sedimentation and River Hydraulics-One Dimension (SRH-1D) model originally developed by Hilldale and Godaire (2010) and later modified by NHC. The two SRH-1D simulations (Run D and Run E) consisted of adding a simplified, trapezoidal channel to the left bank between River Station (RS) 27155 and 32154 to a previously developed run assuming no mitigation is implemented and full avulsion of the channel into the Newland Ponds occurs (Run B). In Run D, the channel was assumed to have a top width of 120 feet and a depth of 6 feet. In Run E, the channel width was increased to 220 feet with the depth remaining at 6 feet. In a one dimensional analysis, the lateral position of the side channel does not affect the hydraulic results. For simplicity, the trapezoidal channel sections were inserted immediately adjacent to the existing channel (Figure 8). Figure 7 illustrates the location and extent of the excavated side channel as positioned in the SRH-1D model. Two-dimensional morphodynamic modeling should be carried out prior to finalizing the design of the side channel.

The top of the channel was set to the computed 2-year water surface elevation, which corresponds approximately to bankfull conditions. Bank stations were shifted to include the added channel to ensure aggradation and degradation would be computed by SRH-1D. Runs D and E simulated 50-year and 100-year periods, respectively.

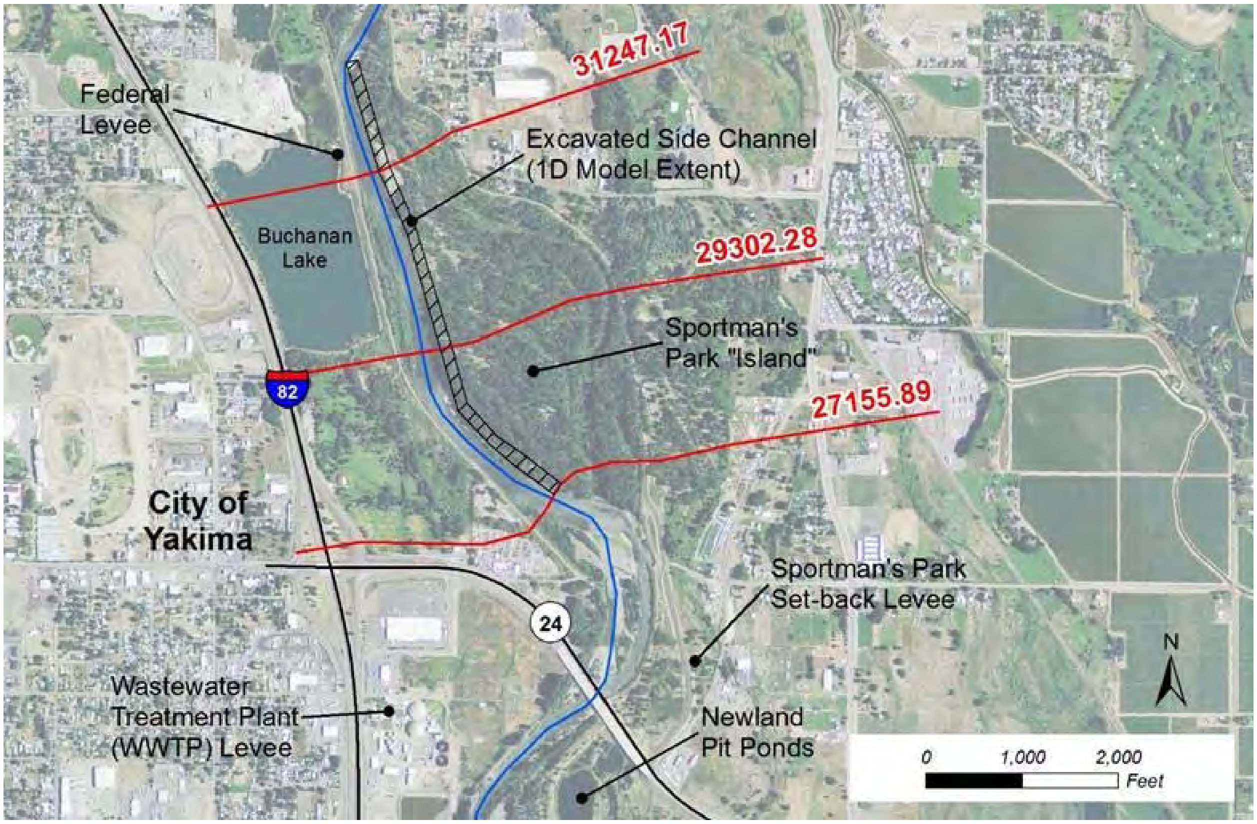

Figure 7: Location of simulated side channel at Sportsman's Park Island

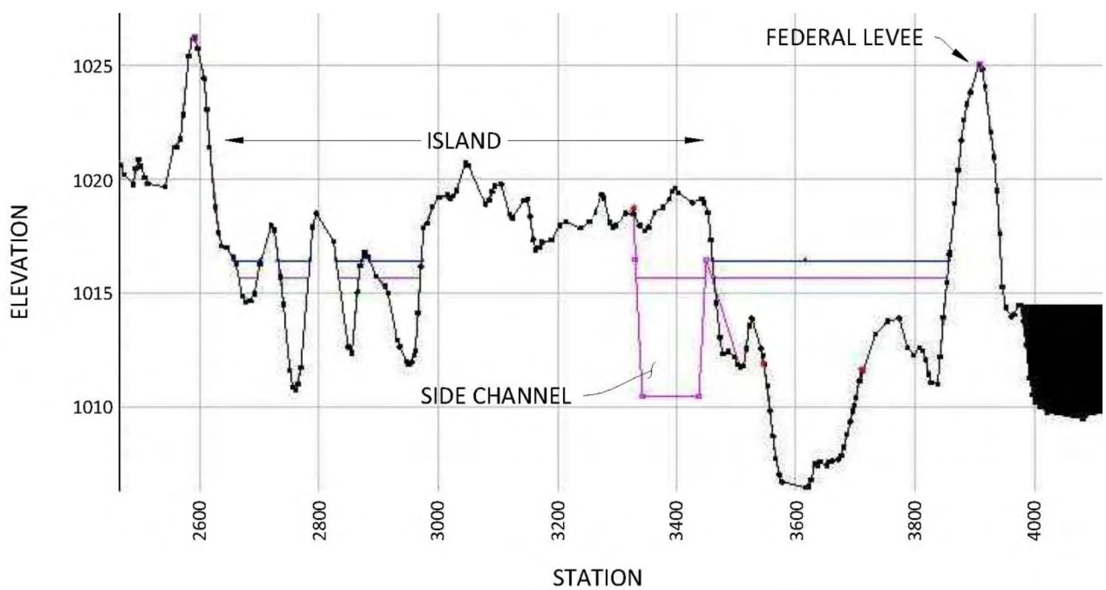

Figure 8: Example of side channel cross section modification

1.5.3 Results of Run D and Run E

Figure 9a to c compare changes in the thalweg elevation over time at three locations adjacent to the Sportsman's Park island for the full avulsion case without mitigation (Run B) and with excavated 120 foot (Run D) and 220 foot (Run E) wide side channels added. The three locations are shown in Figure 7 and correspond to the lower extent of the side channel excavation and the downstream and upstream ends of Buchanan Lake.

Figure 9 illustrates that the excavated side channel slows the upstream progression of the headcut initiated by the pit capture at the Newland Ponds but does not completely eliminate it. The mitigation effect is smallest at the downstream end and increases in the upstream direction. Degradation at the lower end of the side channel is essentially unaffected by the excavations. Further upstream, at RS 29302.3, degradation was predicted to reach 5.5 feet after 25 years in Run B (full avulsion, no mitigation), 4.7 feet in Run D (120 foot side channel) and 3.5 feet in Run E (220 foot side channel). At RS 31247 near the upstream end of the channel, degradation was estimated to reach 5.5 feet in Run B (full avulsion, no mitigation), 3.2 feet in Run D (120 foot side channel) and 2.0 feet in Run E (220 foot side channel. Even with the side channel in-place, net degradation is still in the order of 4 to 5 feet after 50 years and degradation would persist for at least a century. As such, the side channel significantly slows down the rate of the upstream migration of the headcut, but does not suppress it. The other important hydraulic benefit of the side channel is to reduce the deep scour along the toe of the Federal levee near Buchanan Lake.

The 220 foot excavated channel was substantially more effective than the 120 foot channel in attenuating degradation caused by a pit capture. Furthermore, empirical hydraulic geometry equations indicate the dominant or channel-forming discharge for a 220 foot channel ranges between 6,600 cfs and 11,000 cfs, or approximately 41% to 68% of a 2-year flood. This suggests that with the 220 foot side channel the river would be able to maintain two stable approximately equal branch channels, rather than flow in the single channel that presently exists.

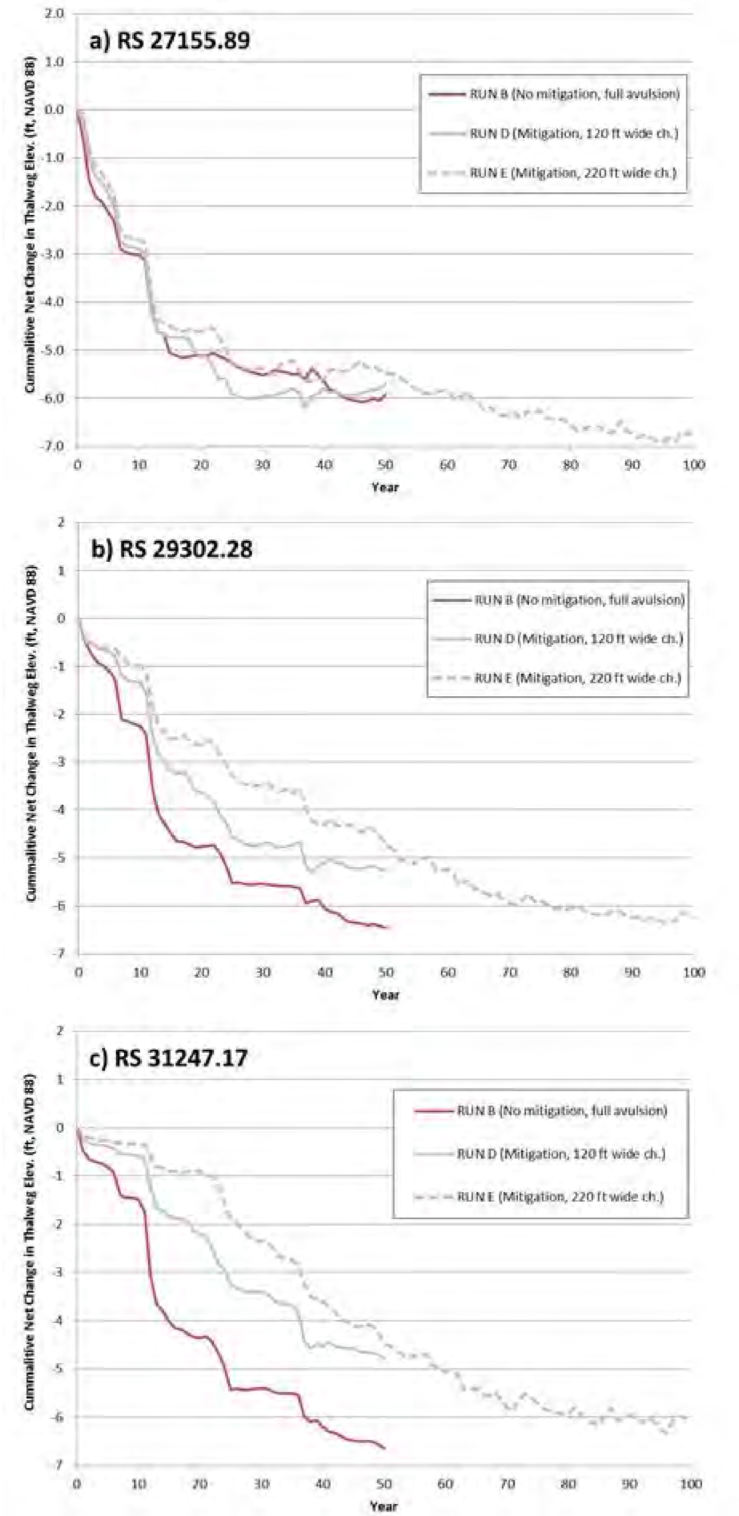

Figure 9: Net change in thalweg elevation at three cross sections near Sportsman's Park Island

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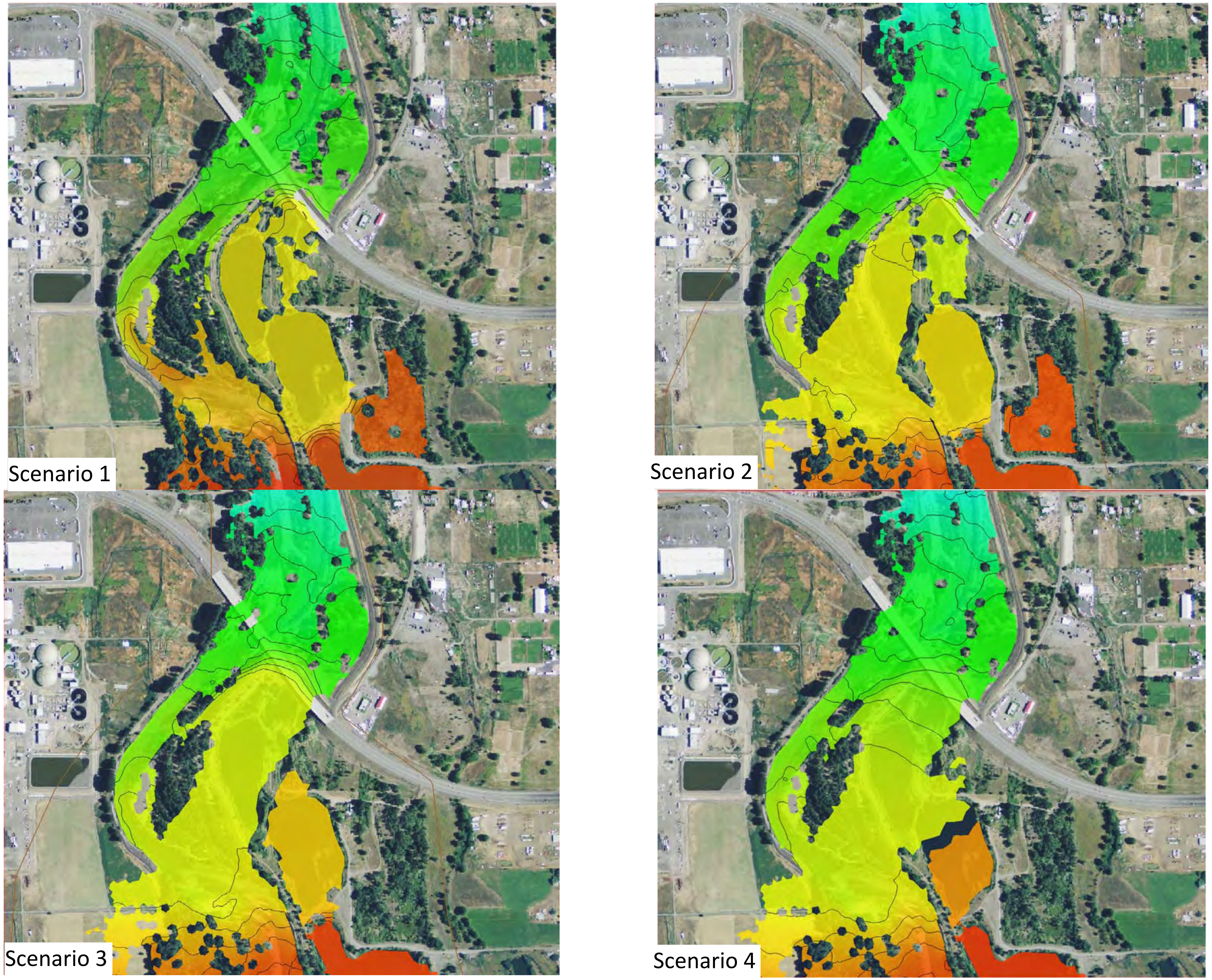
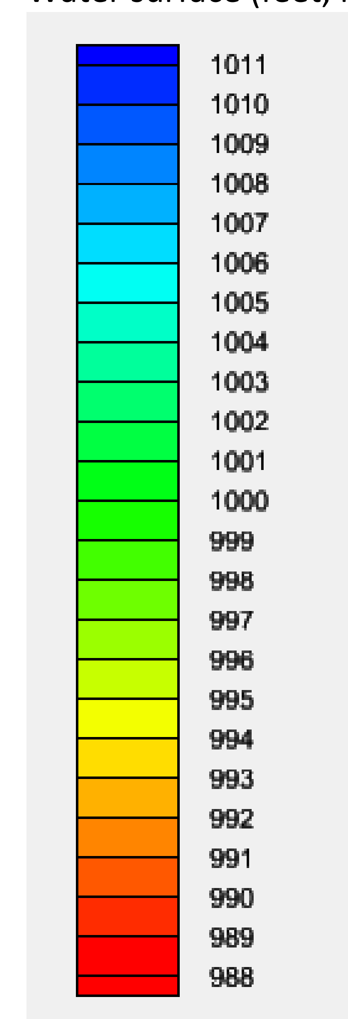
Computed 10-year
Water Surface Elevation



Legend
Water surface (feet, NAVD 88)



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April 15, 2015

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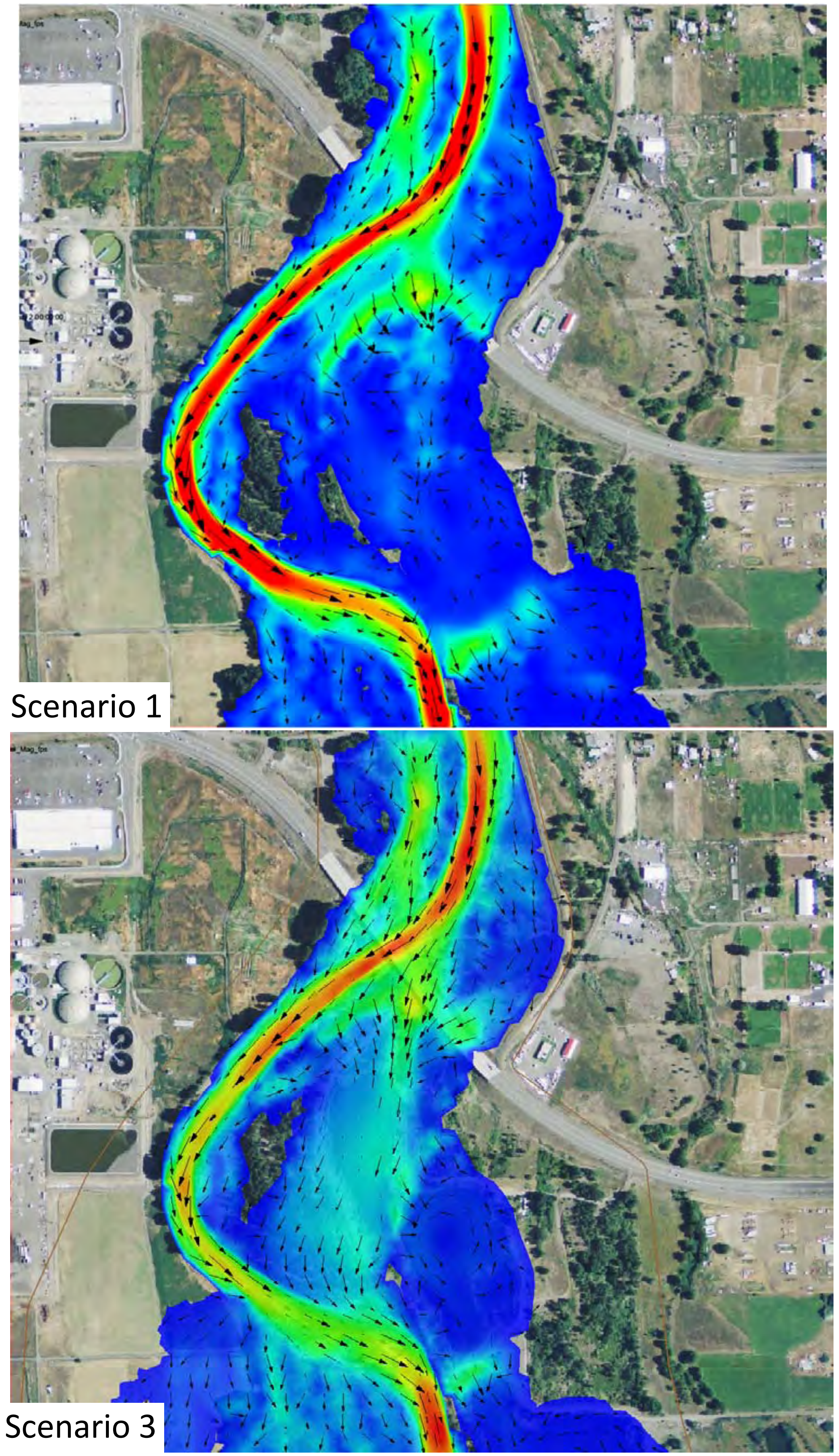
Computed 10-year Flow Velocity



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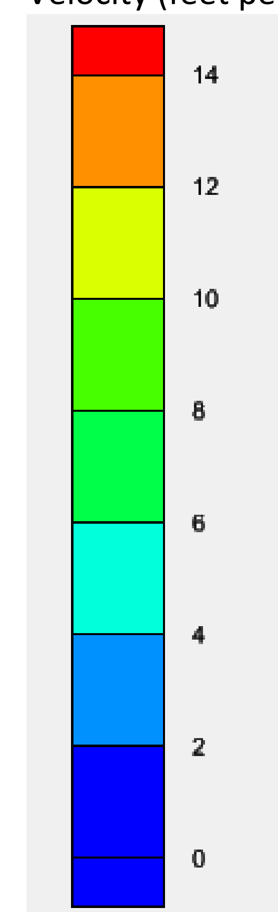
Velocity (feet per second)



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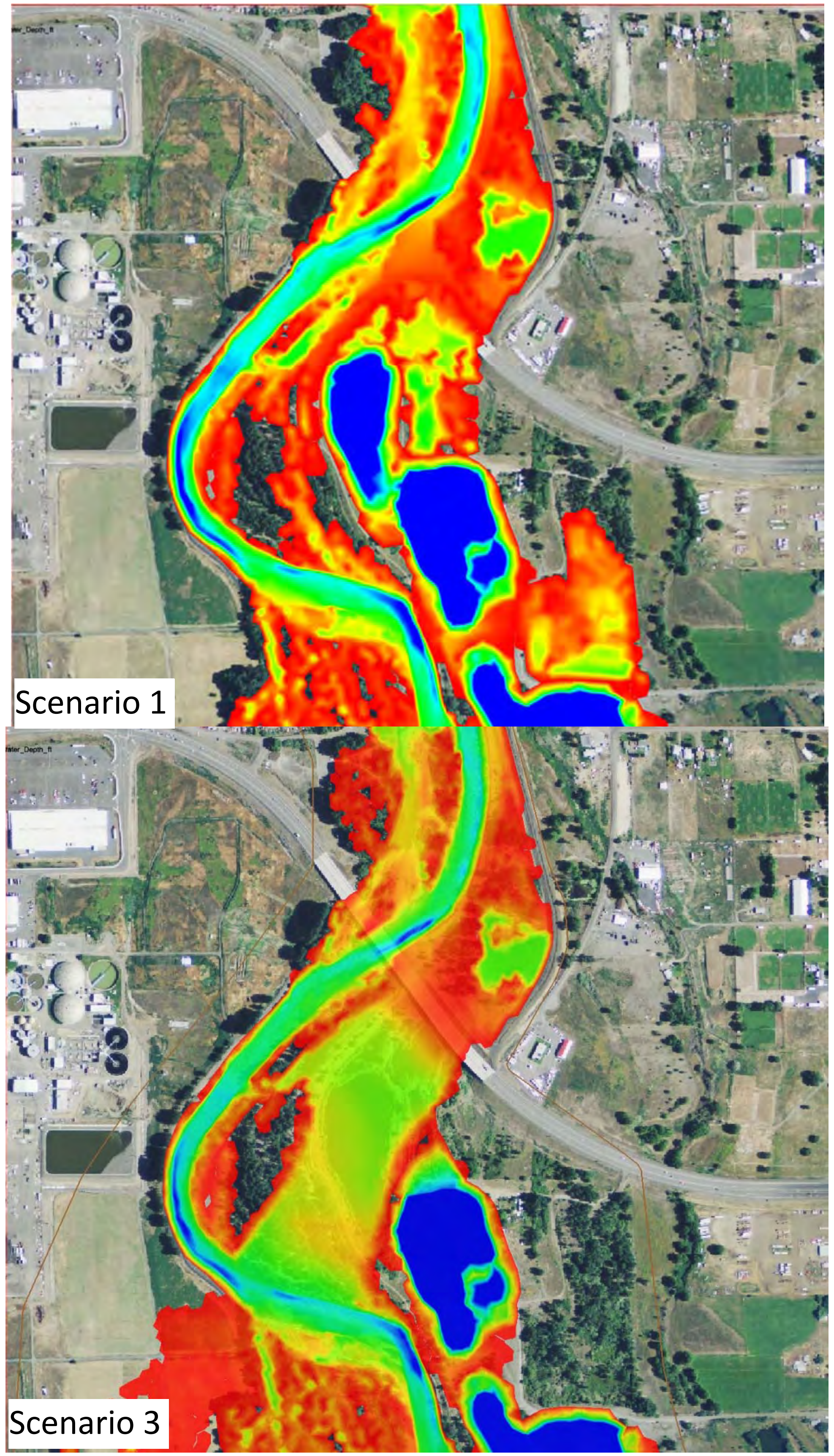

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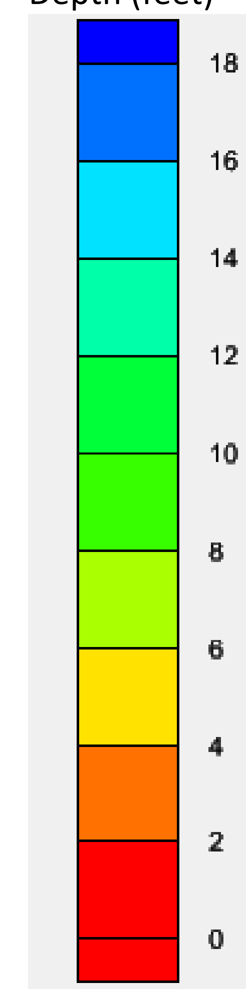
Computed 10-year Flow Depth

Legend

Depth (feet)



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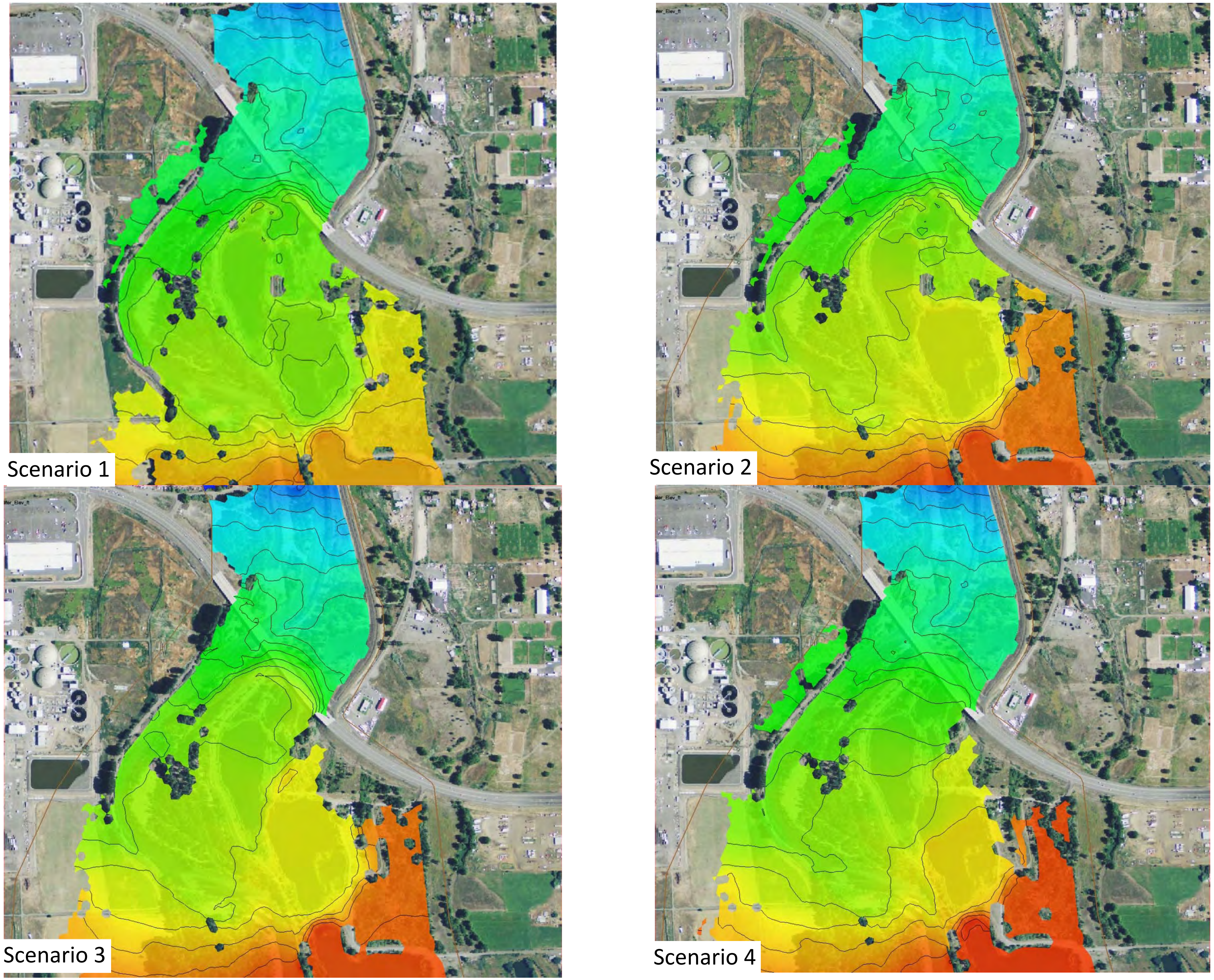

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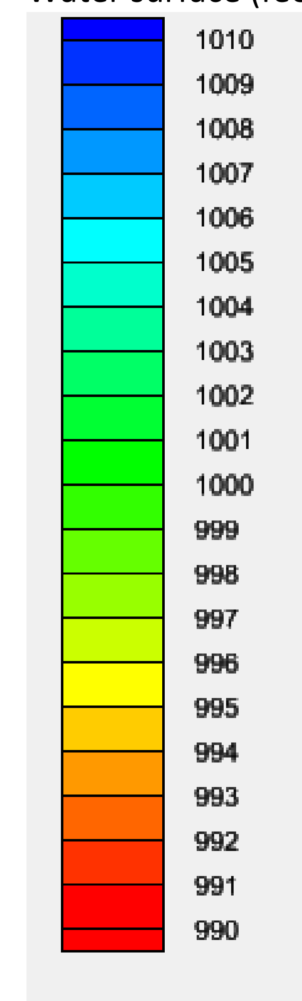
Computed 100-year
Water Surface Elevation

Legend
Water surface (feet, NAVD 88)



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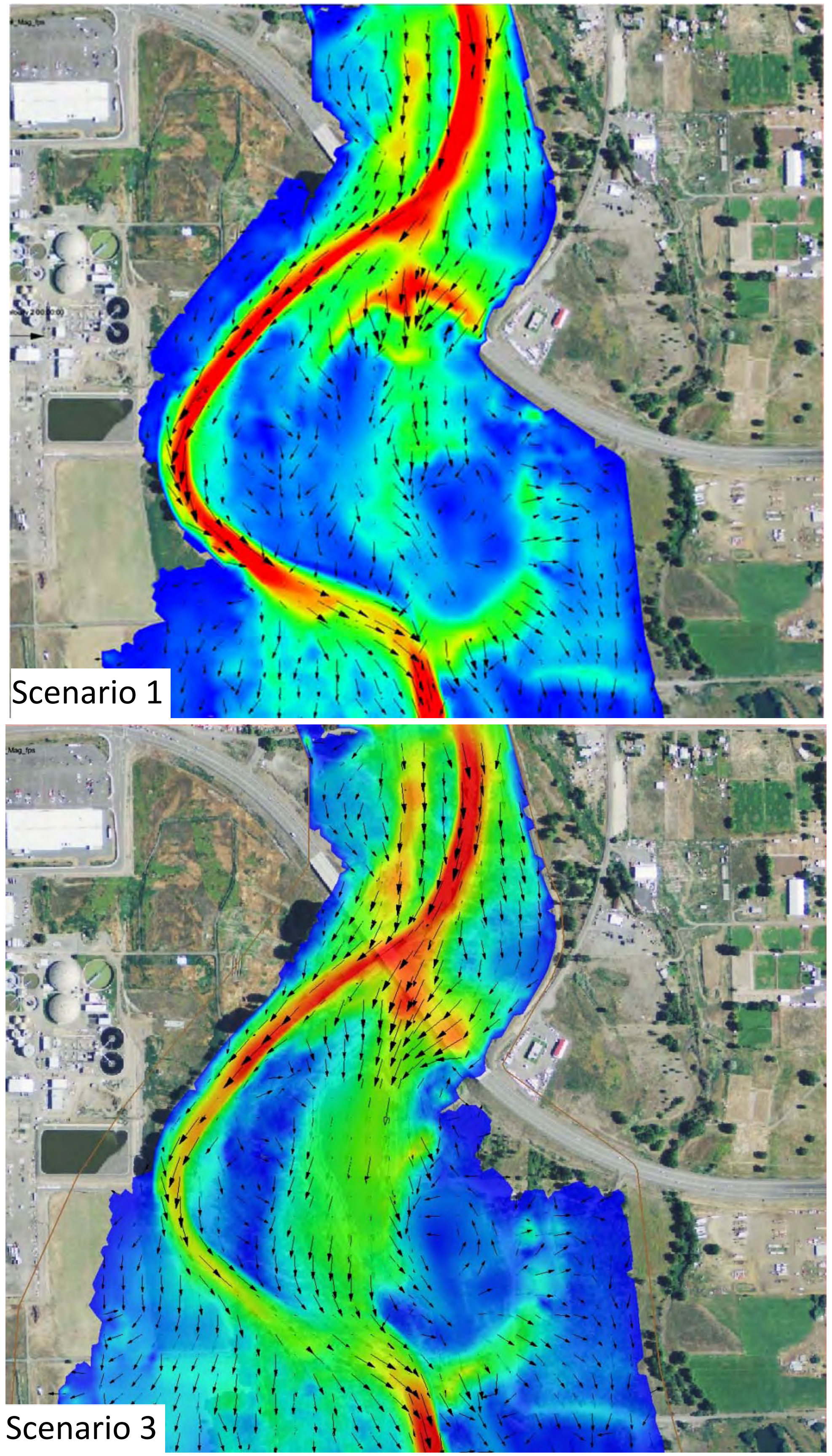

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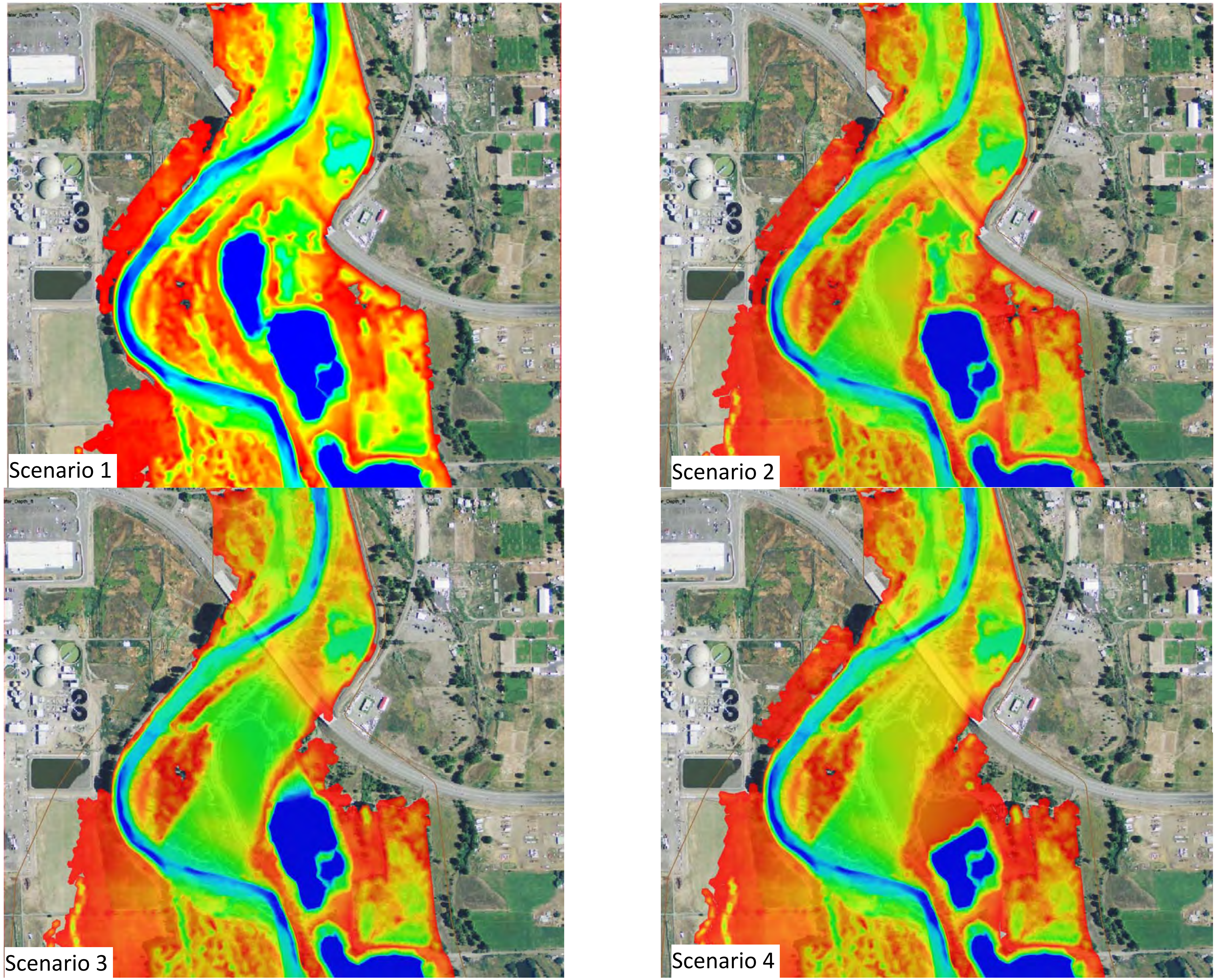
Computed 100-year Flow Velocity

Legend
Velocity (feet per second)

14
12
10
8
6
4
2
0

21792
April 15, 2015



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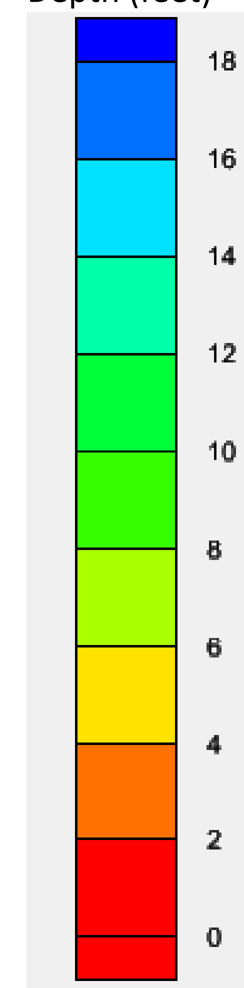
Computed 100-year
Flow Depth

Legend
Depth (feet)



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