Appendix D. Hydraulic Design

- Spillway Alternative Selection TM
- Fish Passage Conduits TM
- Construction Bypass TM

April 25, 2024

Technical Memorandum

Date:	April 24, 2024
Project:	Chehalis River Basin Flood Damage Reduction Project
To:	Chehalis Basin Flood Control Zone District
From:	HDR Engineering, Inc.
Subject:	**Spillway Alternative Selection**

1.0 Background

The Chehalis River Basin Flood Damage Reduction project objective is to develop recommendations for a series of measures aimed at reducing damage to the communities of the Chehalis River Basin from Pe Ell to Centralia during major flood events. Among these measures is a proposed Flood Retention Expandable (FRE) structure on the Chehalis River, south of the town of Pe Ell.

The Chehalis River Basin Flood Damage Reduction, Revised Project Description Report (RPDR) is a supplemental report documenting the relocation of and changes to the FRE facility (Proposed Project) as originally documented within the Combined Dam and Fish Passage Conceptual Design Report (HDR Engineering, Inc. [HDR] 2017) and FRE Dam Alternative Report (HDR 2018).

The RPDR describes, supports, contrasts, and illustrates the changes to the Proposed Project in a single comprehensive document.

2.0 Introduction and Purpose

As one document in Appendix D to the RPDR, this Technical Memorandum documents the hydraulic analysis of the spillway alternatives and recommendations for the selected alternative.

3.0 Pertinent Data

Table 1 shows the pertinent data used in the hydraulic design of the spillway. The vertical datum for the project is the North American Vertical Datum of 1988 (NAVD88). As the hydrology is still being revised, the originally developed probable maximum flood (PMF) inflow estimate of 69,800 cubic feet per second (cfs) was used for the design flow. After the hydrology is finalized, these values will be updated. The tailwater elevation is based on a 1D HEC-RAS model and may change with updated hydrology. Any changes to the pertinent data likely would affect the spillway design.

Table 1. Pertinent Data for Hydraulic Design of Spillway

Description	Value*
Design Flow (Probable Maximum Flood)	69,800 cfs
Tailwater Elevation at the Design Flow	473.25 ft
Dam Upstream Face Slope	0.10:1 (H:V)
Dam Downstream Face Slope	0.85:1 (H:V)
FRE Spillway Crest Elevation	628.0 ft
FRE Top of Dam Elevation	651.0 ft
FRE-FC Spillway Crest Elevation	691.1 ft
FRE-FC Top of Dam Elevation	714.1 ft

*Vertical Datum is NAVD88. cfs = cubic feet per second. ft = feet. H:V = horizontal to vertical. FRE-FC = Flood Retention Expandable, Future Construction

4.0 Spillway Alternatives

Three spillway alternatives were evaluated that utilize an elliptical crest shape, as defined in Engineering Manual (EM) 1110-2-1603 (USACE 1992). The spillway chute for each alternative is assumed to converge at the same rate as the radius of the dam, which is ideal for structural design as the spillway is maintained on the same structural monolith.

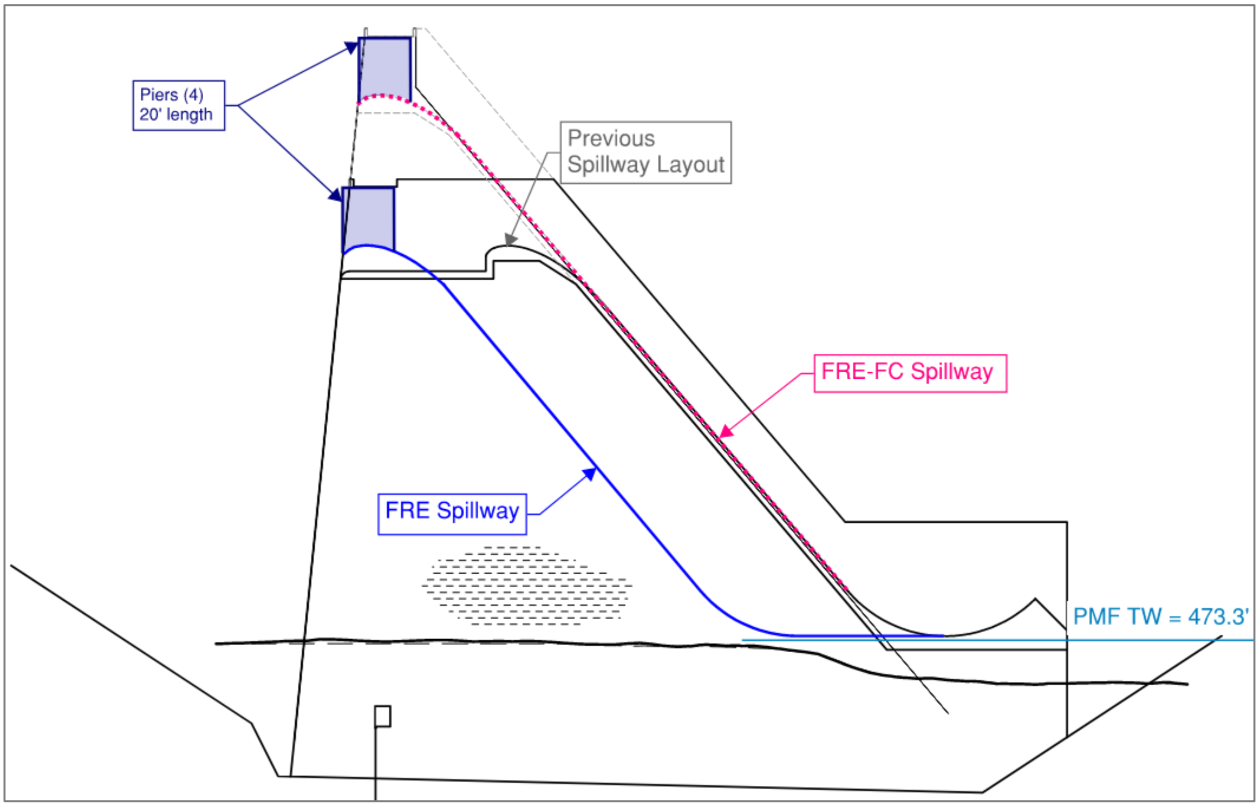
All spillway alternatives for the FRE condition were moved upstream to match the non-overflow FRE dam section, which complicates how the FRE and the FRE-FC conditions overlap and the stilling basin design, but simplifies the structural design of the training walls.

4.1 Flip Bucket Spillway

The flip bucket spillway consists of a smooth spillway chute, converging at the radius lines of the dam and terminating at a flip bucket which would direct the flow farther downstream. Due to the curvature of the dam, the flow likely will be more concentrated and behave like a jet leaving the flip bucket raising concerns about downstream erosion on sensitive areas.

Moving the spillway back to match the non-overflow sections of the FRE dam complicates how the flip bucket and spillway interact, as there would be more than 55 feet of horizontal distance between the back curve and the flip bucket (Figure 1).

Figure 1. Flip Bucket Spillway Alternative



Due to downstream erosion concerns in sensitive cultural areas, and complicated flip bucket behavior at the FRE and FRE-FC conditions, the flip bucket spillway alternative was not assessed further.

4.2 Smooth Spillway

In a smooth spillway, smooth means the spillway is without obstructions or irregularities down the spillway face. This alternative is designed to pass the design flow over an uncontrolled spillway crest and dissipate energy in a stilling basin.

4.2.1 Spillway Crest and Rating Curve

Design parameters for the smooth spillway are presented in Table 2. The spillway crest elevation is the only difference between the FRE and FRE-FC conditions, therefore, the shape, length, and discharge capacity of the spillway crest is consistent under both conditions. The crest of the smooth spillway is based on an elliptical crest shape, which has an upstream elliptical shape and a downstream ogee shape. The ogee shape transitions seamlessly to the downstream slope (0.85H:1V). To improve crest efficiency and reduce the spillway length, the ratio of the upstream energy head (H_e) is compared to the design head (H_d) of the spillway crest. The design head references the shape of the spillway at which the spillway shape matches the nappe of the flow, or the pressures on the spillway crest would be zero. The design goal is to have an H_e/H_d ratio less than or equal to 1.33.

Table 2. Smooth Spillway – Design Parameters

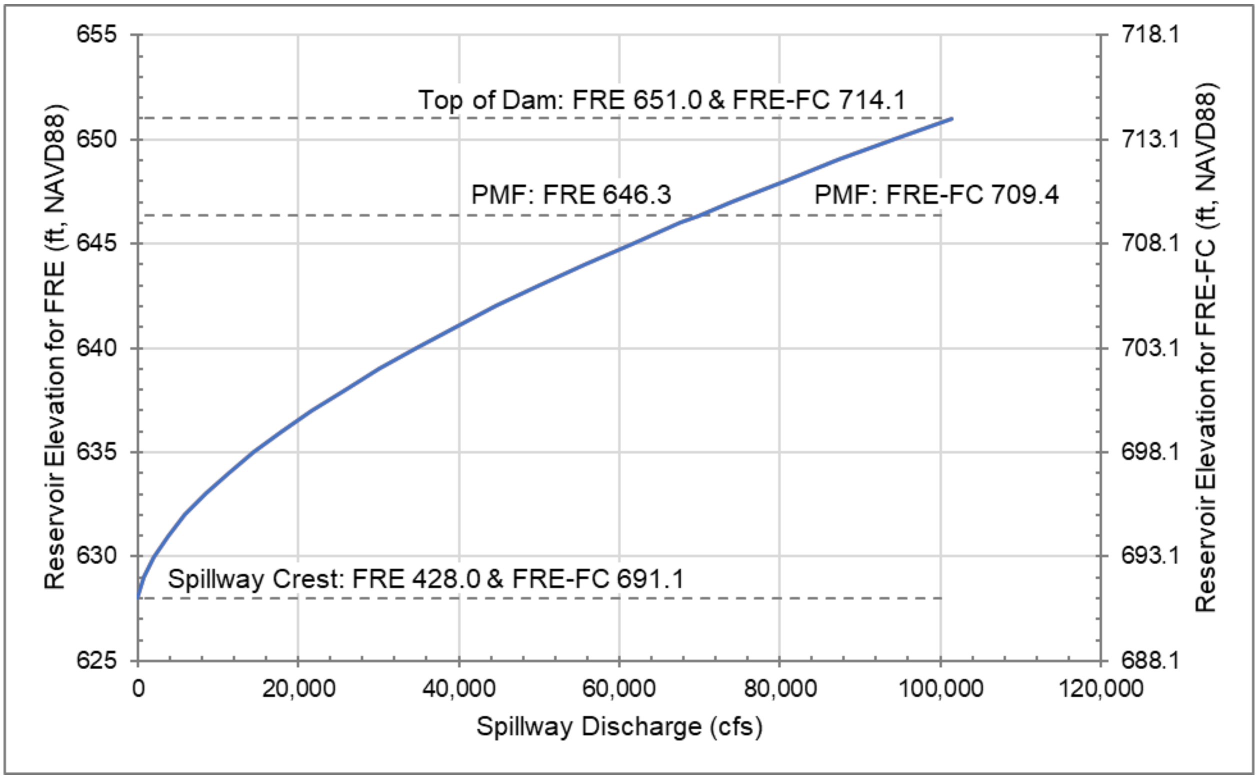
Description	Value*
Design Flow (Probable Maximum Flood)	69,800 cfs
Spillway Length at Crest, *L*	230 ft
Number of Piers	4
Pier Width at Crest	4 ft
Spillway Design Head, H_d	14 ft
Energy Head to Design Head Ratio at Design Flow, H_e/H_d	1.31
Discharge Coefficient at Design Flow, *C*	4.21
FRE Pool Elevation at Design Flow	646.3 ft
FRE-FC Pool Elevation at Design Flow	709.4 ft
FRE Unit Discharge range between Crest and Stilling Basin	326 and 349 cfs/ft
FRE-FC Unit Discharge range between Crest and Stilling Basin	326 and 368 cfs/ft

*Vertical Datum is NAVD88.

The spillway design head was selected so the crest is under designed to an H_e/H_d ratio of 1.3, according to EM 1110-2-1603 (USACE 1992). The spillway includes four Type 2 (EM 1110-2-1603) piers to support a continuous access road along the FRE crest. The piers were assumed to be 4 feet wide at the crest, transitioning to 3 feet wide at the downstream end, resulting in a clear span between piers of 53.5 feet.

Figure 2 shows the rating curve for the FRE and FRE-FC conditions, with the FRE and FRE-FC reservoir elevations reported on the left and right axis, respectively.

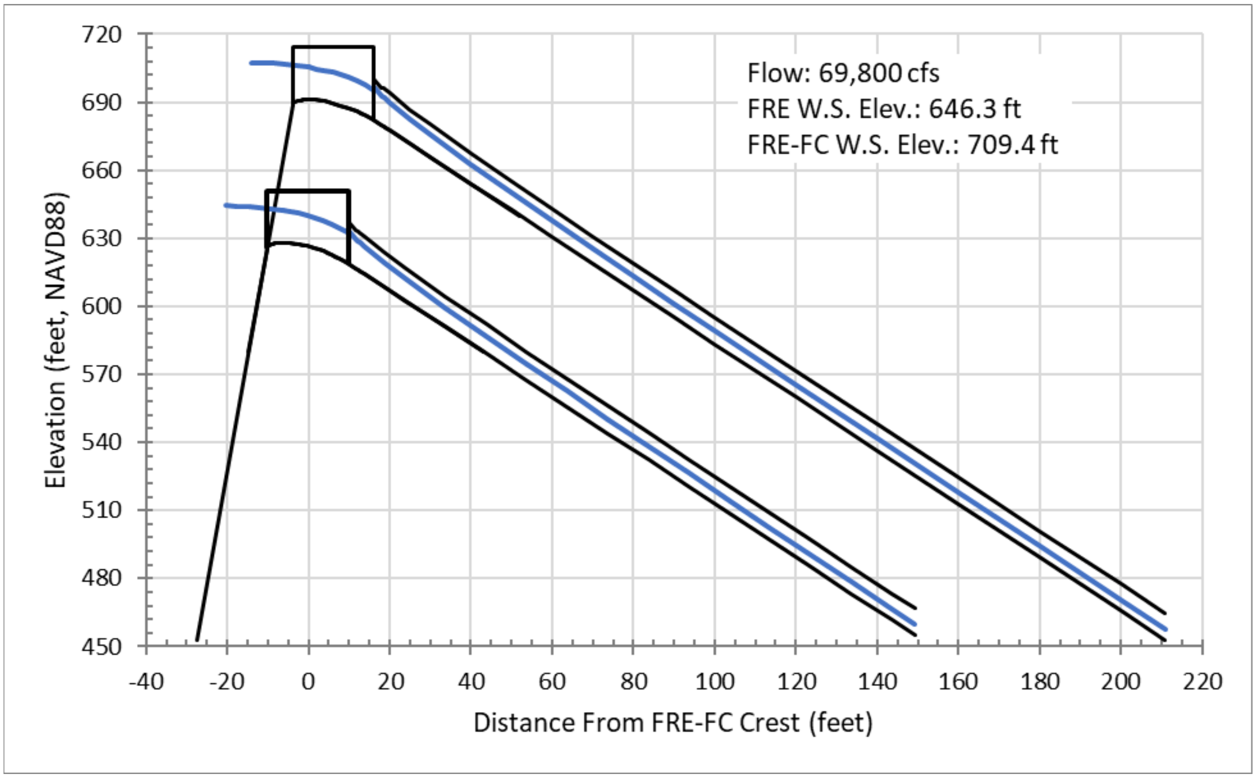
Figure 2. Smooth Spillway – Rating Curve



4.2.2 Water Surface Profile and Wall Heights

Figure 3 illustrates the computed water surface profiles for the FRE and FRE-FC conditions, plotted in reference to the distance along the FRE-FC crest. The computed average velocities at the base of the spillways and design flow are 107 and 122 feet per second (ft/s) for the FRE and FRE-FC, respectively. These velocities are high and have the potential for cavitation damage on the spillway chute and in the stilling basin, depending on the type of stilling basin selected.

Figure 3. Water Surface Profiles for the Smooth Spillway



The piers on the crest were assumed to be 20 feet long, starting at the upstream point of tangency, with a standard Type 2 nose (bullnose) shape on the upstream face. The minimum height of the training walls was estimated using EM 1110-2-1603, which is based on a conservative empirical criterion developed by U.S. Bureau of Reclamation (Reclamation; USACE 1992) and an empirical approximation to account for air entrainment and waves in the spillway resulting in a minimum wall height that varies from 18.3 feet at the end of the piers to 11.8 feet at the stilling basin invert.

4.2.3 Stilling Basin

Stilling basins dissipate energy by rapidly reducing the velocity over a short length, also known as a hydraulic jump. The stilling basin is sized to fully contain the hydraulic jump at the design flow and the invert is set so the conjugate depth of the incoming flow matches the tailwater elevation. A deeper invert is possible if preferred for structural considerations, but the minimum invert levels are reported. The stilling basin width is based on the spillway width at the stilling basin invert elevation. Table 3 presents the stilling basin design parameters for the FRE and FRE-FC conditions based on the flow profile entering the stilling basins.

Table 3. Smooth Spillway – Stilling Basin Design Parameters

Description	Value, FRE*	Value, FRE-FC*
Design Flow (PMF)	69,800 cfs	69,800 cfs
Unit Discharge	349 cfs/ft	368 cfs/ft
Depth, d_1	3.26 ft	3.02 ft
Froude Number, F_1	10.45	12.35
Conjugate depth, d_2	17.5 ft	20.0 ft
Required Floor Invert Elevation	455 ft	453 ft
Stilling Basin Width	200.1 ft	189.4 ft

*Vertical Datum is NAVD88.

The FRE and FRE-FC structures are curved with a radius of approximately 1,200 feet. The smooth spillway converges to align the walls of the spillway with the radial lines of the structure. This convergence results in a stilling basin width under the FRE and FRE-FC conditions of 200.1 feet and 189.4 feet, respectively.

Table 4 presents the required length of three types of stilling basins for the FRE and FRE-FC smooth spillways. Detailed descriptions of each type of stilling basin can be found in Design of Small Dams, Third Edition (Reclamation 1987). A Type I basin consists of a concrete apron with vertical walls parallel to flow and is typically the longest. A Type II basin has a dentated endsill and is best suited for high velocities but is sensitive to tailwater elevation. A Type III basin has baffle piers and an endsill that has the shortest length compared to the other two basin types. Velocities are recommended to not exceed 50 to 60 ft/s as the baffle piers are prone to cavitation damage. Velocities for the smooth spillway are well above the recommended limit and would require a specialized baffle pier called a supercavitating baffle block to prevent damage.

Table 4. Smooth Spillway – Stilling Basin Required Dimensions

Description	Value, FRE (ft)	Value, FRE-FC (ft)
Type I Basin Length	107	121
Type II Basin Length	76	86
Dentated Sill Height	3.5	4.0
Type III Basin Length	48	55
Baffle Pier Height	7.7	8.1
Endsill Height	5.2	5.1

4.3 Stepped Spillway

A stepped spillway incorporates vertical steps inset into the spillway face to dissipate energy through deflecting a portion of the nappe back on itself. This energy dissipating reduces the required size of the terminal energy dissipation structure. The step height is a sensitive and critical parameter to ensure proper energy dissipation at larger unit flows and flow depths. This alternative is designed to pass the design flow uncontrolled over the spillway crest and dissipate the remaining energy in a stilling basin.

4.3.1 Spillway Crest and Rating Curve

Design parameters for the stepped spillway are presented in Table 5. As discussed in Section 4.2.1, the FRE and FRE-FC conditions yield identical spillway crest shapes. The crest of the stepped spillway is based on an elliptical crest shape, which has an upstream elliptical shape and a downstream ogee shape. The ogee shape transitions seamlessly to the downstream slope (0.85H:1V). To prevent flow from skipping steps and optimize the energy dissipation of the steps, the ratio of the upstream energy head (H_e) is compared to the design head (H_d) of the spillway crest. The design head references the shape of the spillway at which it matches the nappe of the flow, or the pressures on the spillway crest will be zero. The design goal is to have an H_e/H_d ratio less than or equal to 0.90. This would keep the flow in positive contact with the stepped spillway and prevent it from springing free of the spillway once the steps begin and reattaching further downstream (i.e., skipping steps), as described in Hydraulic Laboratory Report HL-2015-06 (Reclamation 2015). Consistent with the smooth spillway alternative, the stepped spillway was analyzed to include four Type 2 (EM 1110-2-1603) piers to support a continuous access road along the FRE crest. The piers were assumed to be 4 feet wide at the crest, transitioning to 3 feet wide at the downstream end resulting in a clear span between piers of 60 feet.

Table 5. Stepped Spillway – Design Parameters

Description	Value*
Design Flow (Probable Maximum Flood)	69,800 cfs
Spillway Length at Crest, L	316 ft
Step Height	4 ft
Number of Piers	4
Pier Width at Crest	4 ft
Spillway Design Head, H_d	17 ft
Energy Head to Design Head Ratio at Design Flow, H_e/H_d	0.90
Discharge Coefficient at Design Flow, C	3.96
FRE Pool Elevation at Design Flow	643.3 ft
FRE-FC Pool Elevation at Design Flow	706.4 ft

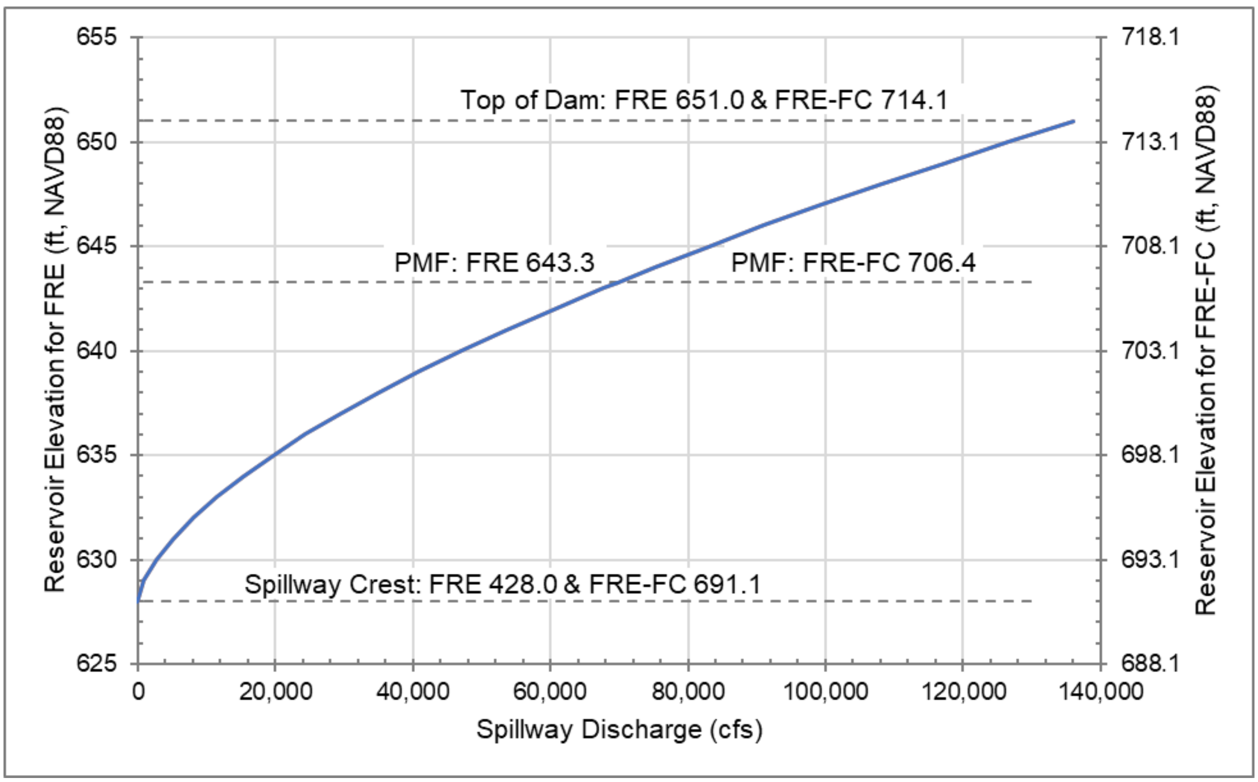
Description	Value*
FRE Unit Discharge range between Crest and Stilling Basin	232 and 260 cfs/ft
FRE-FC Unit Discharge range between Crest and Stilling Basin	232 and 275 cfs/ft

*Vertical Datum is NAVD88

Steps were assumed to begin at the end of the piers with 1-foot steps, and gradually increase in height to 4-foot steps, with the first achieved at the tangency point with the downstream slope. The optimal step height of 4.4 feet was calculated based on being one-third of the critical depth at a unit discharge of 275 cfs/ft (HL-2015-06) at the end of the spillway where flow is most constricted by the converging walls. Desiring to keep the step height at whole foot increments, matching the proposed roller-compacted concrete (RCC) construction practices, a 4-foot step height was chosen.

Figure 4 shows the rating curve for the FRE and FRE-FC conditions, with the FRE and FRE-FC reservoir elevations reported on the left and right axis, respectively.

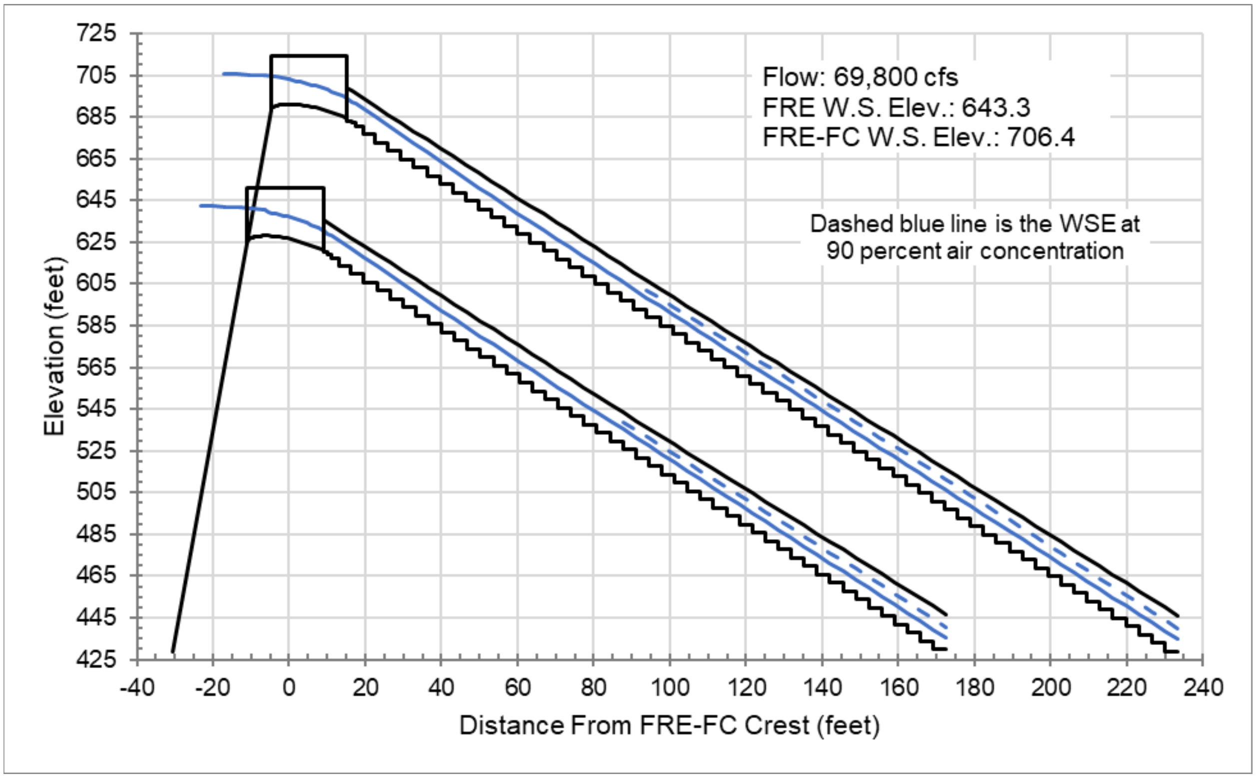
Figure 4. Stepped Spillway – Rating Curve



4.3.2 Water Surface Profile and Wall Heights

Figure 5 illustrates the computed water surface profiles for the FRE and FRE-FC conditions, plotted in reference to the distance along the FRE-FC crest. The computed average velocities at the base of the spillways and design flow are 70 and 72 ft/s for the FRE and FRE-FC, respectively. Because these velocities are high, the cavitation potential on the stepped spillway and stilling basin will need to be further evaluated and considered.

Figure 5. Water Surface Profiles for the Stepped Spillway



The flow regime at the design flow on both the FRE and FRE-FC conditions is characterized as skimming flow, where the flow skims in a reasonably coherent stream along a line connecting the tips of the steps, and dissipate energy as a portion of the nappe is directed back onto itself. Figure 5 shows the water surface profile for skimming flow approximated by assuming the steps act as a roughened surface.

The converging spillway prevents uniform flow from occurring. Air entrainment is expected to occur approximately 116 feet from the FRE and FRE-FC crests (Hunt and Kadavy 2014). Beyond this point, the flow profile is assumed to be highly aerated, and aerated flow depth is approximated when air concentration equals 90 percent.

The piers on the crest were assumed to be 20 feet long, starting at the upstream point of tangency, with a standard Type 2 nose (bullnose) shape on the upstream face. The piers are assumed to be 4 feet wide at the crest and tapering to 3 feet wide at the downstream end of the pier. The minimum height of the training walls was estimated based on the aerated flow depth when the air concentration equals 90 percent plus a factor of safety of 1.5 (Hunt and Kadavy 2014) resulting in a minimum wall height that varies from 14.7 feet at the end of the piers to 16.6 feet at the beginning of the stilling basin. The stilling basin walls are approximately 48 feet tall based on the deepening of the invert due to the estimated bedrock elevation.

4.3.3 Stilling Basin

Stilling basins dissipate energy by rapidly reducing the velocity over a short length, also known as a hydraulic jump. The stilling basin is sized to fully contain the hydraulic jump at the design flow with the invert set so the conjugate depth of the incoming flow matches the tailwater elevation. The stepped spillway stilling basin design follows the same design process as the smooth spillway from Section 4.2.2. Table 6 presents the stilling basin design parameters for the FRE and FRE-FC conditions based on the flow profile entering the stilling basins. Note that the depth entering the stilling basin, d_1, is the clear water depth, meaning no-air entrainment. The flow would be highly aerated due to the steps and bulked in volume. The recommended floor invert elevations in Table 6 are lower than the minimum required elevations for proper hydraulic function. The lower elevation was selected by the design team to minimize concrete buildup from the estimated bedrock elevations. A full step height was maintained to integrate with the proposed RCC construction method.

Table 6. Stepped Spillway – Stilling Basin Design Parameters

Description	Value, FRE*	Value, FRE-FC*
Design Flow (PMF)	69,800 cfs	69,800 cfs
Unit Discharge	260 cfs/ft	275 cfs/ft
Depth, clear water, d_1	3.72 ft	3.83 ft
Froude Number, F_1	6.39	6.47
Conjugate depth, d_2	12.3 ft	12.5 ft
Required Floor Invert Elevation[1]	459.8 ft	459.5 ft
Recommended Floor Invert Elevation[2]	429.7 ft	428.8 ft
Stilling Basin Width	268.6 ft	254.2 ft

*Vertical Datum is NAVD88

[1] The required floor invert elevation is the maximum elevation for hydraulic function of stilling basin.

[2] The recommended floor invert elevation is based on the design teams request and estimated bedrock elevations.

Consistent with the smooth spillway alternative, the stepped spillway converges to align the walls of the spillway with the radial lines of the structure. The convergence results in a stilling basin width under the FRE and FRE-FC conditions of 268.6 and 254.2 feet respectively.

Table 7 presents the required length of three types of stilling basins for the FRE and FRE-FC stepped spillways. See Section 4.2.2 for detailed descriptions of each type of stilling basin as found in Reclamation (1987). Like the smooth spillway, the stepped spillway, velocities also are above the recommended limit and would require a specialized baffle pier called a supercavitating baffle to prevent damage (Reclamation 1987).

Table 7. Stepped Spillway – Stilling Basin Required Dimensions

Description	Value, FRE (ft)	Value, FRE-FC (ft)
Type I Basin Length	75	76
Type II Basin Length	50	51
Dentated Sill Height	2.5	2.5
Type III Basin Length	31	31
Baffle Pier Height	6.2	6.5
Endsill Height	5.1	5.2

Due to the increased energy dissipation of the stepped spillway, there is minimal difference between the stilling basin lengths between the FRE and FRE-FC conditions for all basin types.

5.0 Recommendations

5.1 Discussion

The design team primarily considered two alternatives, a smooth spillway with a stilling basin and a stepped spillway with a stilling basin, assuming the preferred alternative would be consistent between the FRE and FRE-FC conditions.

The smooth spillway requires a large Type I stilling basin (greater than 100 feet under both conditions) with Type II and Type III stilling basins shorter. For a Type III stilling basin, however, the velocities entering the stilling basins would be greater than 100 ft/s which would require a special baffle blocks design to address cavitation damage. Additionally, approximately 55 feet of the FRE stilling basin would lie within the footprint of the FRE-FC. The FRE stilling basin would either need to be demolished, or oversized and extended to the required FRE-FC endsill and then partially covered by the FRE-FC structure.

The stepped spillway construction aligns well with the proposed RCC construction method for concurrent construction with the RCC. The stepped spillway also requires a shorter stilling basin and more moderate velocities. Because the stilling basin sizes are similar as the overlap between the FRE and FRE-FC conditions, it is more reasonable from a cost and constructability perspective to construct the FRE stilling basin as required. The FRE-FC would simply cover the FRE stilling basin and require a new stilling basin to be constructed.

5.2 Recommendation for Preliminary Design

A stepped spillway with 4-foot steps as presented in Section 4.3 is recommended for the selected conceptual design. A Type II stilling basin is recommended because it is shorter than a Type I basin and the 2.5-foot-high dentated endsill is simpler to construct compared to the larger endsill and baffle piers required for a Type III basin. The FRE stilling basin sits within the footprint of the FRE-FC spillway. The stilling basin invert should be located at around elevation 430 feet, which is deeper than hydraulically needed, but closer to the estimated bedrock elevation. To adequately contain the aerated flow entering the stilling basin and hydraulic jump in the stilling basin, a top of wall elevation of 477 feet is required for both the FRE and FRE-FC conditions. The wall heights range from 15 to 16 and 15 to 17 feet tall (measured vertically from the tips of the steps) for the FRE and FRE-FC conditions, respectively.

Figure 6 and Figure 7 illustrate the recommended conceptual spillway and stilling basin described in this section for the FRE and FRE-FC conditions, respectively.

Figure 6. Recommended FRE Spillway and Stilling Basin

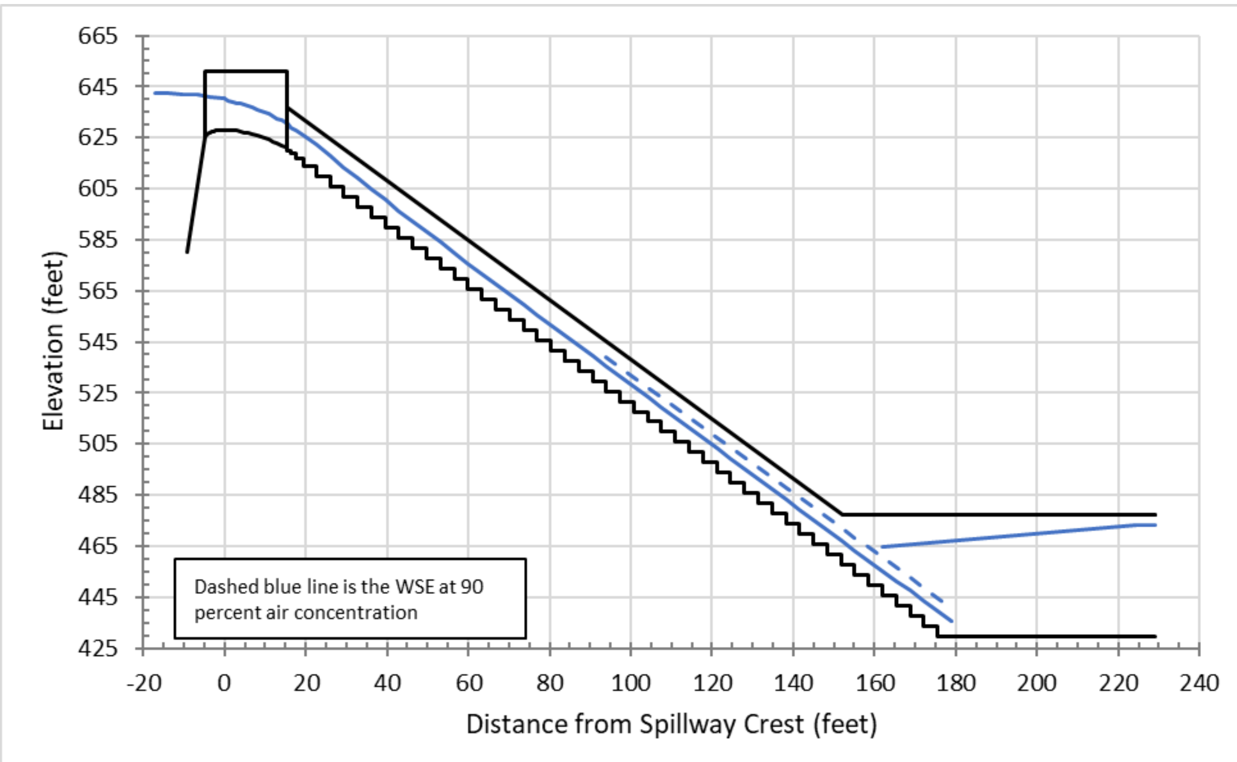
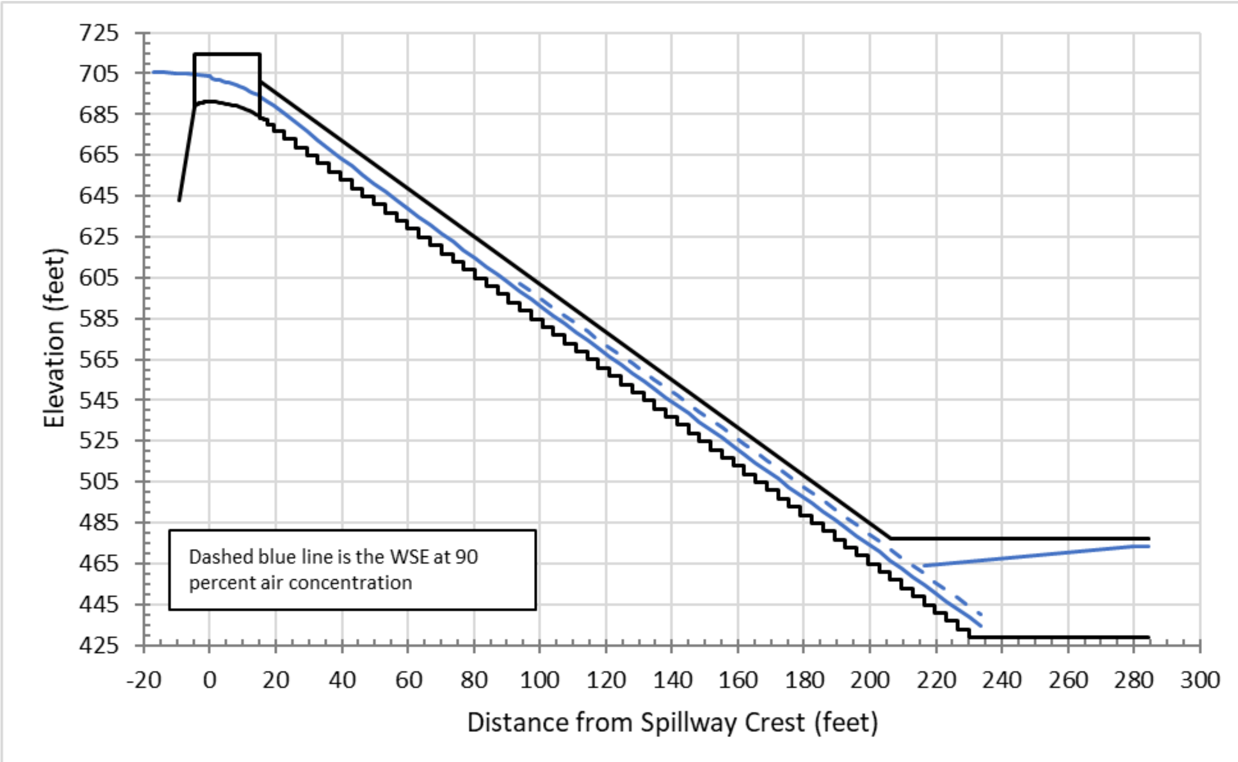


Figure 7. Recommended FRE-FC Spillway and Stilling Basin



6.0 References

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2015 Guidelines for Hydraulic Design of Stepped Spillways. A Water Resources Technical Publication, Hydraulic Laboratory Report HL-2015-06. U.S. Department of the Interior, Bureau of Reclamation, Technical Service Center. September 2015.

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1992 Hydraulic Design of Spillways. EM 1110-2-1603. Department of the Army, U.S. Army Corps of Engineers, Washington DC. August 31, 1992.

7.0 Abbreviations

cfs	cubic feet per second
District	Chehalis Basin Flood Control Zone District
EM	Engineering Manual
FRE	Flood Retention Expandable
FRE-FC	Flood Retention Expandable, Future Construction
ft	feet
ft/s	feet per second
H:V	horizontal to vertical
H_e	energy head
H_d	design head
PMF	probable maximum flood
RCC	Roller Compacted Concrete
USACE	U.S. Army Corps of Engineers

Technical Memorandum

Date:	April 24, 2024
Project:	Chehalis River Basin Flood Damage Reduction Project
To:	Chehalis Basin Flood Control Zone District
From:	HDR Engineering, Inc.
Subject:	**Hydraulic Design of Fish Passage and Evacuation Conduits**

1.0 Background

The Chehalis River Basin Flood Damage Reduction project objective is to develop recommendations for a series of measures aimed at reducing damage to the communities of the Chehalis River Basin from Pe Ell to Centralia during major flood events. Among these measures is a proposed Flood Retention Expandable (FRE) structure on the Chehalis River, south of the town of Pe Ell.

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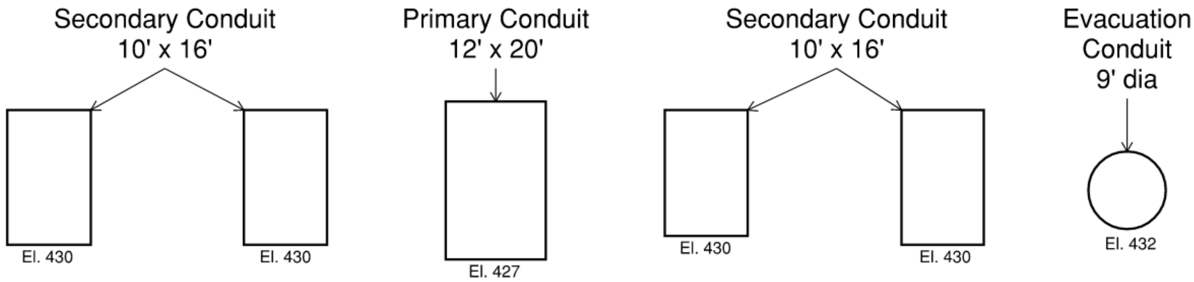
The RPDR describes, supports, contrasts, and illustrates the changes to the Proposed Project in a single comprehensive document.

2.0 Introduction and Purpose

As one document in Appendix D to the RPDR, this Technical Memorandum documents the hydraulic analysis of the fish passage and evacuation conduits. The fish passage conduits are primarily intended to function with the FRE structure and were designed for fish passage flows that were scaled for climate change, where the gates are normally open for fish passage and only closed for flood retention. The conduits are designed to mimic the hydraulic characteristics of the normal river flows when compared to the existing rock channel downstream of the dam. When the fish passage conduit gates are closed, the evacuation conduit will be used for reservoir releases. Figure 1 shows an overview layout of the fish passage and evacuation conduits, based on a section view looking downstream.

Figure 1. Fish Passage and Evacuation Conduits Overview



El. = elevation; dia = diameter.

3.0 Pertinent Data

The fish passage conduits are primarily intended to function with the FRE structure, where the gates are normally open for fish passage and only closed for flood retention. During initial phases of flood retention, the fish passage gates would be used to regulate the flow until flow control is transferred to the evacuation conduit. After the fish passage gates are closed, the evacuation conduit would be used for reservoir releases. Three of the five fish passage conduits would be permanently closed for the flood retention expandable – future condition (FRE-FC) structure. The two remaining fish passage conduits would be used for emergency flood releases or reservoir drawdown. For the FRE-FC, normal flow releases would be through the water quality ports.

Figure 2 shows a schematic layout of the primary and secondary fish passage conduits. The primary fish passage conduit would maintain a constant width of 12 feet from inlet to outlet. The secondary fish passage conduits would have a minimum width of 10 feet, while two conduits merge into one as they approach the outlet. The convergence is a function of the radial orientation that aligns with the dam’s curvature and maintaining a minimum wall thickness between the conduits.

Figure 2. Schematic Fish Passage Conduit Layout

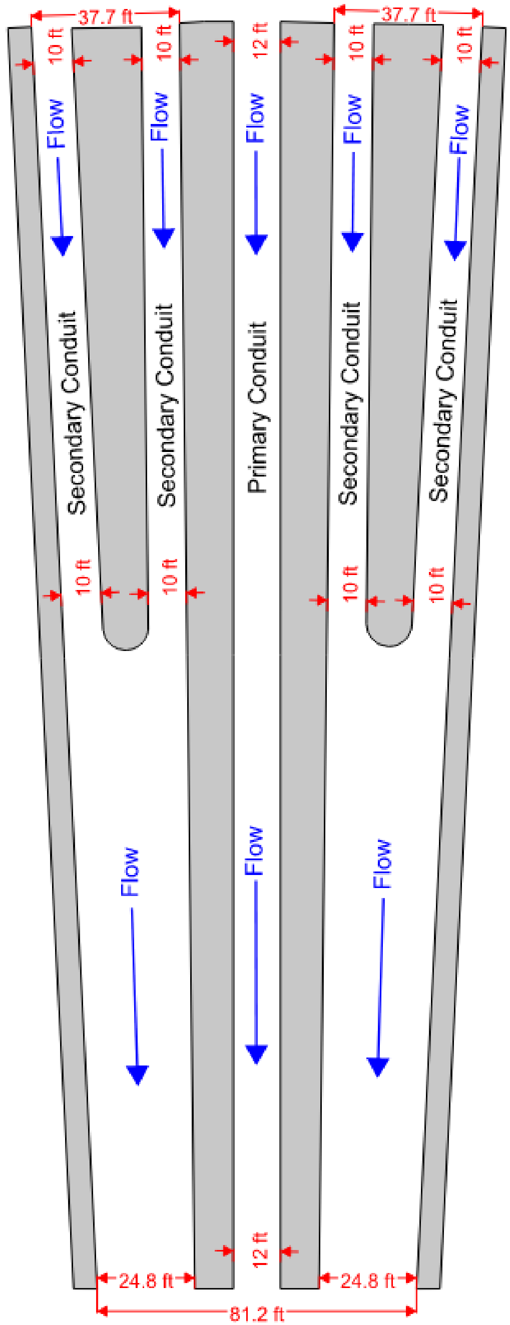


Table 1 shows the pertinent data used in the hydraulic design of the fish passage conduits. The vertical datum for the project is the North American Vertical Datum of 1988 (NAVD88). The fish passage conduits are designed to pass the 5 percent and 95 percent exceedance flow to meet FRE-FC fish passage requirement and regulating flows. The evacuation conduit is designed to regulate flows and meet required drawdown rates during an emergency reservoir drawdown. Because the hydrology is still being revised, the originally developed flow assumptions were used for designing the stilling basin. Once the hydrology is finalized, the fish passage conduits and stilling basin design will be re-evaluated and revised as appropriate. The tailwater elevation is based on a one-dimensional (1D) HEC-RAS model and may change with updated hydrology and modeling updates. The PMF tailwater is provided as informational to verify the fish passage conduits are designed in accordance with the maximum tailwater elevation. Any changes to the pertinent data likely would affect the fish passage conduit design.

Table 1. Pertinent Data for Hydraulic Design of Conduits

Description	Primary Conduit	Secondary Conduit
Number of Conduits	1	4
Fish Passage Conduit Width	12 ft	10 ft
Fish Passage Conduit Height	20 ft	16 ft
Invert Elevation at Control Gate	427 ft	430 ft
Conduit Crown Elevation	447 ft	446 ft
Evacuation Conduit Diameter	9 ft	
Evacuation Conduit Invert Elevation	432.0 ft	
Total Stilling Basin Width	81.2 ft	
Stilling Basin Invert Elevation	412 ft	
Stilling Basin Endsill Elevation	436 ft	
Stilling Basin Length	110 ft	
95% Exceedance Flow for Fish Passage	16 cfs	
Climate Change 95% Exceedance Flow for Fish Passage	14 cfs	
5% Exceedance Flow for Fish Passage	2,200 cfs	
Climate Change 5% Exceedance Flow for Fish Passage	3,400 cfs	
FRE-FC Reservoir Evacuation Rate	7,400 cfs	
100-yr Flow, 1% Annual Exceedance Probability (AEP)	38,000 cfs	
95% Exceedance Tailwater Elevation	436 ft	
5% Exceedance Tailwater Elevation	437.27 ft	
100-yr Tailwater Elevation	460.86 ft	
PMF Tailwater Elevation	473.25 ft	

*Vertical Datum is NAVD88. ft = feet.

4.0 Outlet Works Workshop

The following is a summary of the project team's decisions regarding the outlet works function and requirements.

4.1 Emergency Reservoir Evacuation

An initial evaluation of the emergency reservoir evacuation rate was performed to inform the general layout, shape, location, equipment sizing, and general operation. The evacuation time was determined using U.S. Bureau of Reclamation guidance (2022) along with a high hazard dam assumption to set the required reservoir drawdown rates. The required discharge to meet the evacuation times was evaluated by looking at a simplified daily average flow method and a level pool routing using a historical hydrograph from the 2009 flood event. An FRE-FC evacuation rate of 7,400 cfs was determined, which includes a factor of safety of 1.33 to account for uncertainty due to the early design stages.

4.2 Fish Passage Conduits

Three of the five fish passage conduits would have a reduced operating condition and only operate at differential heads of less than 100 feet providing simpler gate design and operation. Two gates on the secondary fish passage conduits would utilize a high head bonneted slide gate that could operate at FRE-FC pool elevations as a redundancy to release large flows or for emergency reservoir evacuation.

Overall, the fish passage conduits would provide fish passage, meeting fish passage criteria, for river flows up to the 5 percent exceedance probability. The fish passage conduits are not intended to throttle flow, but would be either open or closed, except during flood retention and transitioning flow control to the evacuation conduit. Two of the gates will be capable of flow regulation for operational redundancy. The primary conduit is the main river channel through the dam to pass sediment. Over time, the primary conduit likely would be damaged on the invert due to the abrasive nature of by-passing sediment. The secondary fish passage conduits are for fish passage, with two of the gates able to produce releases at high pool elevations.

4.3 Evacuation Conduit

To simplify fish passage gate design and stilling basin size, a 9-foot-diameter evacuation conduit would be included for high head flow releases to provide large flow releases when the fish passage conduit gates are closed during a flood retention event. This also would allow regular flow releases for the FRE-FC condition with a permanent pool. The 9-foot-diameter conduit likely would contain a large and small diameter Howell Bunger valve for large and normal flow releases. The Howell Bunger valve would discharge into a baffle hood to dissipate energy and reduce exit velocities. Flow discharge would be released into the spillway stilling basin and then flow back into the river. Releases would be over a velocity barrier or vertical barrier to keep fish out.

4.4 Water Quality Port Outlets

Water quality port outlets are intended to withdraw water at different reservoir elevations to maintain a desired downstream water temperature. The conduits for the outlets would be constructed during the FRE but blind flanged for future FRE-FC condition engagement.

4.5 Flood Retention and Evacuation Transition

During a rainfall event, and as determined by the downstream grand mound gage at Chehalis, flood retention operation would begin when the fish passage conduit gates are closed, and incoming floodwaters are impounded behind the FRE.

Once flood operations are initiated, the fish passage conduit gates would close to reduce flows passing the FRE. As the water surface elevation upstream of the FRE rises, the conduit gates would close rapidly until the flows are approximately 3,000 cfs and before pool elevation 510 feet. To protect the gates, the primary and outer secondary fish passage conduit gates need to be closed before the pool elevation reaches 510 feet. The flow discharge could then be transferred to the evacuation conduit following an allowable ramp-down rate. Flood operation steps include:

- Step 1 – begin simultaneously closing the secondary conduits on the outside and the primary fish passage conduit gates to a gate opening of approximately 50 percent.
- Step 2 – close the secondary fish passage conduit gates on the outside, while maintaining the primary gate at 50 percent open.
- Step 3 – begin closing the primary fish passage gate and the two secondary fish passage conduits with the high head bonneted slide gates. To protect the gates from damage, it is required that the primary fish passage conduit gate and the two outer secondary fish passage conduit gates be closed before the pool elevation exceeds 510 feet.
- Step 4 – at pool elevation 510 feet, ensure that the primary fish passage gate is fully closed.
- Step 5 – begin alternating between closing the high head bonneted slide gates, and opening the reservoir evacuation conduit, while maintaining ramp-down rates. The secondary fish passage conduits with the high head bonneted slide gates are intended to be used for emergency releases at pools greater than elevation 530 feet. While high pool operation is possible, closing these gates at pool elevation 530 feet is recommended to protect the gate.
- Step 6 – after all fish passage conduit gates are closed, continue flow reduction using the reservoir evacuation conduit at ramp-down rates until a minimum outflow of 300 cfs is reached.

After the incoming flood has peaked and receded, an initial reservoir drawdown through the reservoir evacuation conduit would lower the storage pool to elevation 510 feet. At pool elevations below 510 feet the flow control would be transferred to the primary fish passage conduit gate and the reservoir evacuation conduit closed. The primary conduit gate is utilized first during the drawdown period to move accumulated bedload past the dam. Standard ramp-up rates would be followed using the primary conduit gate, until the gate is completely open. Depending on the pool level, the secondary conduits would be opened individually following ramp-up rates until all gates are open and run-of-river operation resumes.

5.0 Fish Passage Conduits, Hydraulic Design

The hydraulic design of the fish passage conduits focused on the size, invert elevation, and profile of the conduits. The fish passage conduits were sized to pass the 95 percent and 5 percent exceedance probability during fish migration periods, that were scaled for climate change, to satisfy flow depths and velocities when compared to a downstream river control section, or reference reach. The stilling basin was sized to function at the 100-year Annual Exceedance Probability (AEP) flow event with all gates fully open, and during emergency reservoir drawdown with the two high-head bonneted slide gates operating.

5.1 Climate Change

The design flows for the fish passage conduits are determined using NOAA Fisheries West Coast Region Guidance to Improve the Resilience of Fish Passage Facilities to Climate Change (NOAA 2022b). The fish passage conduits have a life expectancy greater than 10 years so determination of the fish passage design flows for the fish passage conduits must follow the process for long-term projects defined in Section 2.3 of the guidance. This process is underway and collaboration with NOAA Fisheries is ongoing at the time of publication of this document. Because development of fish passage design flows incorporating climate change following NOAA Fisheries guidance is not complete, interim fish passage design flows incorporating climate change have been adopted for use in the design documented herein.

Climate change information is incorporated into the fish passage design flows using peak flow scalars that were derived from the 12 global climate models produced by WDOE's consultants for the SEPA EIS (WSE 2023). The late-century ensemble average maximum scalar (+55 percent) is applied to the historic high fish passage flow. The historic high fish passage flow is 2,200 cfs, corresponding to 5 percent exceedance (HDR 2017a). The mid-century average minimum scalar (-14 percent) is applied to the historic low fish passage flow. The historic low fish passage flow is 16 cfs, corresponding to the 95 percent exceedance (HDR 2017a). The high and low fish passage design flows used in the fish passage conduit design documented herein are 3,400 cfs and 14 cfs, respectively. These climate change scalars are conservative. This approach to approximating fish passage design flows incorporating climate change conditions is conservative and consistent with a conceptual level of design development.

5.2 Flow Capacity

5.2.1 Gates Full Open

The flow capacity of the five fish passage conduits was evaluated assuming that each gate was fully open. For return intervals of 2-year (AEP) and more frequent, the fish passage conduits operate in an open channel condition. For return intervals of 5-year (AEP) and less frequent, the fish passage conduits are submerged due to the tailwater elevation. When the fish passage conduits are flowing full, some simplifying assumptions were made on the secondary conduits and approximating losses in the converging section, by using an average width for the section. Table 2 shows the flow capacity of the fish passage conduits with the gates fully open.

Table 2. Flow Capacity of Fish Passage Conduits with Gates Full Open

AEP / Return Interval	River Flow (cfs)	Headwater Elevation (ft, NAVD88)	Primary Conduit Flow (cfs)	Primary Conduit Velocity (ft/s)	Secondary Conduit Flow (cfs)	Secondary Conduit Velocity (ft/s)	Tailwater Elevation (ft)
99% / 1-yr	2,580	441.2	748	4.4	458	4.1	437.6
50% / 2-yr	9,496	447.3	2,725	11.4	1,693	10.6	444.5
20% / 5-yr	15,531	452.7	3,496	14.6	3,009	18.8	448.9
10% / 10-yr	20,175	458.2	4,541	18.9	3,908	24.4	451.8
6.7% / 15-yr	23,011	461.7	5,180	21.6	4,458	27.9	453.4
5% / 20-yr	25,098	464.4	5,649	23.5	4,862	30.4	454.5
4% / 25-yr	26,756	466.7	6,023	25.1	5,183	32.4	455.4
2% / 50-yr	32,169	474.4	7,241	30.2	6,232	38.9	458.1
1% / 100-yr	38,014	483.5	8,557	35.7	7,364	46.0	460.9
0.4% / 250-yr	46,478	498.7	10,462	43.6	9,004	56.3	464.8
0.2% / 500-yr	53,491	512.8	12,041	50.2	10,363	64.8	467.9

*Vertical Datum is NAVD88. ft/s = feet per second.

5.2.2 Gate Control Capacity

The flow capacity of the primary and secondary fish passage conduit was assessed assuming the gates are partially open and controlling the flow. This information is valuable to determine how the flow will transition from full open, fish passage, to flood retention and transitioning flow control to the reservoir evacuation conduit. The analysis uses the orifice equation with the opening being defined by the gate opening for one conduit. If more than one secondary conduit is in operation, the flows should be assumed to be accumulative based on the number of gates in operation. Figure 3 and Figure 4 show the flow capacity for one gate in operation for the primary and secondary fish passage conduit respectively.

Figure 3. Primary Conduit Gate Control Flow Capacity

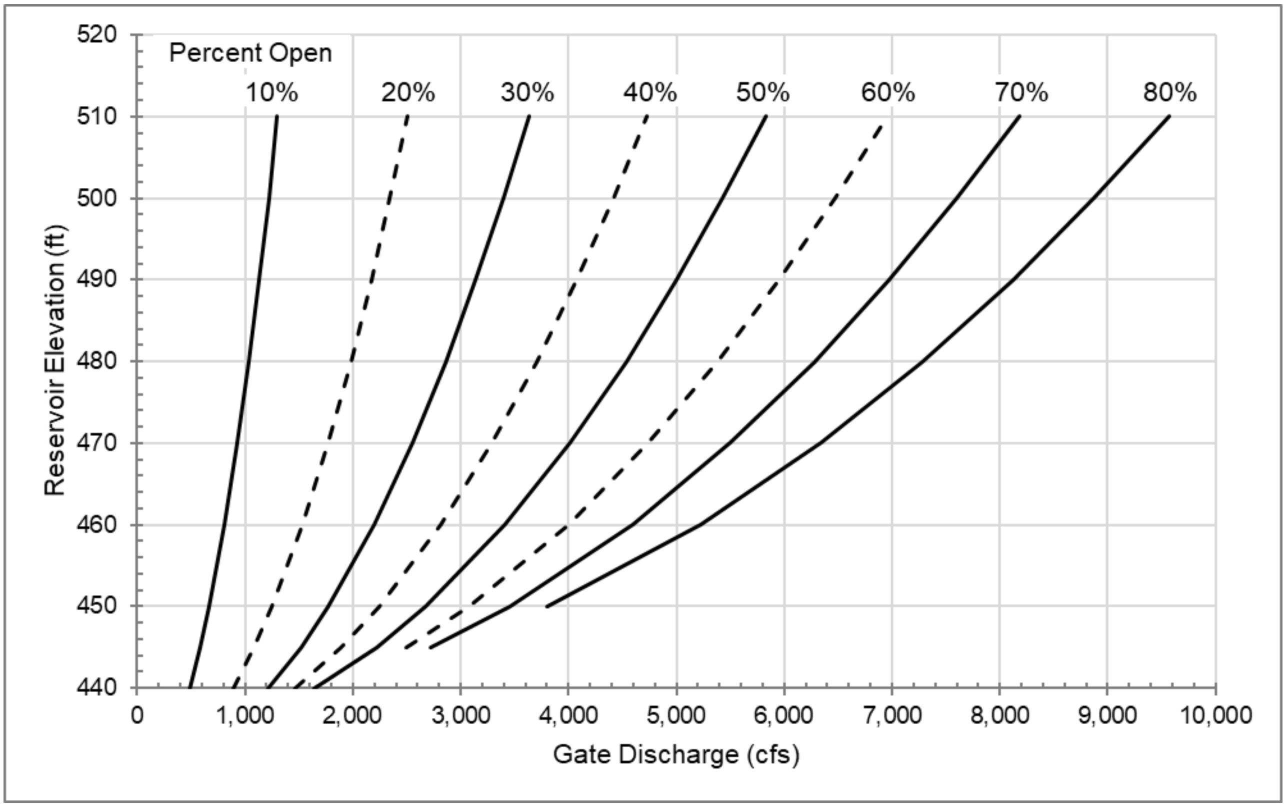
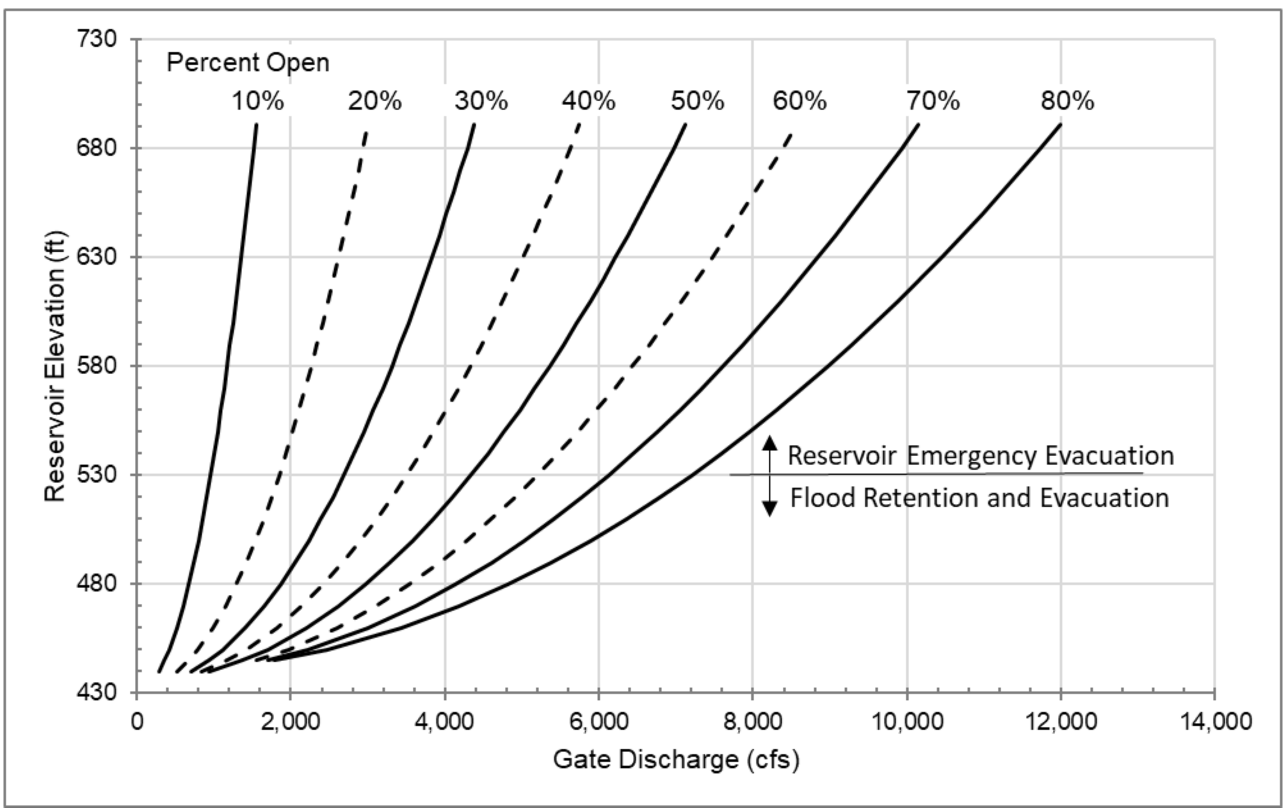


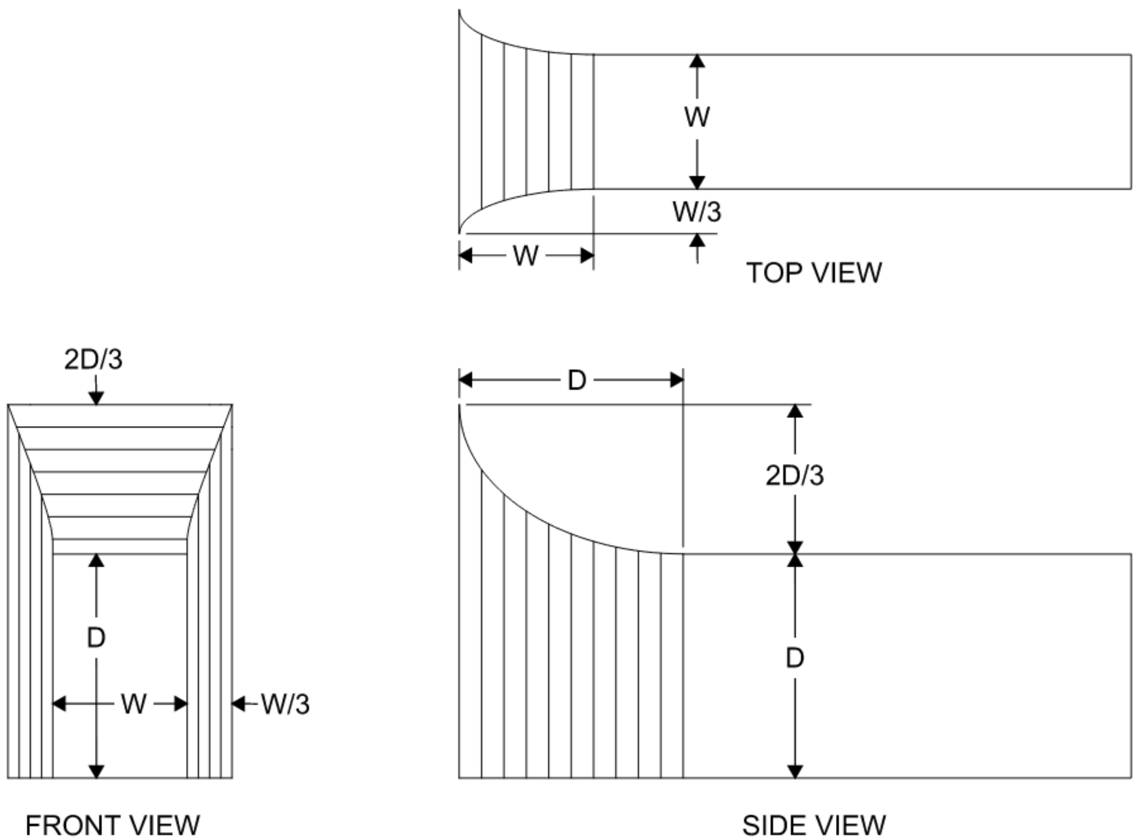
Figure 4. Secondary Conduit Gate Control Flow Capacity, One Gate in Operation



5.3 Entrance Curves

To avoid flow separation and unsatisfactory pressure conditions, a roof curve and sidewall curves were added to the fish passage conduit entrances. The roof and sidewall curves are based on an elliptical curve. The roof entrance curve is based on an entrance with an invert that is level with the approach channel floor (Plate C-37, Engineering Manual [EM] 1110-2-1602; U.S. Army Corps of Engineers 1980), where the major and minor radius of the ellipse is based on the conduit height and two-thirds of the conduit height respectively. The sidewall entrance curve is based on a sluice entrance (Plate C-22, EM 1110-2-1602), where the major and minor radius of the ellipse is based on the conduit width and one-third of the conduit width respectively. Figure 5 shows the profile view of the elliptical roof and sidewall entrance curves based on the fish passage conduit height and width.

Figure 5. Profile Views of Roof and Sidewall Entrance Curves

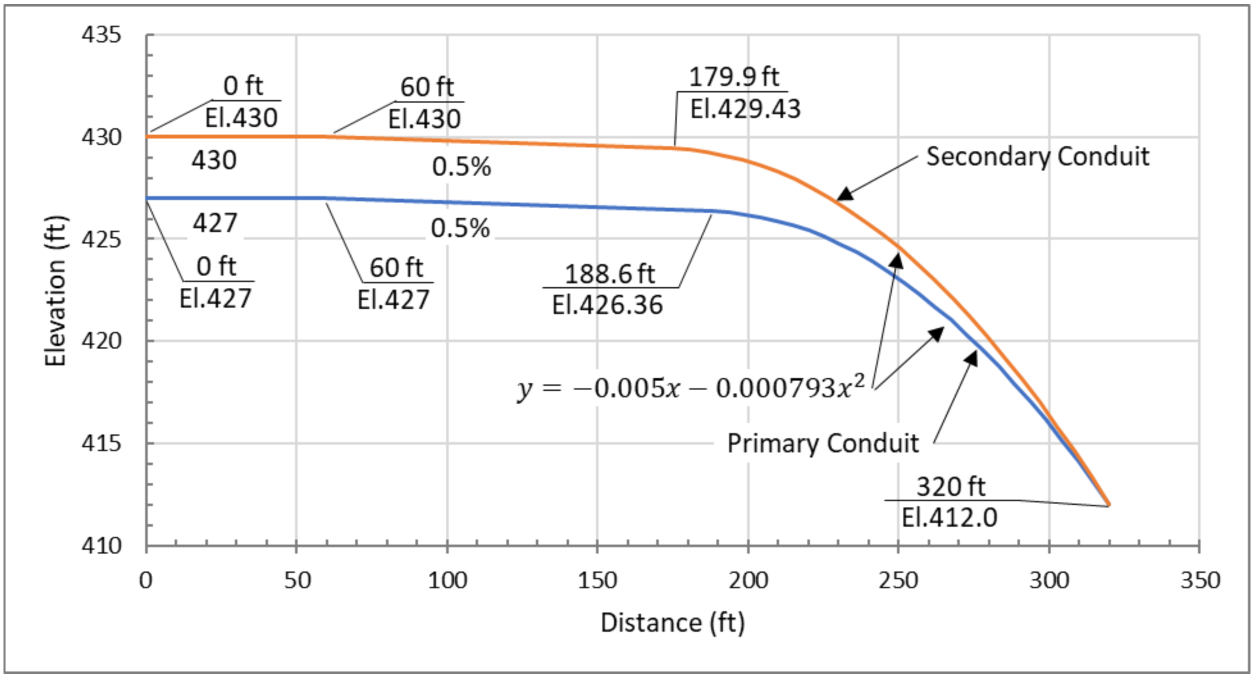


5.4 Fish Passage Conduit Profile

The fish passage conduit profile is based on three sections, the horizontal section through the gates, followed by a constant slope at 0.5 percent, and a parabolic drop into the stilling basin elevation. The parabolic drop is based on the trajectory of a jet under the action of gravity. The estimated velocity, including a 25 percent safety factor, assuming gate control at the FRE-FC spillway crest was 142 ft/s. This ensures positive pressures on the invert during all flow events. The following equation was used for the parabolic drop, where x and y are referenced from the beginning of the curve. Figure 6 shows the profile for the primary and secondary fish passage conduits.

$$y = -0.005x - 0.000793x^2$$

Figure 6. Profile for Primary and Secondary Fish Passage Conduits



5.5 Water Surface Profile

A basic water surface profile was developed, assuming that all gates were fully open, through the primary and secondary fish passage conduits to inform how the conduits were performing. Information from the water surface profile was also used to size the stilling basin. Figure 7 and Figure 8 show the water surface profiles for the primary and secondary fish passage conduits respectively.

Figure 7. Primary Fish Passage Conduit Water Surface Profile

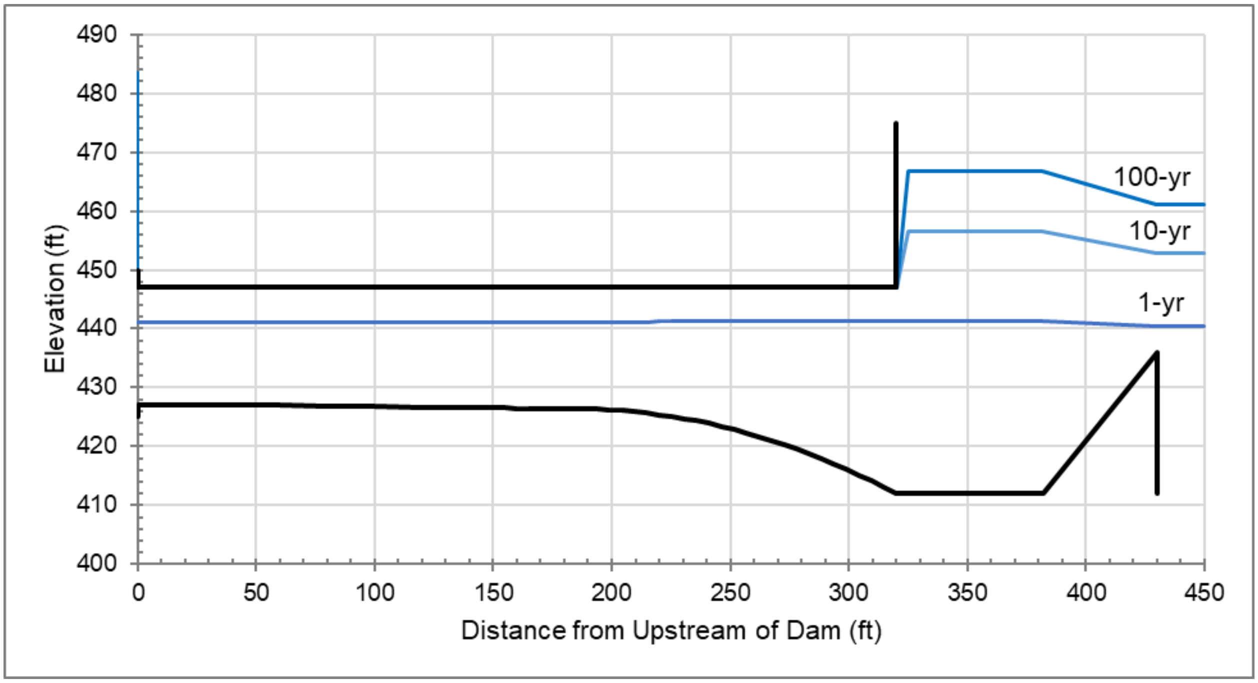
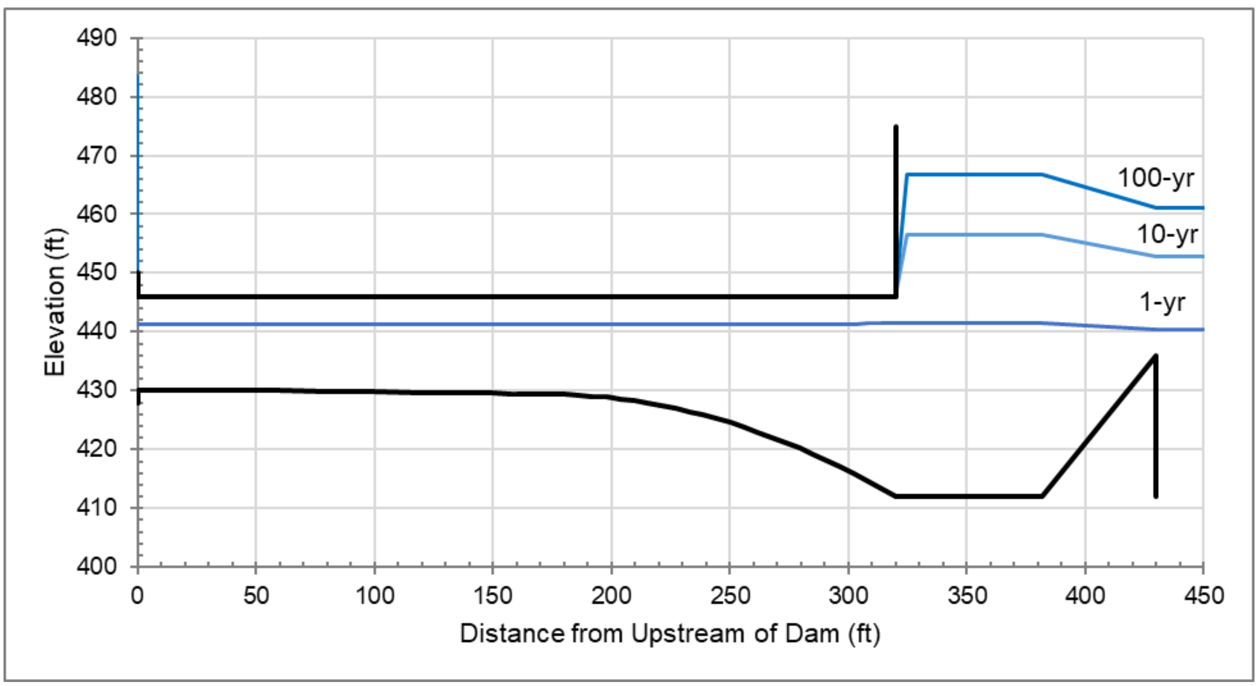


Figure 8. Secondary Fish Passage Conduit Water Surface Profile



5.6 Gate Flow Water Surface Profile

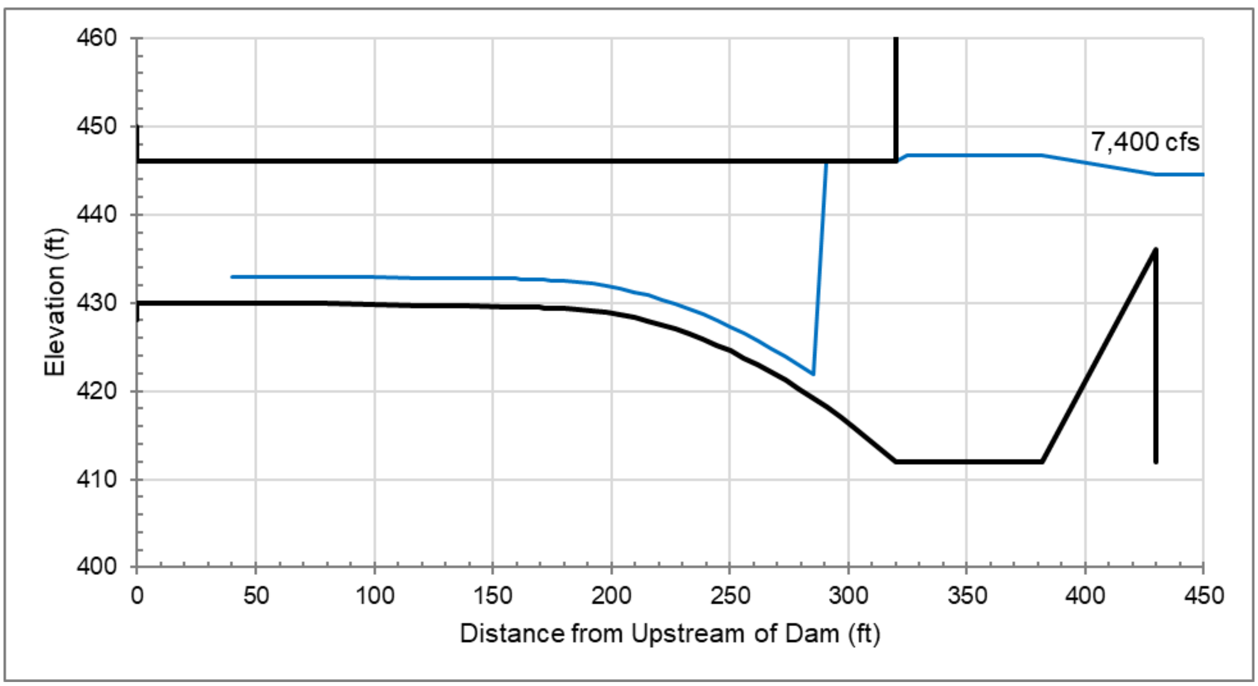
A basic water surface profile was developed for when the high head bonneted slide gates control the flow. During an emergency reservoir evacuation and assuming a reservoir elevation at the FRE-FC spillway crest, the two gates would be 24 percent open, discharging 7,400 cfs. The discharging flow under the gate has a high Froude number and the water surface profile was developed to inform how the conduits were performing and determine the flow depth and velocity entering the stilling basin. Figure 9 shows the water surface profile for the secondary fish passage conduits with the high head bonneted slide gates in operation.

5.7 Conduit Stilling Basin

The conduit stilling basin was sized to contain the discharge with the greatest unit energy, which is the condition where the two secondary fish passage conduits with the high head bonneted gates controlled the flow. The stilling basin is sized to pass 7,400 cfs (emergency reservoir evacuation rate) utilizing the two secondary fish passage conduit gates with the pool elevation at the FRE-FC spillway crest. This produces a minimal gate opening of approximately 24 percent with high velocities. The velocities controlled the shape of the fish passage conduit and the conduit stilling basin design. To reduce the basin length, a type III basin with baffle blocks was selected. For normal fish passage operation, when the gates are fully open, the basin will function satisfactorily without the baffle blocks. Therefore, the baffle blocks could be installed during the future FRE-FC condition.

The conduit stilling basin endsill elevation is set for fish passage and much larger than normally recommended for a type III basin but could cause rapid sedimentation of the conduit stilling basin. The basin is anticipated to function normally with the larger endsill, but additional future studies will evaluate the performance of the stilling basin and its operational limitations.

Figure 9. Water Surface Profile, Emergency Reservoir Evacuation



5.8 Sediment Mobilization

The fish passage conduits are designed to pass river sediments through the dam during run of river operations. The goal is to pass the sediment though the primary conduit, which has a lower invert elevation. As a result, the stilling basin will begin to collect sediment until flows are sufficient to flush out the sediment. As a preliminary analysis, the sediment incipient motion was evaluated to determine which sediment sizes can be expected to mobilize. The assumption is if the flow can mobilize a sediment particle at the invert of the stilling basin, then it could be carried out of the basin as the velocities and shear stresses increase with proximity to the endsill. Table 3 shows the flow, velocity, shear stress, and estimated sediment size that can be moved out of the stilling basin.

Table 3. Estimated Sediment Size Mobilization

AEP / Return Interval	Flow (cfs)	Stilling Basin Velocity (ft/s)	Mobilized Grain Size (mm)	Shear Stress (lb/ft^2)	Sediment Classification
99% / 1-yr	2,580	1.1	2.2	0.03	Fine Sand
Gate Control	7,400	2.6	2.2	0.03	Coarse Sand
50% / 2-yr	9,496	3.2	3.0	0.04	Very Fine Gravel
20% / 5-yr	15,531	4.6	7.8	0.11	Fine Gravel
10% / 10-yr	20,175	5.6	12.7	0.17	Medium Gravel
6.7% / 15-yr	23,011	6.1	16.3	0.22	Coarse Gravel
5% / 20-yr	25,098	6.5	19.1	0.26	Coarse Gravel
2% / 50-yr	32,169	7.7	30.1	0.41	Coarse Gravel
1% / 100-yr	38,014	8.6	40.8	0.55	Very Coarse Gravel

*cfs = cubic feet per second. ft/s = feet per second. mm = millimeter. lb/ft^2 = pounds per square foot.

5.9 Fish Passage

Additional information regarding target species and fish passage requirements can be found in Section 11 of the Revised Project Description report.

5.9.1 Methodology

To analyze the range of hydraulic conditions present at the Project location, a two-dimensional (2D) hydraulic model was employed. The hydraulic analysis was performed utilizing HEC-RAS, Version 6.3 (RAS) hydraulic modeling software. RAS is applicable for flows in surface water bodies where vertical velocities and accelerations are small or relatively negligible in comparison to those in horizontal directions. It can simulate subcritical flow, supercritical flow, and the transition between the two.

The 2D model was used to compare hydraulic conditions within existing reference reaches to hydraulic conditions within the proposed fish passage conduits, stilling basin, and constructed river channels. The primary criterion for this evaluation is that the proposed flow velocity and depth through the structures mimic the flow velocity and depth occurring naturally through the reference reaches. This methodology is derived from the Washington Department of Fish and Wildlife's (WDFW) stream simulation design approach, which assumes that fish are present in the natural channel are not expected to be challenged by the stream simulation channel that looks and performs similarly to adjacent natural channels (WDFW, 2013). Details on the techniques used to build the 2D RAS model are presented in the following subsections.

5.9.2 Model Extent

The limits of the 2D hydraulic model begin approximately 1,500 feet upstream of the confluence on both Crim Creek and the Chehalis River and terminate approximately 3,800 feet downstream of the confluence. The boundaries of the model encompass the entire floodplain of the existing river, the FRE, and all reference reaches. The model extents were conservatively extended beyond the FRE location such that the influences of the boundary conditions were negligible.

5.9.3 Surface Roughness

Surface roughness (Manning's) in the model was assigned to corresponding land surface characteristics approximated from aerial photographs and site visit photos from September 2022. In the modeling process, surface roughness was adjusted within standard tolerance limits (U.S. Geological Survey 1989; Yochum et al. 2014). A summary of Manning's roughness coefficients as they relate to land surface designations is provided in Table 4.

Table 4. Summary of Modeled Manning's Roughness Coefficients

Land Cover	Manning's Value
Forested Overbank	0.1
Gravel Road	0.03
Exposed Bedrock	0.025
Existing Chehalis River Channel[a]	0.032 – 0.065
Existing Crim Creek Channel	0.045
Rip Rap	0.07
Constructed Channels	0.05
Conduit (Smooth Concrete)	0.013

[a] Chehalis River channel composition varies significantly, from primarily small gravel and cobbles to very large boulders.

5.9.4 Inflow Boundary Conditions

The model was developed to evaluate the flow depths and velocities within the conduits at the lowest 95 percent (16 cfs) and highest 5 percent (2,200 cfs) exceedance probability during target species fish migration periods at the FRE location. These exceedance flows are based on the best available, original (2018) hydrology estimates (HDR 2023), and do not include impacts due to climate change. As stated in Section 5.1, climate change modeling is ongoing and final climate change scalers are not expected until 2024. At the request of the National Marine Fisheries Service (2022a), the most recent available climate change scalers were applied at this time.

Flows at the FRE location were split between Crim Creek and the Chehalis River based on their respective drainage basins upstream of the confluence. Table 5 presents the design discharges selected and Table 6 the scaled design discharges including climate change. With climate change, the low fish passage design flow decreases to 14 cfs, while the high fish passage design flow increases to 3,400 cfs. The modeling discharge was held constant throughout each simulation (i.e., steady state).

Table 5. Modeled Inflow Boundary Conditions

Boundary Condition	Drainage Area (sq mi) / % of total	Low Fish Passage Design Flow (cfs)	High Fish Passage Design Flow (cfs)
Chehalis River	56.5 / 82%	13	1,804
Crim Creek	12.4 / 18%	3	396

Table 6. Modeled Inflow Boundary for Climate Change

Boundary Condition	Low Fish Passage Design Flow (cfs)[a]	High Fish Passage Design Flow (cfs)[b]
Chehalis River	11	2,788
Crim Creek	3	612

[a] The low fish passage design flow (Table 4) is applied the mid-century ensemble average minimum scalar (-14%).
[b] The high fish passage design flow (Table 4) is applied the late-century ensemble average maximum scalar (+55%).

5.9.5 Outflow Boundary Conditions

The 2D model was simulated using the normal depth boundary condition at the sole outflow location. The energy slope assigned to calculate the normal depth water surface elevation was assumed to match the channel slope at the corresponding location of the digital terrain model. Measurement tools in HEC-RAS were used to calculate an approximate channel slope of 0.4 percent at the outflow location.

5.9.6 Existing Conditions

5.9.6.1 Topographic Data

Topographic data currently being used for project components is Light Detection and Ranging (LiDAR) survey data downloaded from the Washington State Department of Natural Resources' online LiDAR portal. LiDAR data available at the FRE location was flown in 2019 and the corresponding digital terrain model has a 3-foot resolution (Quantam Spatial 2019). Table 7 provides the coordinate system in which LiDAR data collected.

Table 7. LiDAR Coordinate System

Coordinate System Component	Description	Units
Projection	WA State Plane South	n/a
Horizontal Datum	North American Datum of 1983	US Survey Feet
Vertical Datum	North American Vertical Datum of 1988	US Survey Feet

No bathymetric data is currently available for the Project location. Future studies will incorporate bathymetric data as it becomes available.

5.9.6.2 Reference Reach

Reference reaches that are naturally occurring in the river channel were selected to closely represent a selected design feature. The intent of the reference reaches is to verify design features have the same or better fish passage capabilities than the existing river channel. The selected reference reaches are identified in Figure 10 and described below:

- **Conduit Reference Reach:** This reach is an incised rock channel 280 feet long and about 30 feet wide. During 2022 discussions with the Fish Passage Technical Subcommittee, it was agreed that this would be a natural channel with comparable characteristics to the concrete fish passage conduits.
- **Chehalis River Channel Reference Reach:** This reach is approximately 520 feet long and located downstream of the conduit reference reach, where the Chehalis River widens and the gradient increases. It is comprised of large cobbles and boulders with an approximate slope of 2.4 percent.
- **Crim Creek Channel Reference Reach:** This reach is located approximately 200 feet upstream of the confluence with the Chehalis River. It is comprised of small cobbles and some small boulders with an approximate slope of 1.3 percent.

5.9.7 Proposed Conditions

The proposed terrain was developed using existing LiDAR topography as a baseline, modified using terrain modification tools in RAS Mapper. The proposed Chehalis River and Crim Creek channels were modeled based on the typical cross sections illustrated in Figure 11. These channel cross sections and slopes are conceptual based on the Washington Department of Fish and Wildlife's design approach to mimic observed geomorphology. Bankfull widths, visually approximated bathymetry, and reference reaches for the Chehalis River and Crim Creek channels were obtained during September 2022 field work, and cross sections and slopes were obtained from LiDAR. Refinement of the permanent approach and discharge channels will occur during preliminary design, after bathymetry data has been incorporated. Table 8 compares the proposed slopes to the reference reach slopes.

The conduits were added per the conduit layout described in Section 3.0. For simplicity, the conduits were modeled as open channels with vertical side walls high enough to contain the modeled flow. Under the high fish passage design flow, all fish passage conduits are open and under the low fish passage design flow only the primary fish passage conduit is open.

Table 8. Summary of Proposed Slopes

Proposed Channel	Proposed Slope (%)	Reference Reach Slope (%)
Crim Creek Channel	2.4	1.3
Chehalis River Channel – Upstream	1.9	2.4
Chehalis River Channel - Downstream	1.6	2.4

Figure 10. Locations of Reference Reaches

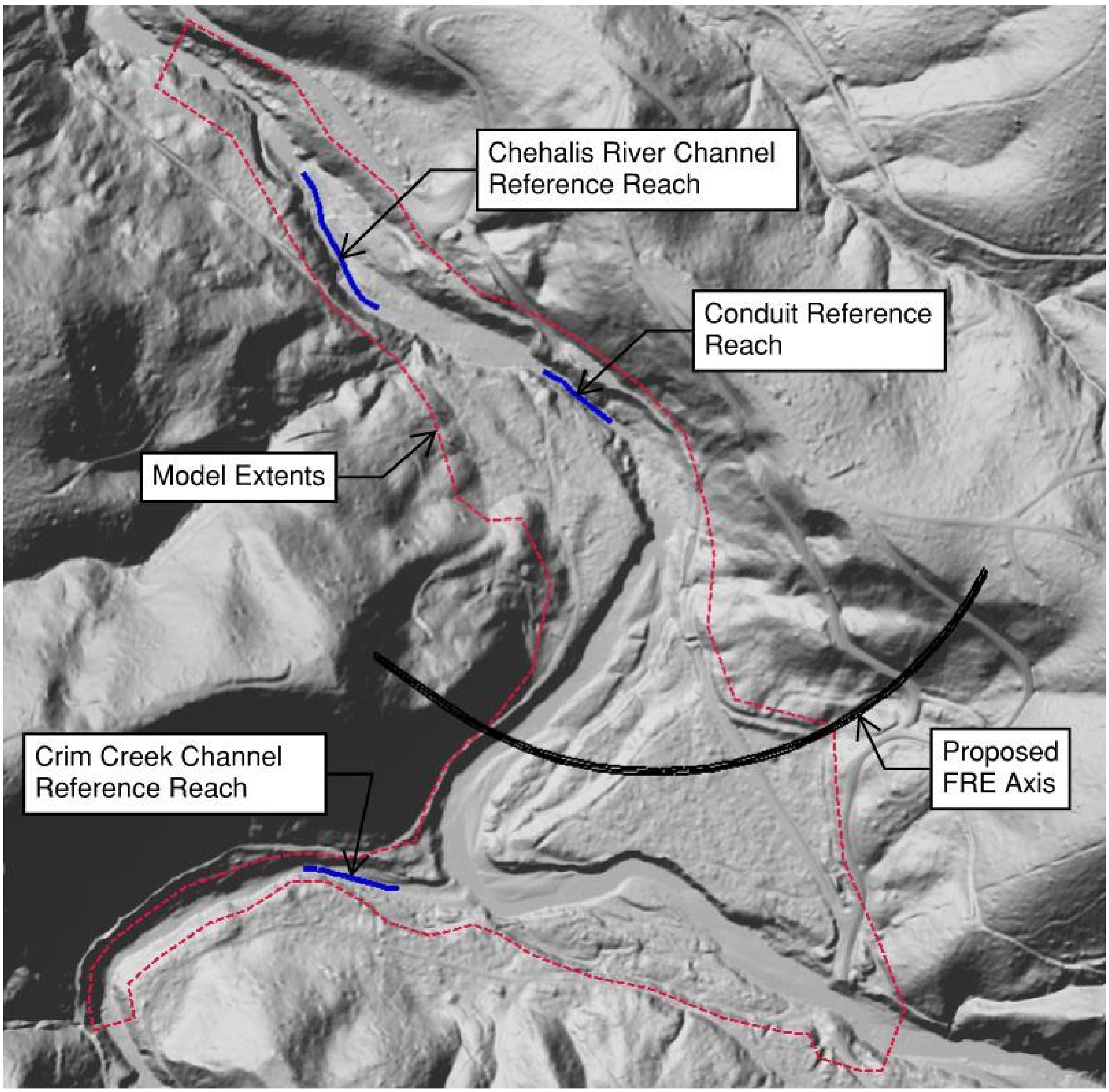
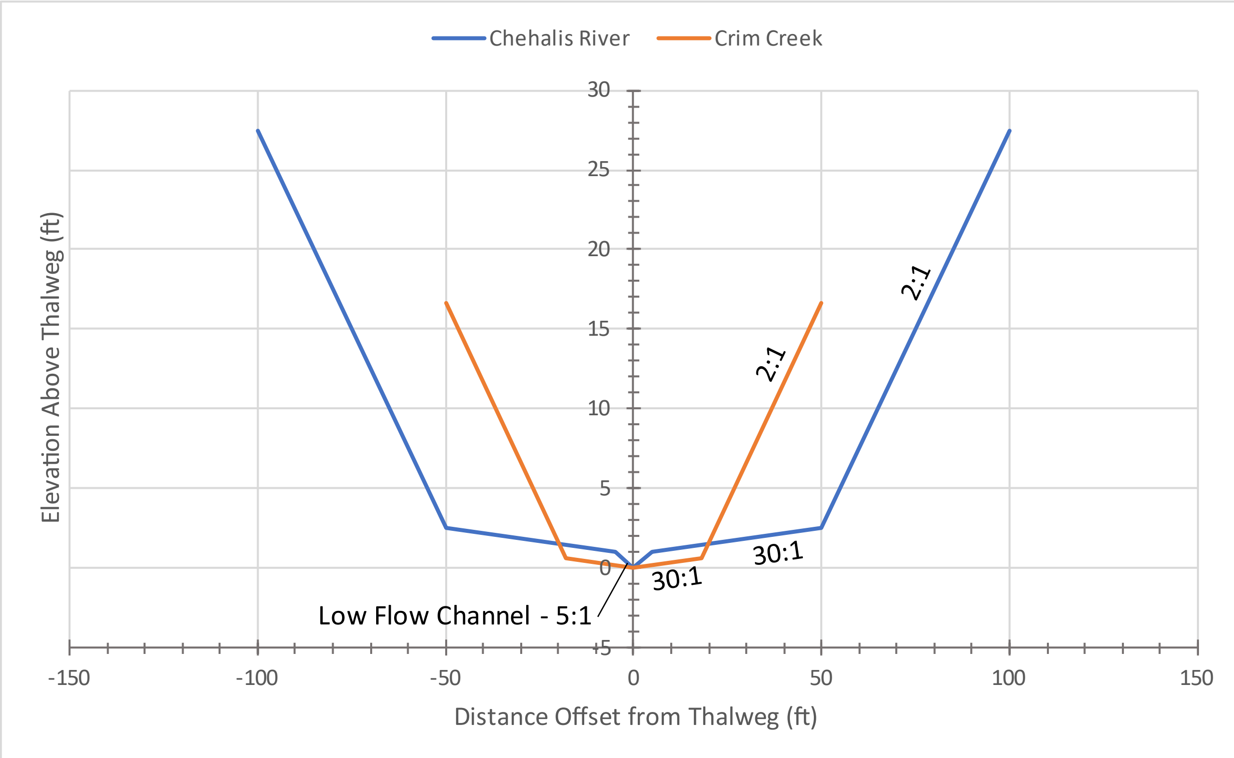


Figure 11. Typical Constructed Channel Cross Sections



5.9.8 2D HEC-RAS Model Results

Detailed hydraulic modeling results are presented in Attachment 1. Existing and proposed conditions were evaluated at the high and low fish passage design flows, both with and without climate change scalers.

5.9.8.1 High Fish Passage Design Flow

Velocities in the proposed conduits and channels do not exceed naturally occurring velocities in the Chehalis River and Crim Creek reference reaches (Section 5.9.6.2). The fish passage conduits experience maximum velocities of 4 to 5 ft/s. The bend in the Chehalis River channel upstream of the FRE appears to be causing a low-velocity pocket/eddy to form on the right bank of the approach channel and left side of the stilling basin. This phenomenon should be further investigated during preliminary design.

Depths in the proposed conduits and channels are greater than the naturally occurring depths in the Chehalis River and Crim Creek reference reaches. The conduit stilling basin endsill causes much deeper water depths through the conduits, and for a distance upstream of the FRE, than is modeled through the reference reaches.

Overall, the proposed conditions do not negatively affect fish passage conditions at the high fish passage design flow.

When climate change scalers are included, the comparison between existing and proposed conditions remains relatively constant (i.e., the proposed conditions fall within the range of conditions modeled within the existing reference reaches). Compared to the unscaled results, the depths and velocities increase. Fish passage conduit velocities also increase from approximately 4 to 5 ft/s (unscaled) to 5 to 7 ft/s (scaled).

5.9.8.2 Low Fish Passage Design Flow

Velocities in the proposed conduits and channels do not exceed naturally occurring velocities in the Chehalis River and Crim Creek reference reaches (Section 5.9.6.2). Velocities in the conduits and stilling basin are much lower than existing conditions due to backwater from the stilling basin endsill but are about equal within the approach and discharge channels.

Depths in the proposed conduits and channels are generally equal to or slightly greater than the naturally occurring depths in the Chehalis River and Crim Creek reference reaches. The stilling basin endsill causes much deeper water depth through the conduits, and for a distance upstream of the FRE, than is modeled through the reference reaches. Because depths in the proposed discharge channel are shallowest just downstream of the conduit stilling basin endsill at approximately 0.5 feet, care should be taken to ensure the low-flow channel is maintained.

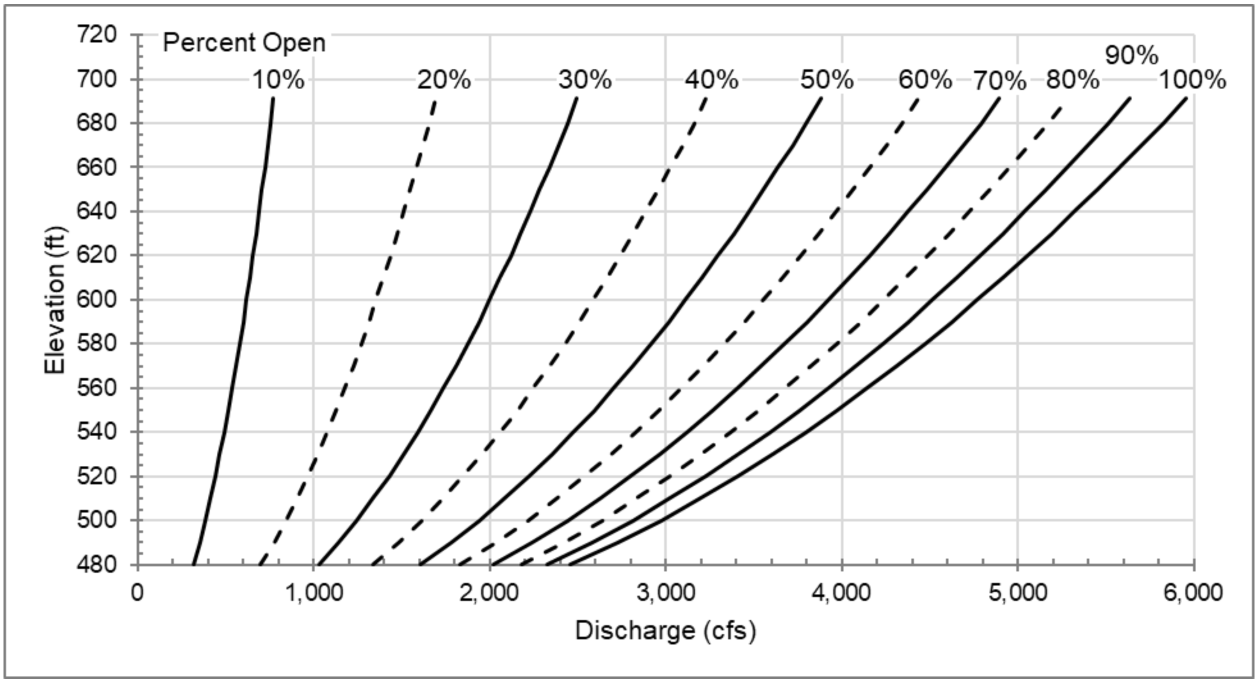
Overall, the proposed conditions do not negatively affect fish passage conditions at the low fish passage design flow compared to existing conditions.

When climate change scalers are included, the comparison between existing and proposed conditions remains relatively constant. Compared to the unscaled results, the depths and velocities slightly decrease, reiterating the importance of a functioning low-flow channel in the constructed channels.

6.0 Evacuation Conduit, Hydraulic Analysis

A conceptual evaluation of the reservoir evacuation conduit was conducted. The evaluation included determining the flow capacity of the Howell Bunger valve (HBV) at different valve openings and reservoir elevations to determine when the valve could be safely operated. A conceptual baffle hood size was assessed. This analysis is limited as it only considered one valve size, but the conceptual results can be used to inform potential reservoir operations and future modifications to the design. Figure 12 shows the flow capacity of a 9-foot-diameter HBV with a baffled hood diameter and length of approximately 22.5 feet and 28 feet, respectively.

Figure 12. Flow Capacity of 9-foot-diameter HBV, Evacuation Conduit



7.0 Recommendations for Conceptual Design

7.1 Discussion

The fish passage conduits are primarily intended to function with the gates open for fish passage. The conduits will need to be used during transitional phases between fish passage and flood retention when the gates are used to control the flow and eventually closed for flood retention. After the fish passage gates are closed, the evacuation conduit is used for reservoir releases. Only two of the five fish passage conduits are intended to be used to regulate emergency flood releases or reservoir drawdown flows for the FRE or FRE-FC. When the fish passage gates are closed, all normal flow releases are made using the evacuation conduit, or water quality ports for the FRE-FC.

The full open flow capacity of the fish passage conduits was evaluated. The conduits size, entrance shape, and invert elevation profile were defined. The conduit velocities controlled the invert elevation profile and stilling basin design. The water surface profiles for several different return flows were evaluated, including gate control flows, to properly size the stilling basin. The elevation of the stilling basin endsill is set for fish passage and much larger than the size recommended. Baffle blocks are only required in the stilling basin during flows where the gates are partially opened, otherwise the stilling basin will function without baffle blocks while the gates are fully open. Additional future studies are needed to evaluate the performance of the stilling basin and its operational limitations.

A 2D HEC-RAS model was used to compare the flow depths and velocities in the fish passage conduits which was compared to the river control section located downstream of the FRE-FC structure.

A conceptual valve size was assessed for the evacuation conduit. Additional HBV sizes will be considered for high and normal flow releases for better flow control during flood retention for the FRE and FRE-FC.

7.2 Recommendation for Preliminary Design

Additional detail during preliminary design will continue to evaluate how the transition from fish passage to flood retention occurs and when the pool is being released.

During preliminary design, the expected thalweg and sediment transport location will be further evaluated and additional modification to the upstream approach channel considered. Additional details also need to be considered for the fish passage conduits and to evaluate conduit stilling basin performance during partial gate opening for the FRE-FC.

8.0 References

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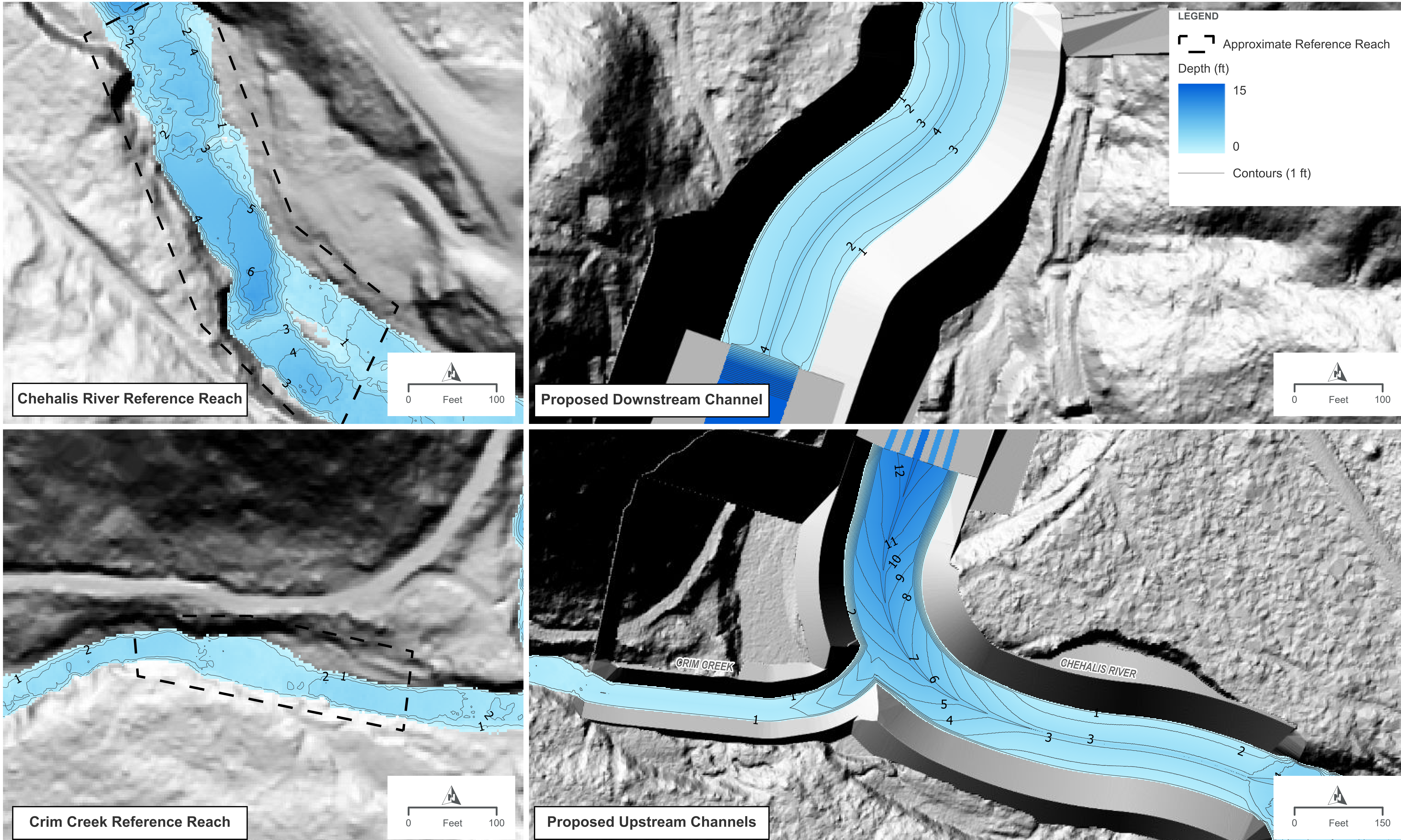
9.0 Abbreviations

1D	one-dimensional
2D	two-dimensional
AEP	Annual Exceedance Probability
cfs	cubic feet per second
District	Chehalis Basin Flood Control Zone District
EM	Engineering Manual
FRE	Flood Retention Expandable
FRE-FC	Flood Retention Expandable, Future Construction

ft	feet
ft/s	feet per second
HBV	Howell Bunger valve
LiDAR	Light Detection and Ranging
NAVD88	North American Vertical Datum of 1988

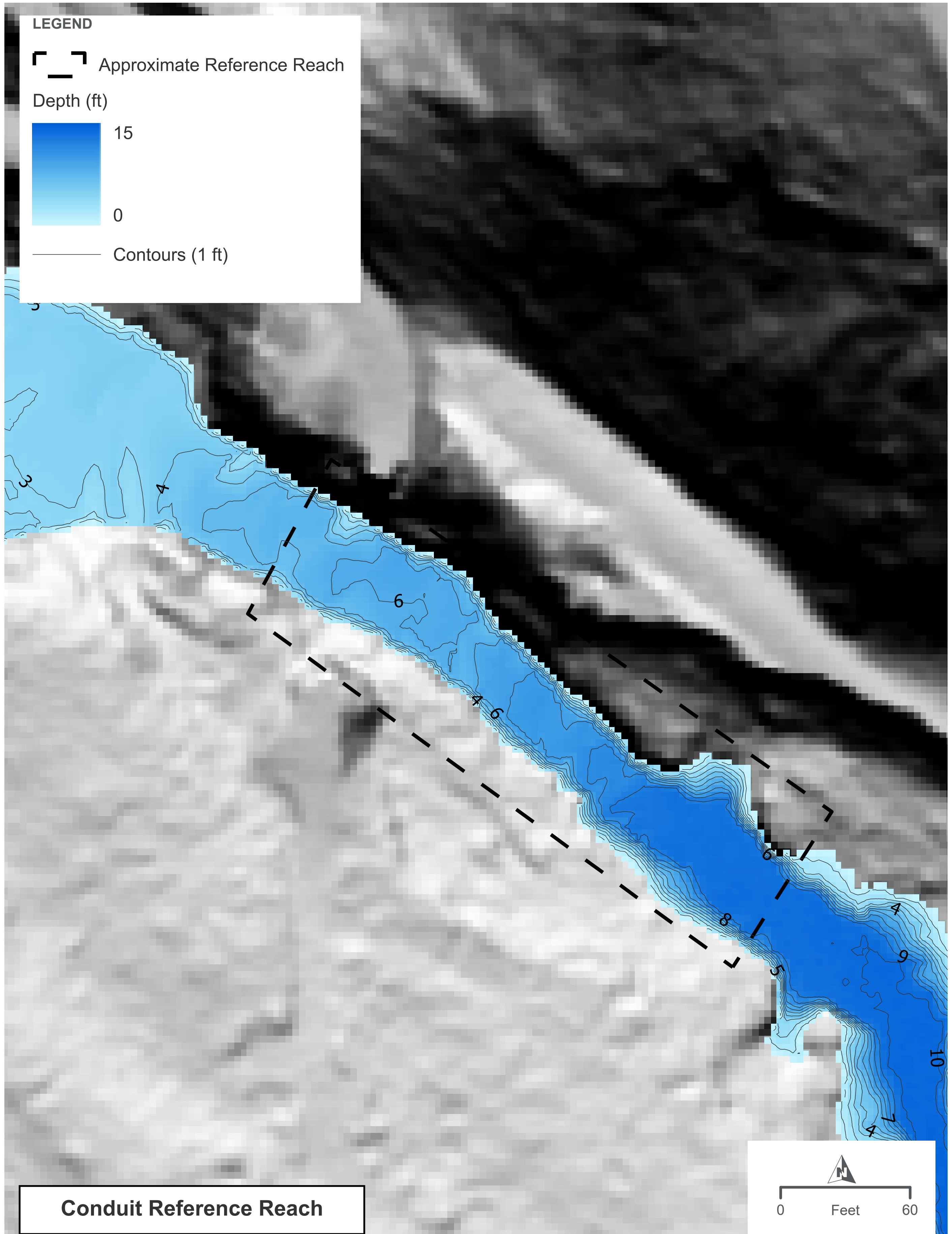
Attachment 1. 2D HEC-RAS Modeling Results

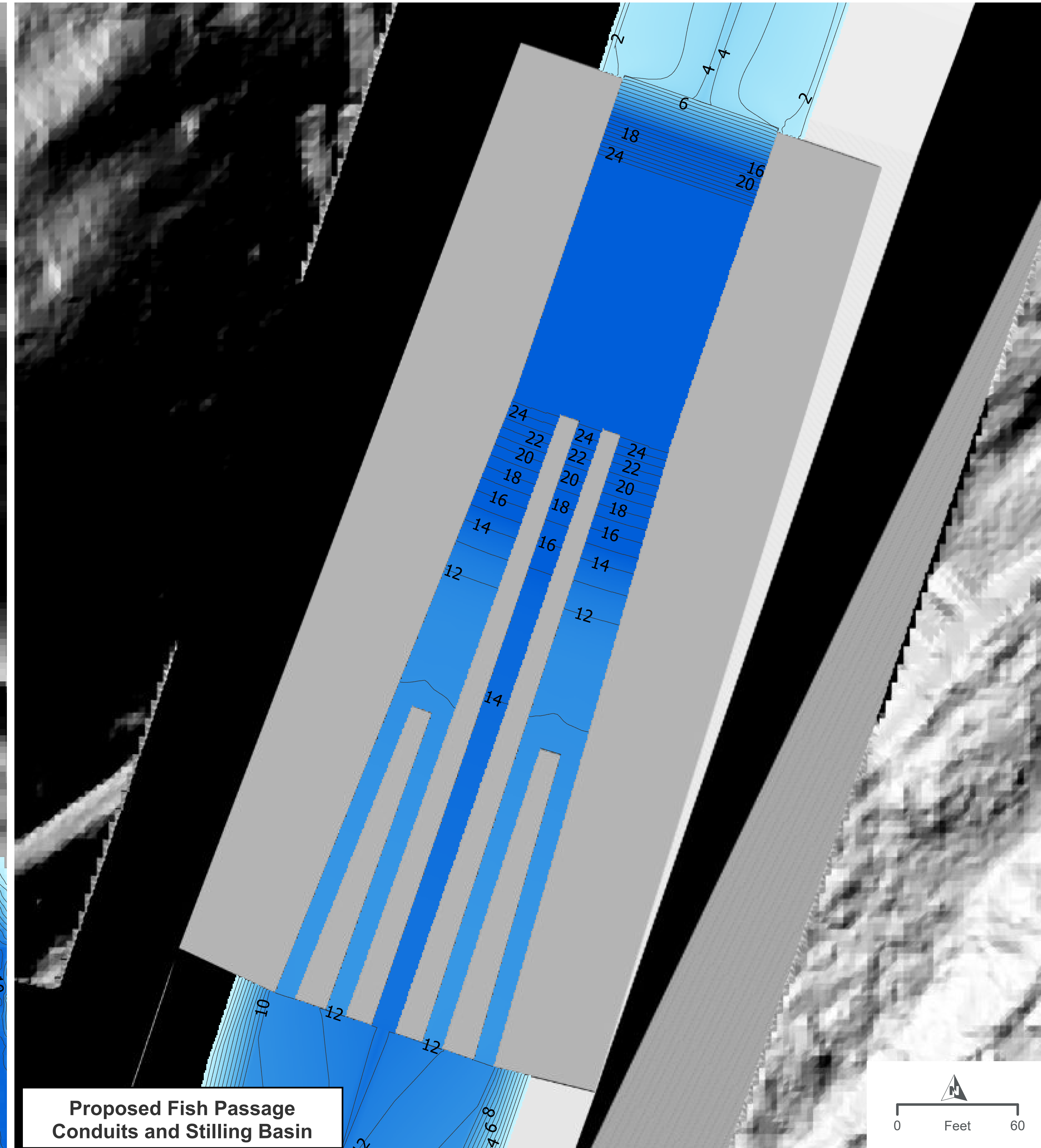


EXISTING VS PROPOSED (CHANNELS) - FISH PASSAGE HIGH DESIGN FLOW

DEPTH

FIGURE 1

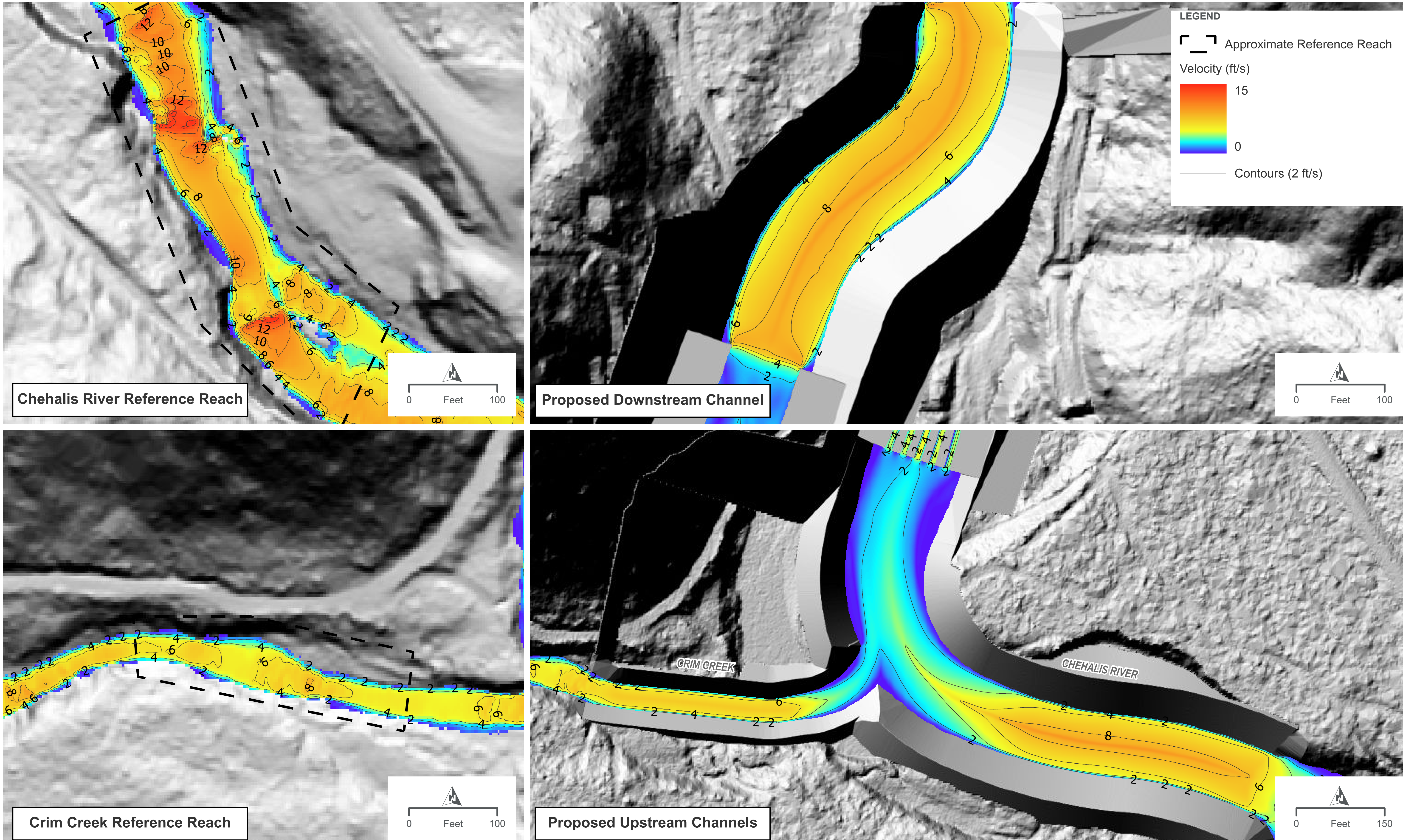




EXISTING VS PROPOSED (FISH PASSAGE CONDUITS) - FISH PASSAGE HIGH DESIGN FLOW

DEPTH

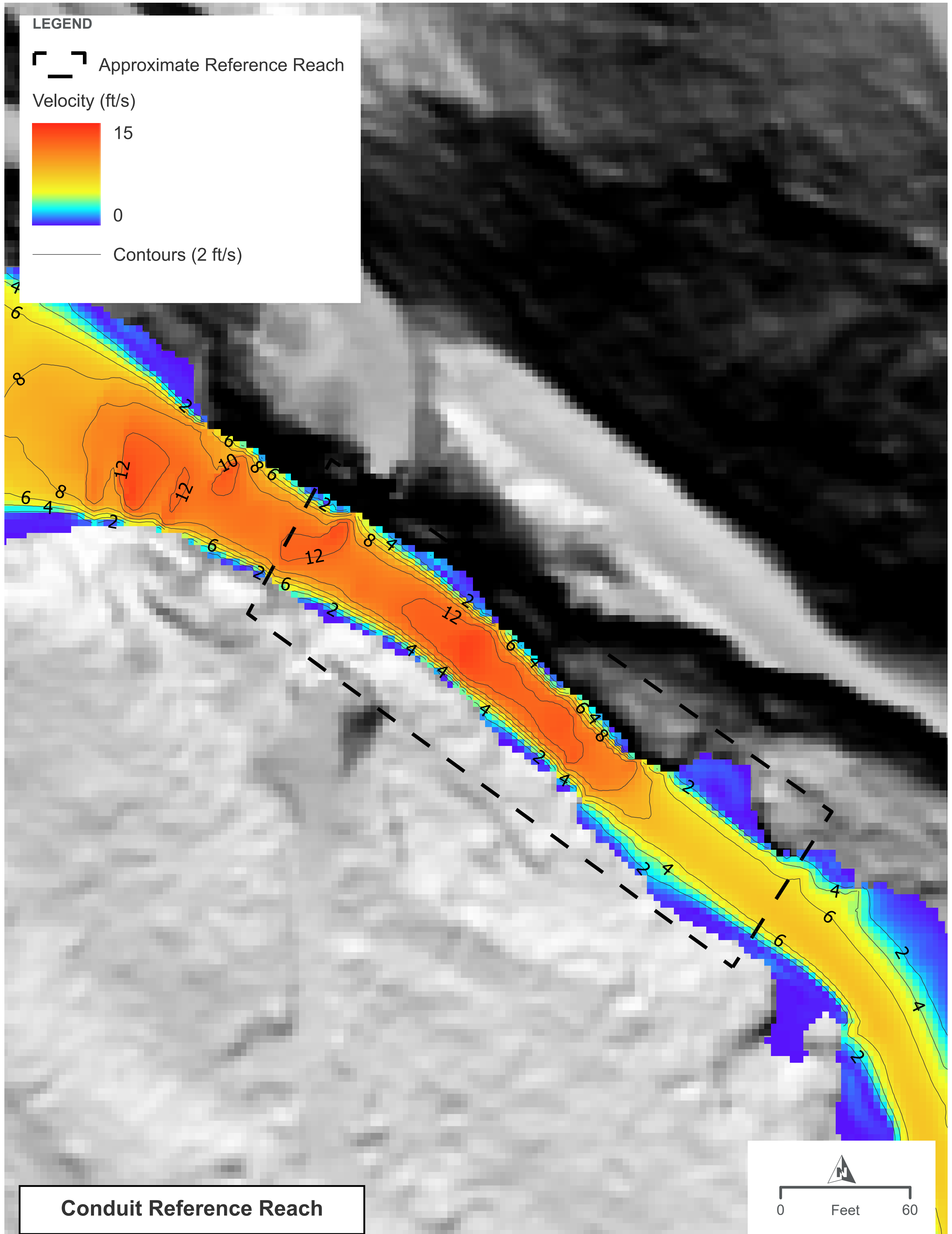
FIGURE 2

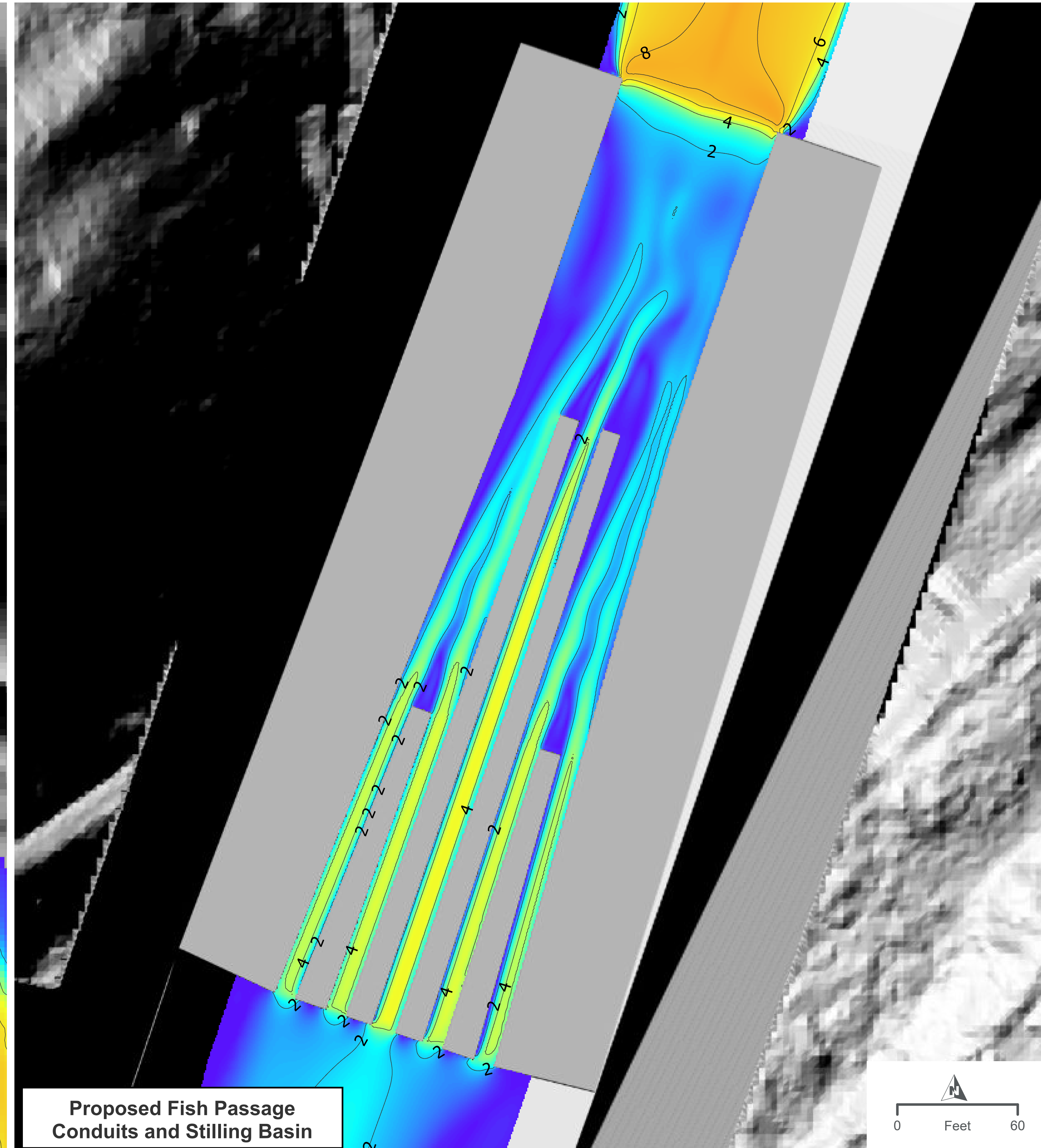


EXISTING VS PROPOSED (CHANNELS) - FISH PASSAGE HIGH DESIGN FLOW

VELOCITY

FIGURE 3

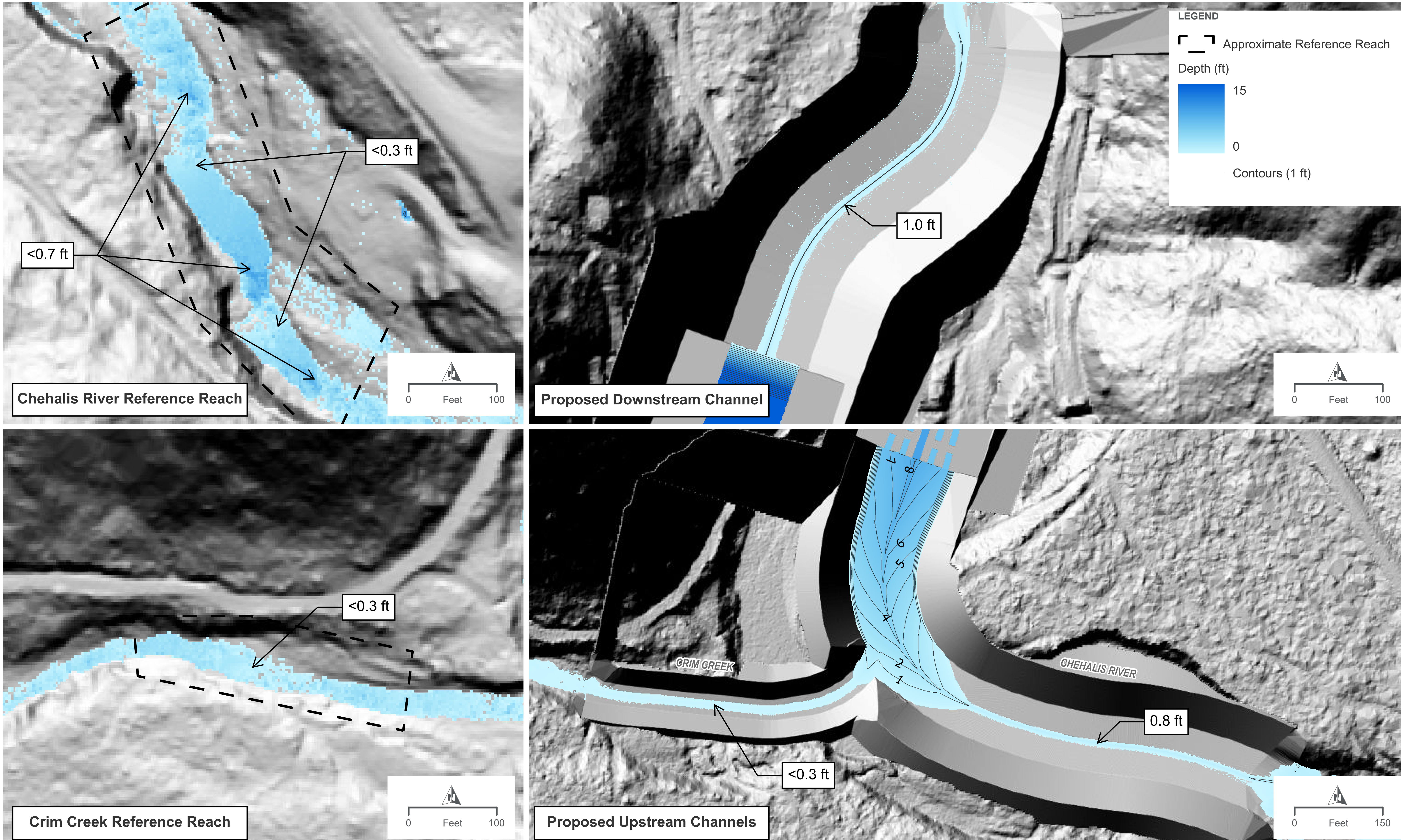




EXISTING VS PROPOSED (FISH PASSAGE CONDUITS) - FISH PASSAGE HIGH DESIGN FLOW

VELOCITY

FIGURE 4

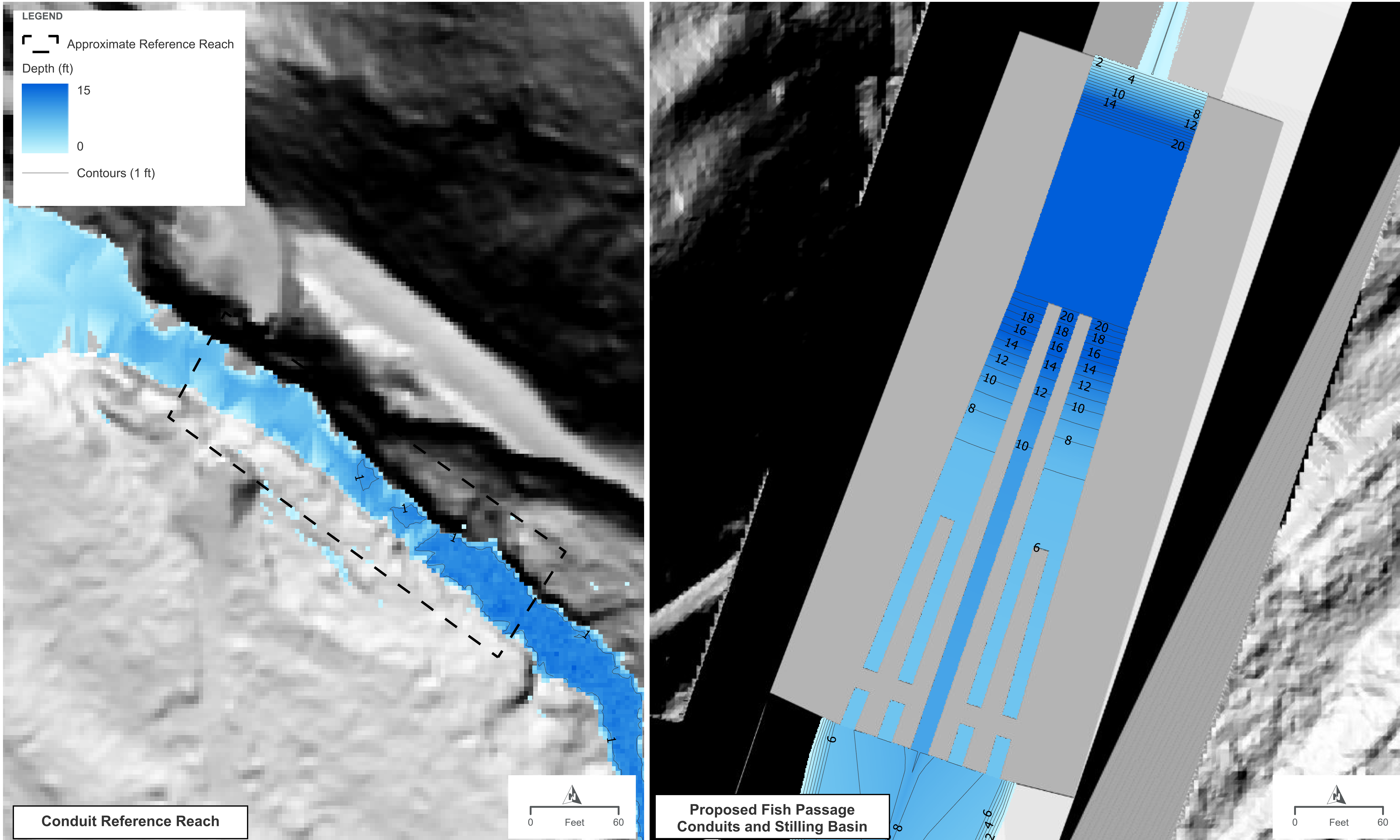


HDR

EXISTING VS PROPOSED (CHANNELS) - FISH PASSAGE LOW DESIGN FLOW

DEPTH

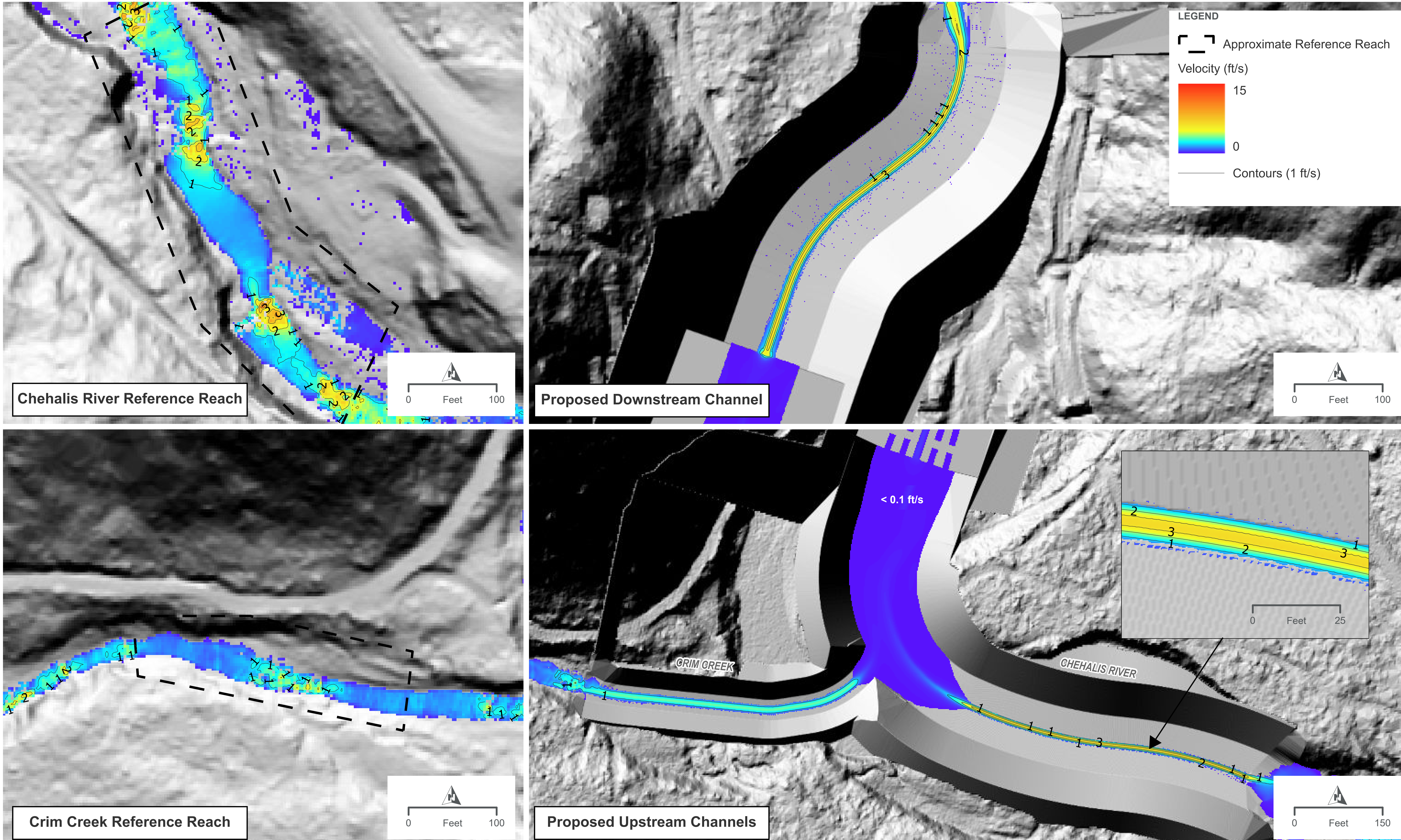
FIGURE 5



EXISTING VS PROPOSED (FISH PASSAGE CONDUITS) - FISH PASSAGE LOW DESIGN FLOW

DEPTH

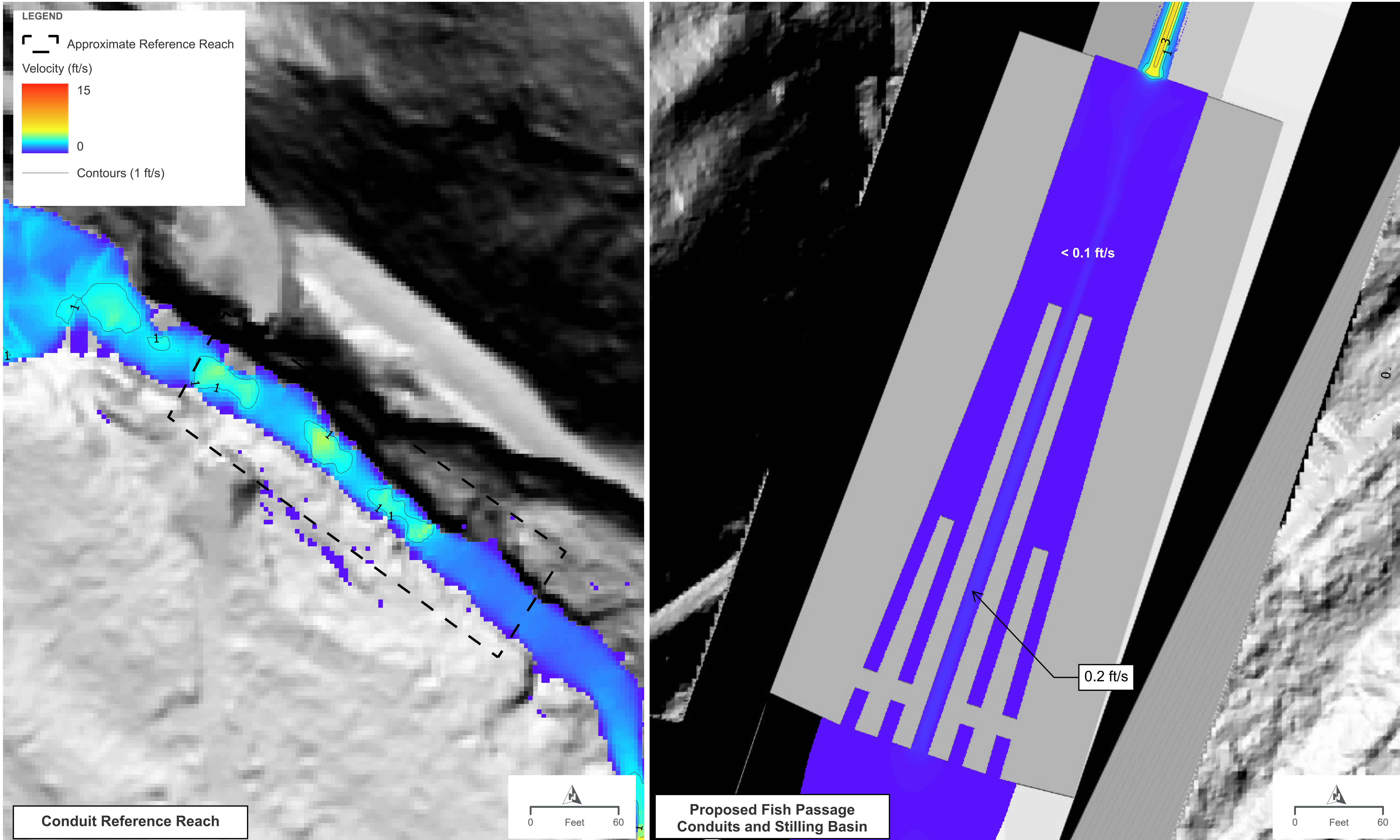
FIGURE 6



EXISTING VS PROPOSED (CHANNELS) - FISH PASSAGE LOW DESIGN FLOW

VELOCITY

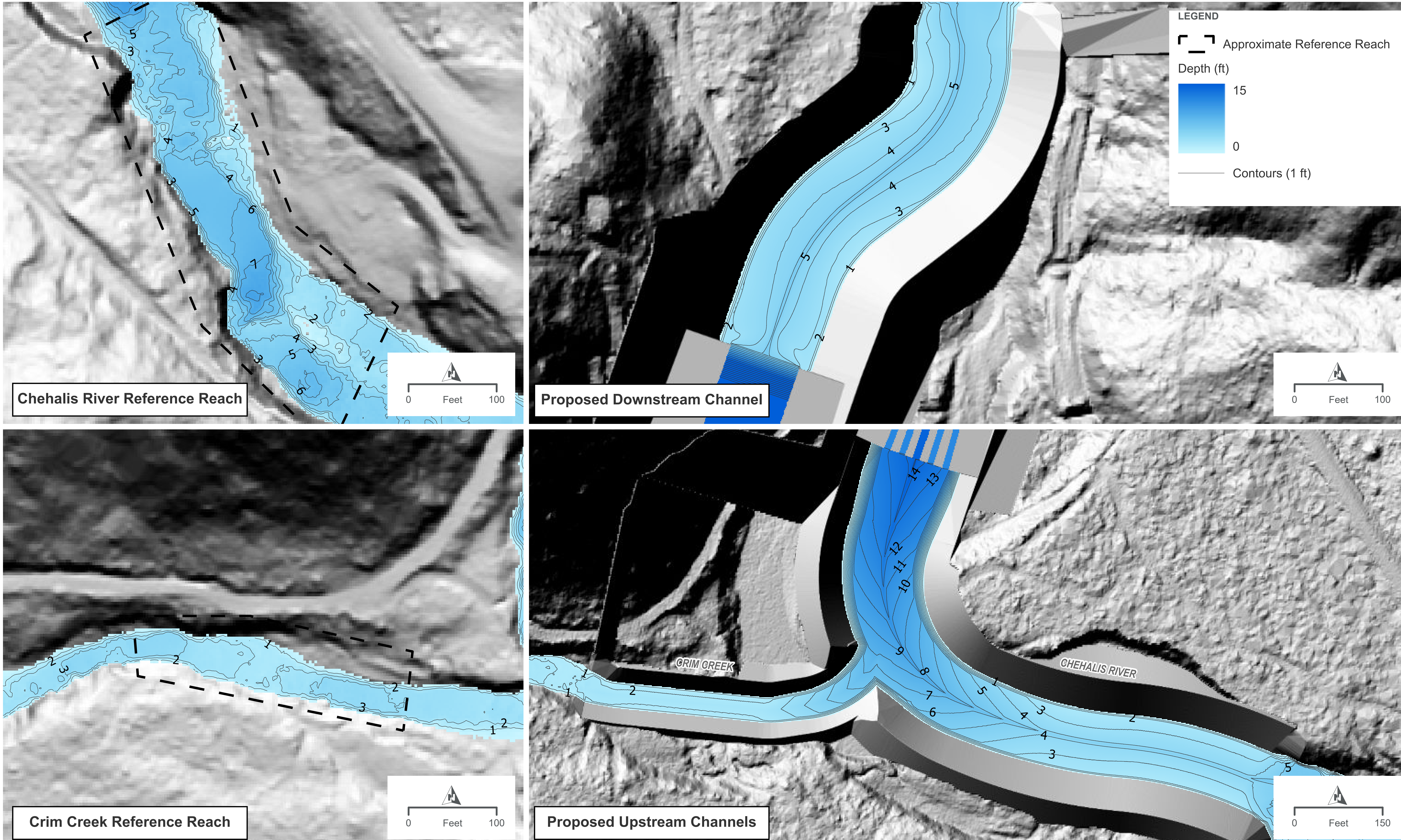
FIGURE 7



EXISTING VS PROPOSED (FISH PASSAGE CONDUITS) - FISH PASSAGE LOW DESIGN FLOW

VELOCITY

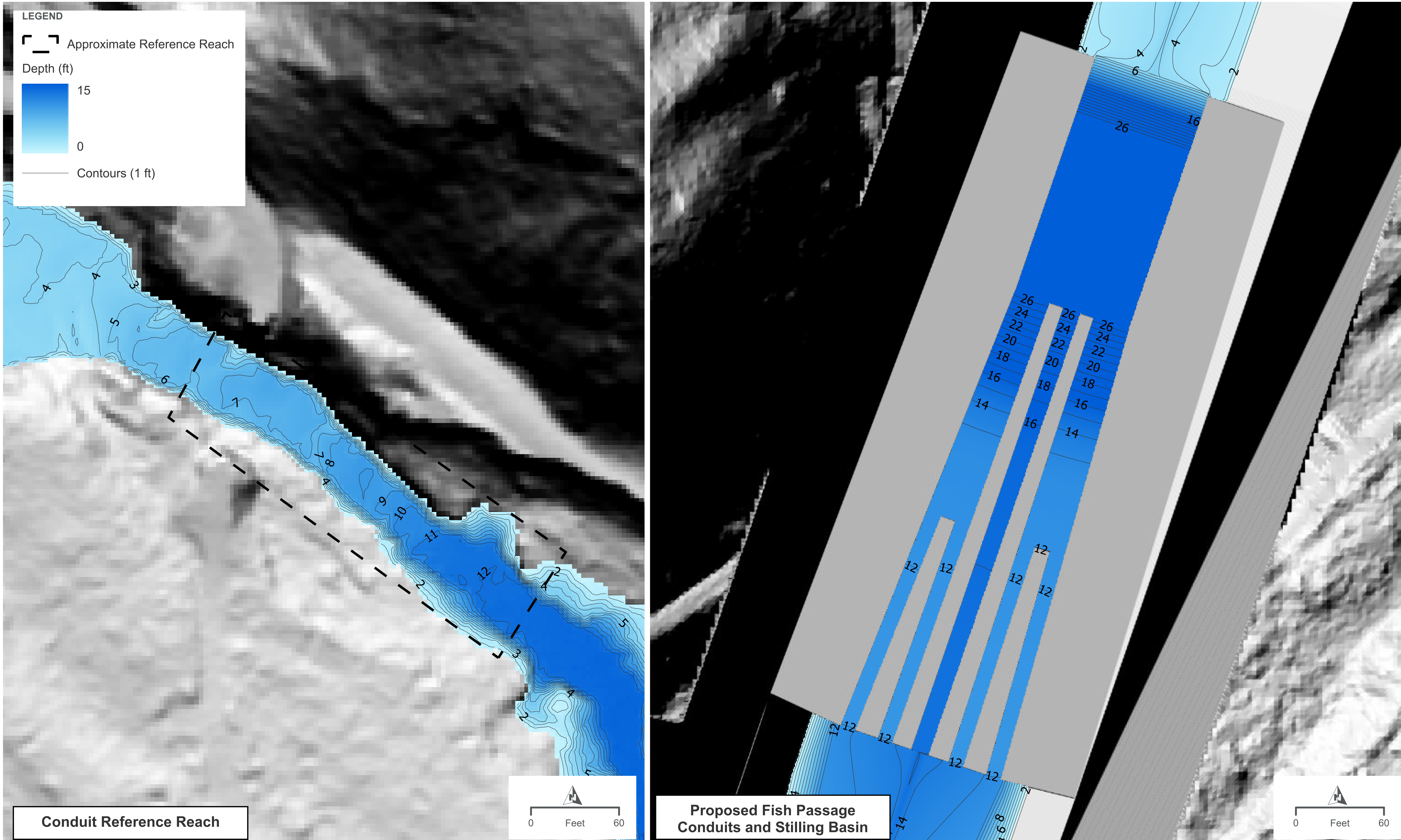
FIGURE 8



EXISTING VS PROPOSED (CHANNELS) - FISH PASSAGE HIGH DESIGN FLOW - CLIMATE CHANGE

DEPTH

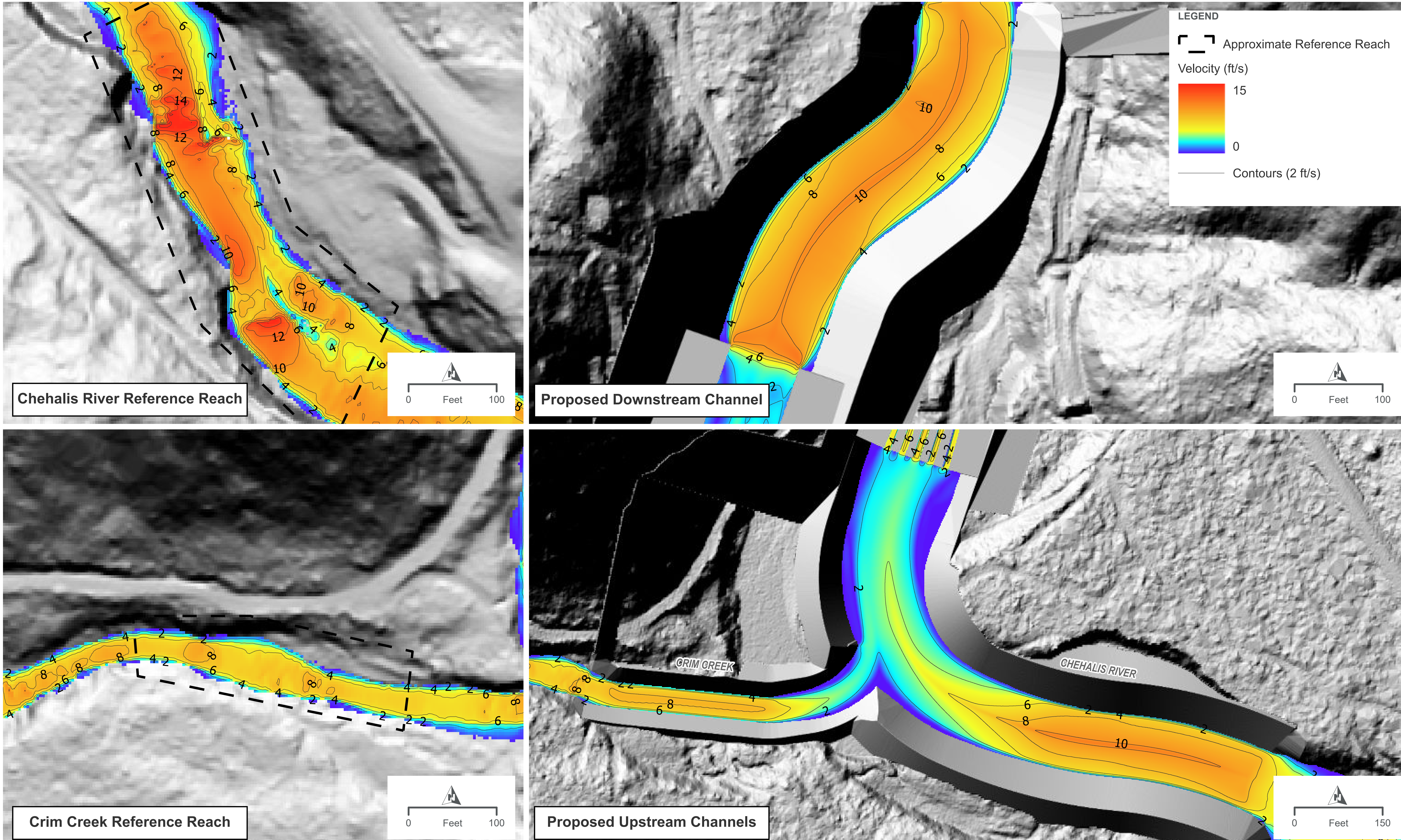
FIGURE 9



EXISTING VS PROPOSED (FISH PASSAGE CONDUITS) - FISH PASSAGE HIGH DESIGN FLOW - CLIMATE CHANGE

DEPTH

FIGURE 10

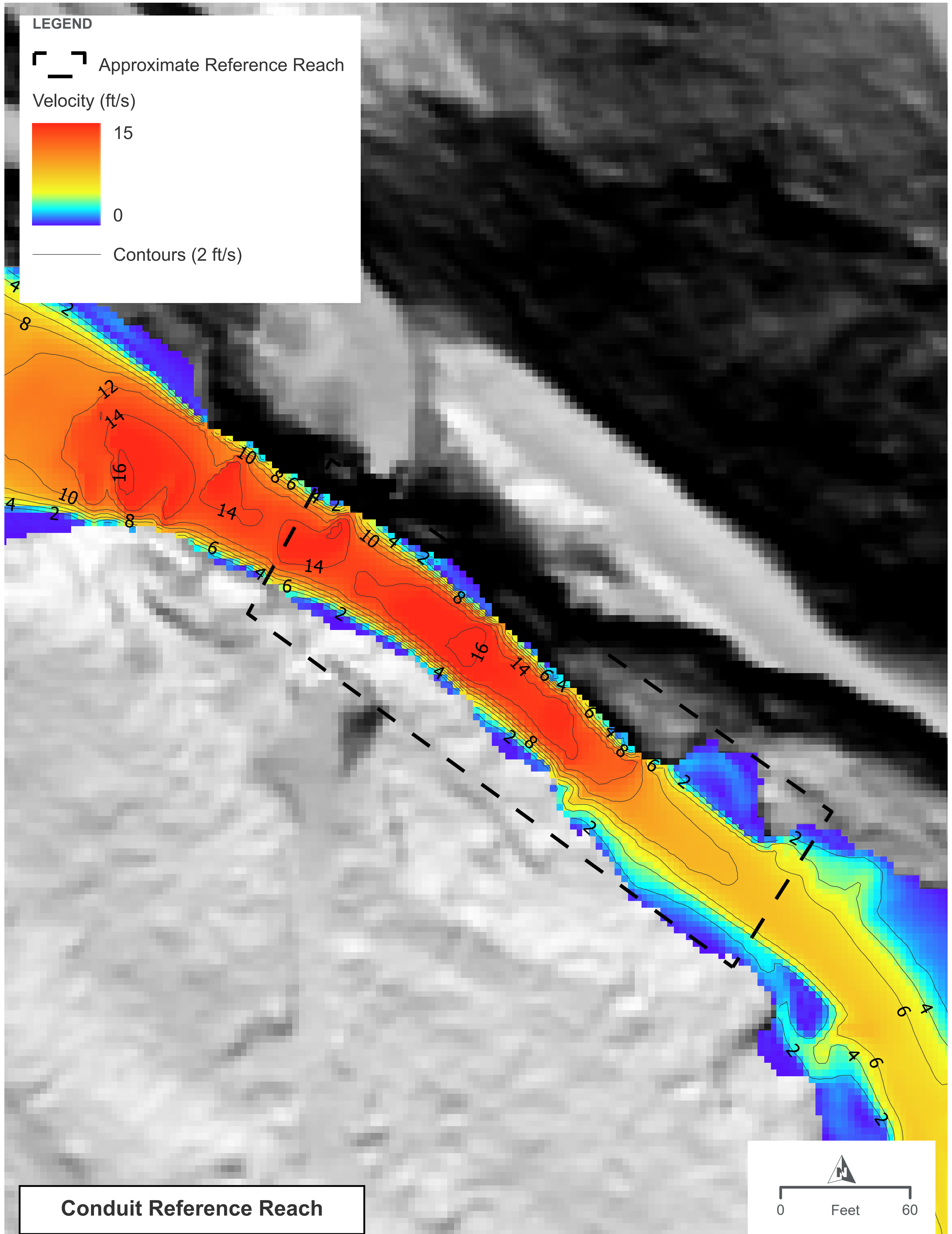


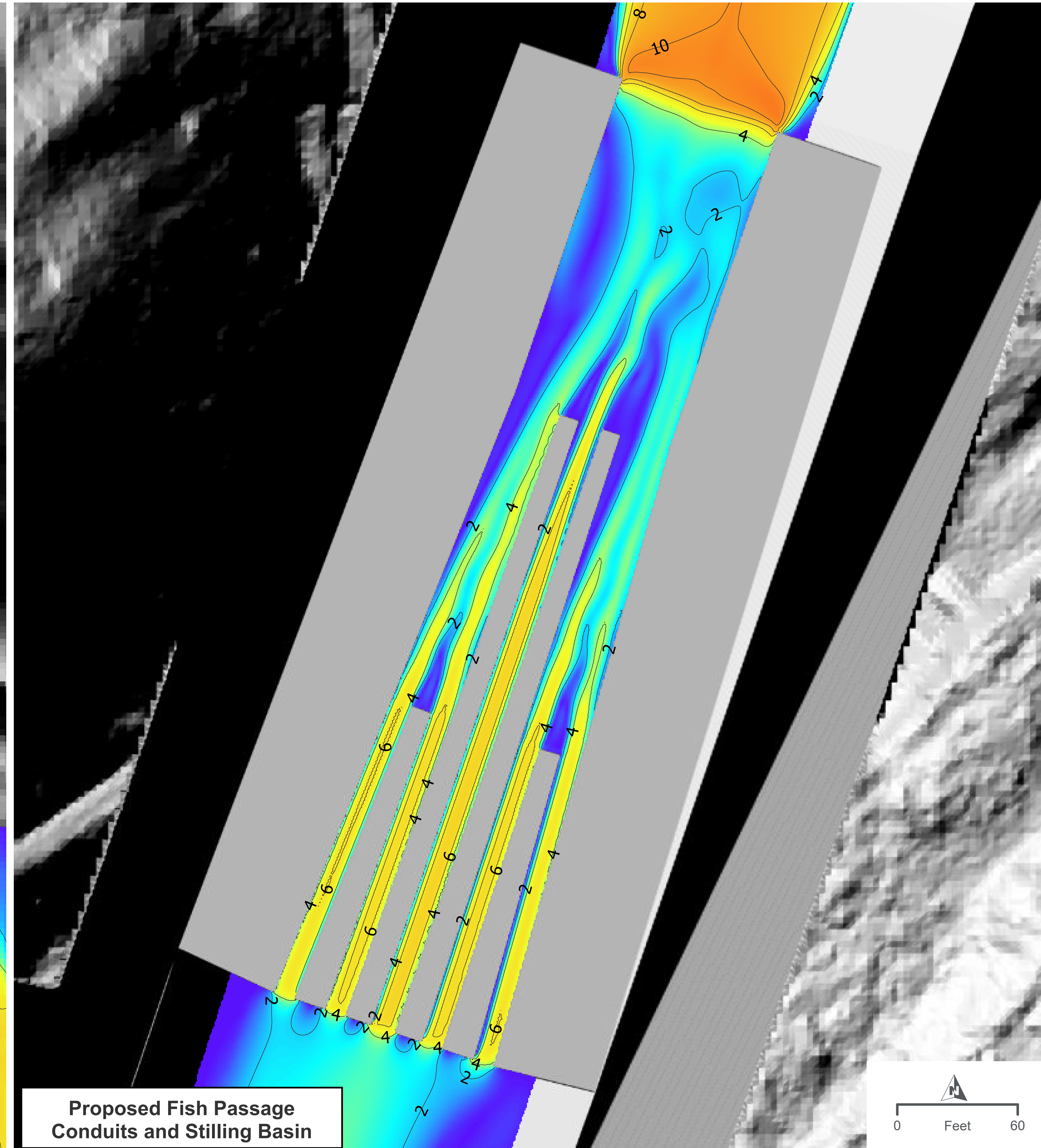
HDR

EXISTING VS PROPOSED (CHANNELS) - FISH PASSAGE HIGH DESIGN FLOW - CLIMATE CHANGE

VELOCITY

FIGURE 11

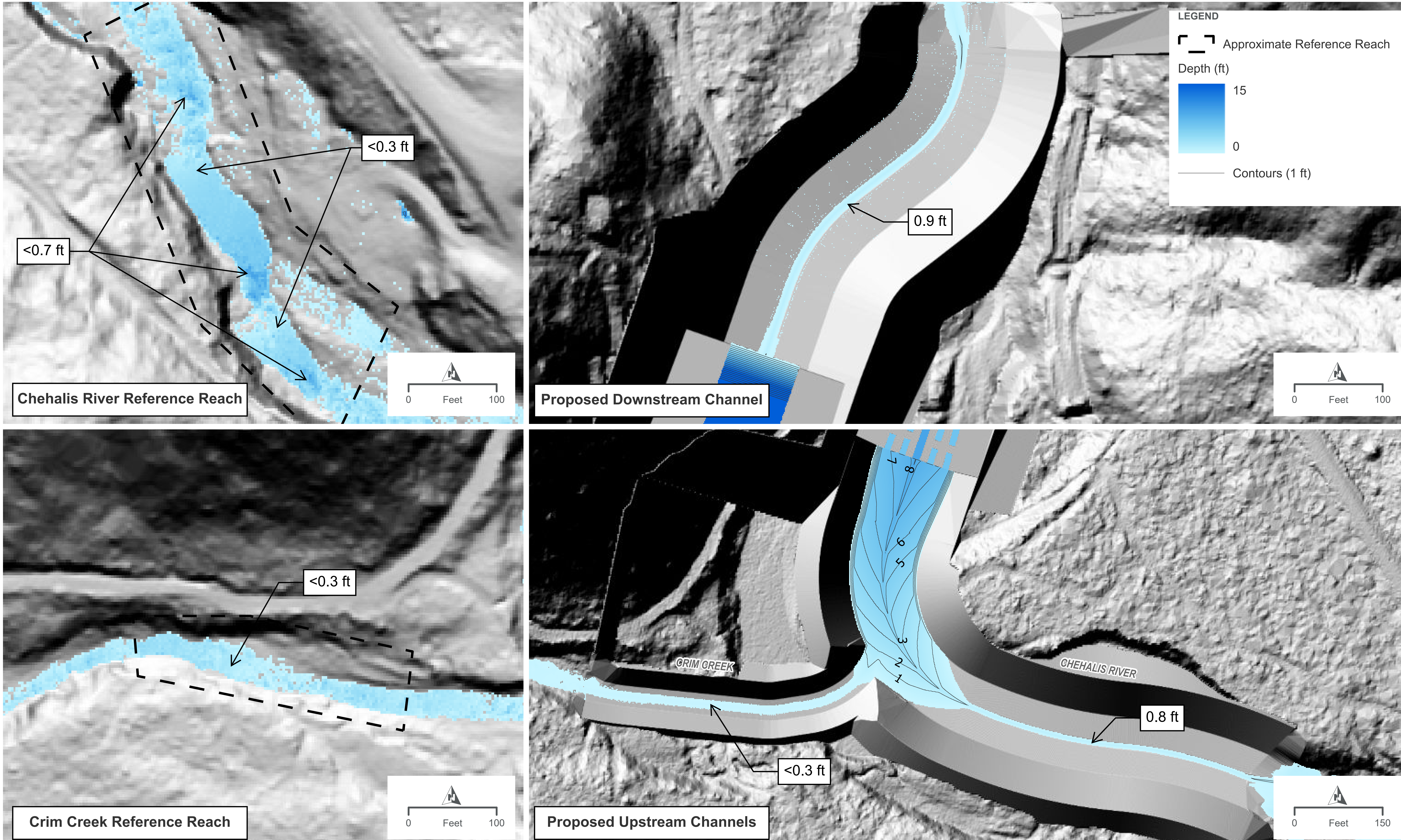




EXISTING VS PROPOSED (FISH PASSAGE CONDUITS) - FISH PASSAGE HIGH DESIGN FLOW - CLIMATE CHANGE

VELOCITY

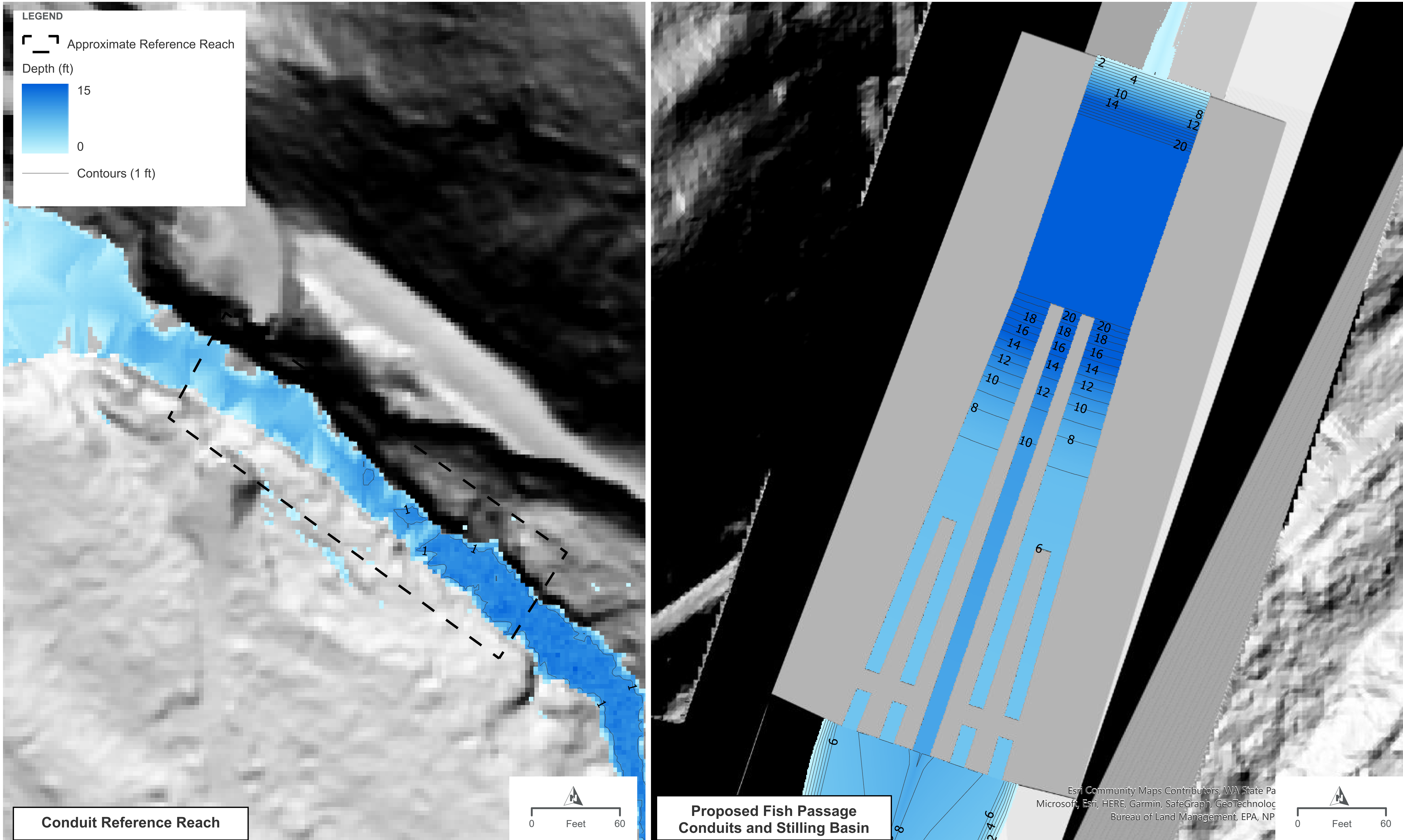
FIGURE 12



EXISTING VS PROPOSED (CHANNELS) - FISH PASSAGE LOW DESIGN FLOW - CLIMATE CHANGE

DEPTH

FIGURE 13

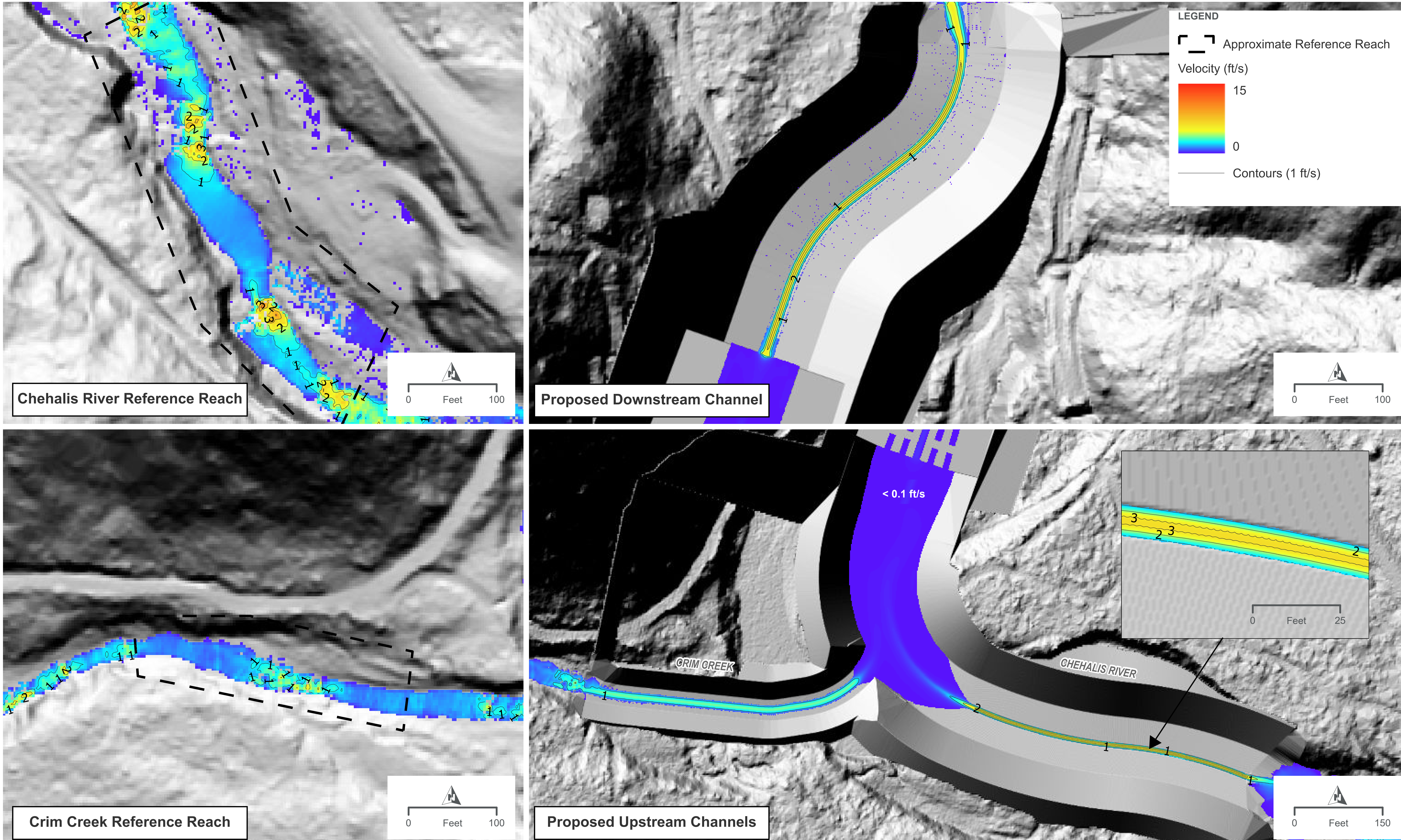


EXISTING VS PROPOSED (FISH PASSAGE CONDUITS) - FISH PASSAGE LOW DESIGN FLOW - CLIMATE CHANGE

DEPTH

FIGURE 14

PATH: C:\USERS\NSZIGETI\DESKTOP\CHEHALIS WORKING_GIS\CHEHALIS_HYDRAULICS\CHEHALIS_HYDRAULICS.APRX - USER: NSZIGETI - DATE: 1/2/2024

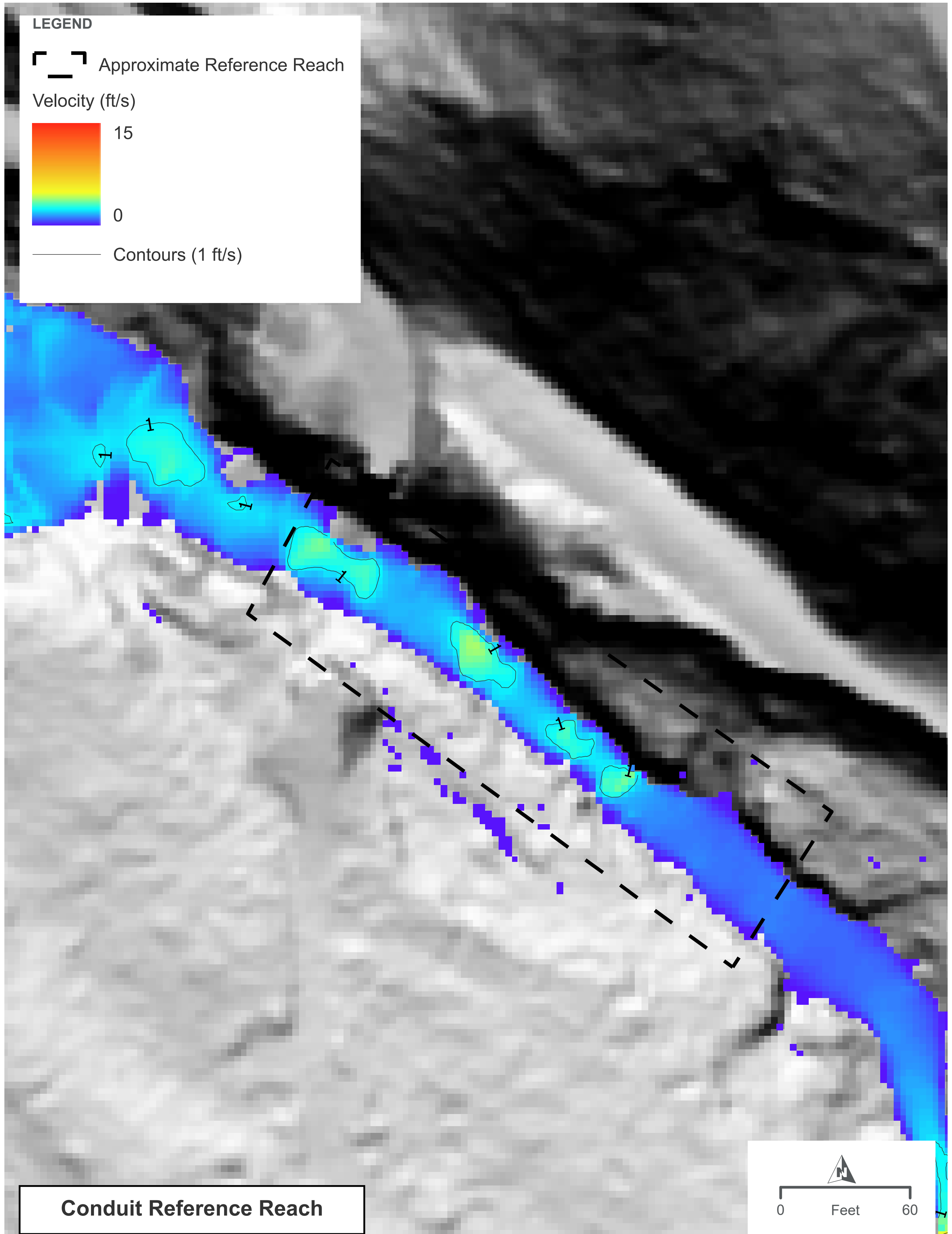


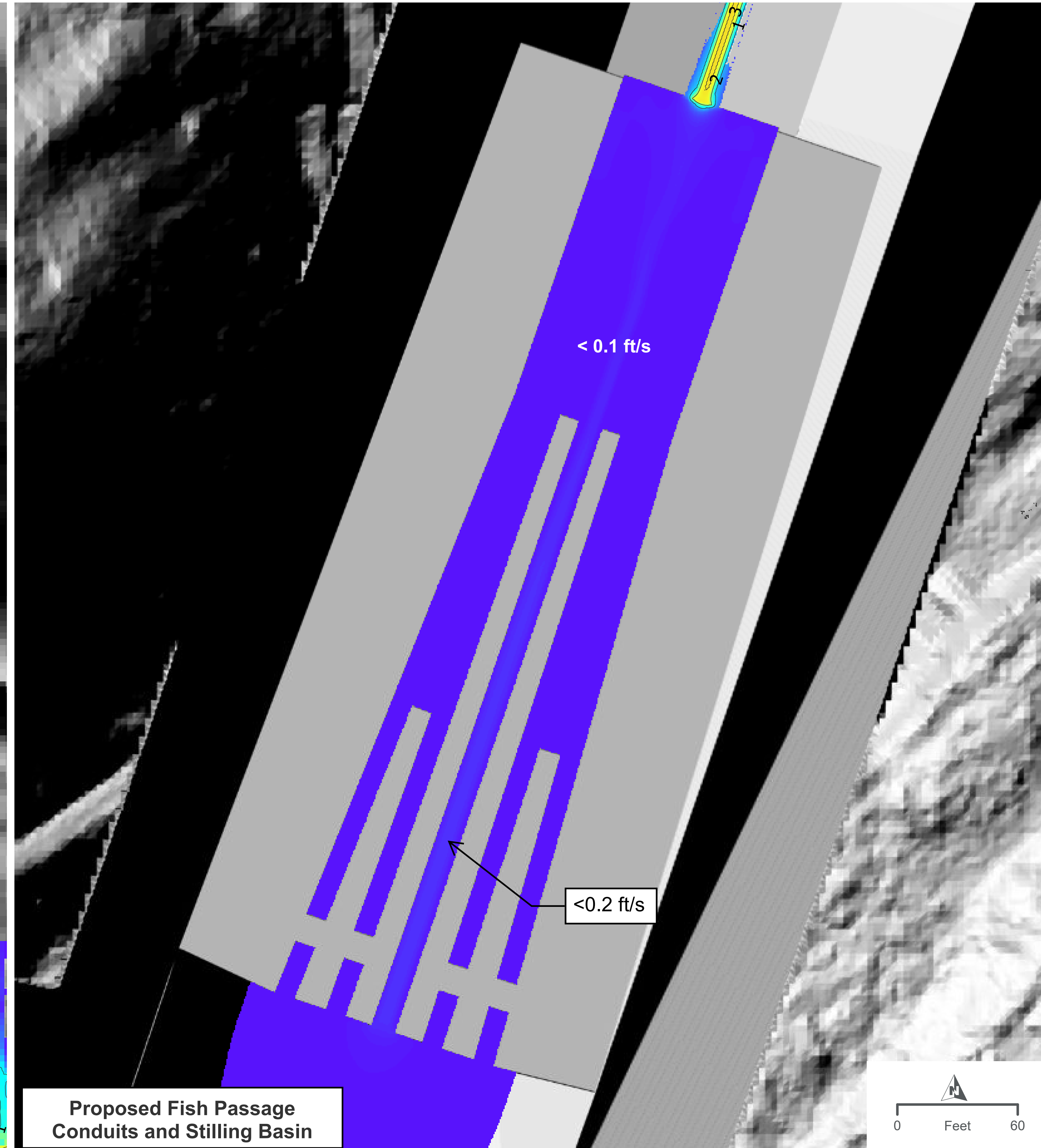
HDR

EXISTING VS PROPOSED (CHANNELS) - FISH PASSAGE LOW DESIGN FLOW - CLIMATE CHANGE

VELOCITY

FIGURE 15





EXISTING VS PROPOSED (FISH PASSAGE CONDUITS) - FISH PASSAGE LOW DESIGN FLOW - CLIMATE CHANGE

VELOCITY

FIGURE 16

Technical Memorandum

Date:	April 24, 2024
Project:	Chehalis River Basin Flood Damage Reduction Project
To:	Chehalis Basin Flood Control Zone District
From:	HDR Engineering, Inc.
Subject:	**Chehalis Construction Bypass Hydraulic Modeling**

1.0 Background

The Chehalis River Basin Flood Damage Reduction project objective is to develop recommendations for a series of measures aimed at reducing damage to the communities of the Chehalis River Basin from Pe Ell to Centralia during major flood events. Among these measures is a proposed Flood Retention Expandable (FRE) structure on the Chehalis River, south of the town of Pe Ell.

The Chehalis River Basin Flood Damage Reduction, Revised Project Description Report (RPDR) is a supplemental report documenting the relocation of and changes to the FRE facility (Proposed Project) as originally documented within the Combined Dam and Fish Passage Conceptual Design Report (HDR Engineering, Inc. [HDR] 2017) and FRE Dam Alternative Report (HDR 2018).

The RPDR describes, supports, contrasts, and illustrates the changes to the Proposed Project in a single comprehensive document.

2.0 Introduction and Purpose

The Chehalis Basin Flood Control Zone District is proposing to construct a new flood retention structure to reduce damage to life and property along the Chehalis River (Project). The Proposed Project is located just downstream of the Chehalis River's confluence with Crim Creek and design of the FRE structure is in a conceptual level of development.

As one document in Appendix D to the RPDR, this technical memorandum documents the hydraulic analysis of the proposed Chehalis River and Crim Creek construction bypass channels, which characterizes hydraulic conditions (i.e., depth, velocity) within the proposed channels in relation to cost estimating, constructability, and fish passage.

3.0 Methodology

To analyze the range of hydraulic conditions present at the Project location, a two-dimensional (2D) hydraulic model was employed. The hydraulic analysis was performed using HEC-RAS, Version 6.3 (RAS) hydraulic modeling software. RAS is applicable for flows in surface water bodies where vertical velocities and accelerations are small or relatively negligible in comparison to those in horizontal directions. It can simulate subcritical flow, supercritical flow, and the transition between the two. The Shallow Water Equation, Eulerian-Lagrangian Method (SWE-ELM) equation set in RAS was used for all conditions. Details on the techniques used to build the 2D RAS model are presented in the following sections.

3.1 Constructability

The 2D model was used to determine hydraulic conditions within the proposed construction bypass channels under a range of flows to inform, at a conceptual level, the minimum size of the channels and considerations for construction methods and quantities.

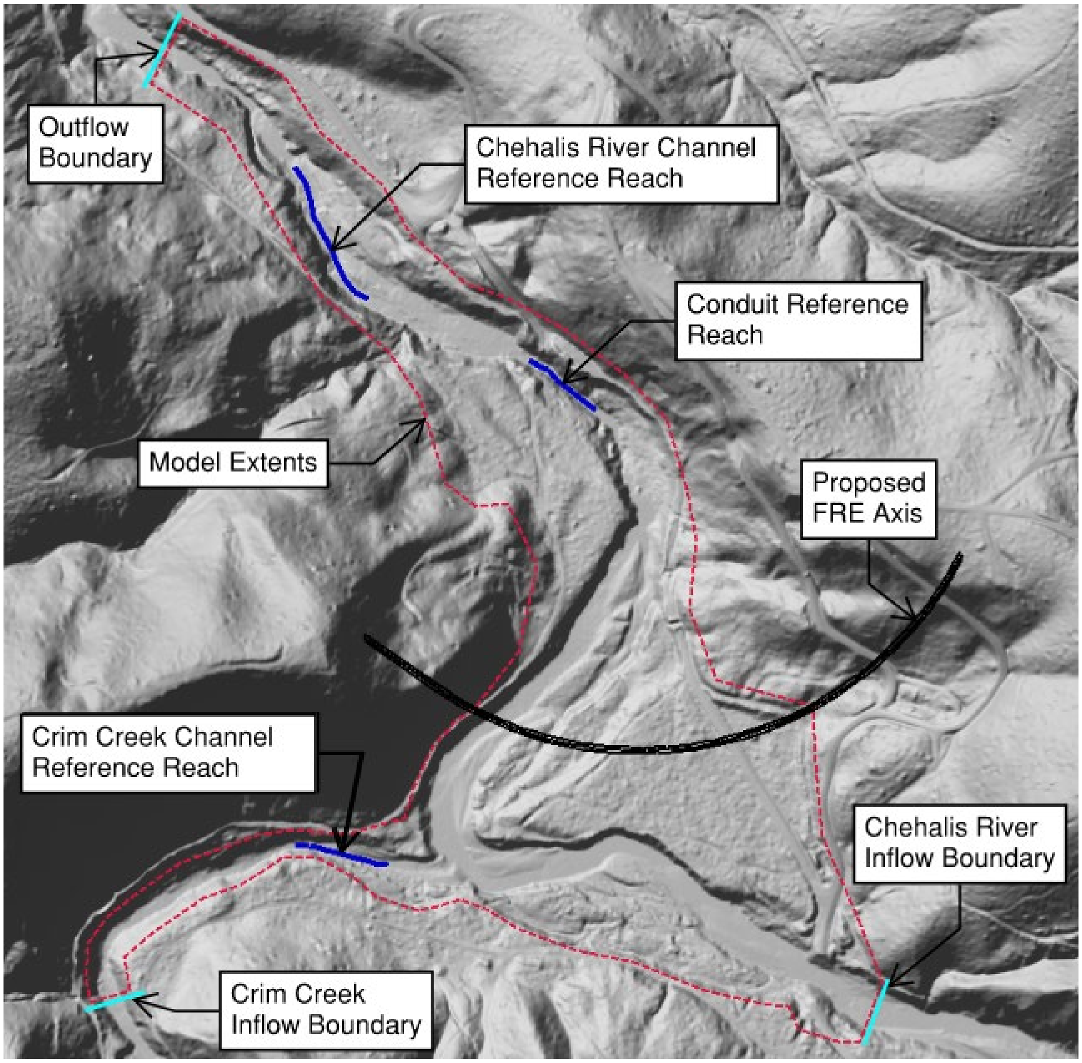
3.2 Fish Passage

The 2D model also was used to compare hydraulic conditions within existing reference reaches to those within the proposed construction bypass channels at the fish passage design flows. The primary criterion for this evaluation is for the proposed flow velocity and depth in the channels to mimic the flow velocity and depth occurring naturally in the reference reaches. This methodology is derived from the Washington Department of Fish and Wildlife's (WDFW) stream simulation design approach, which assumes that any fish present in the natural channel are not expected to be challenged by the stream simulation channel that looks and performs similarly to adjacent natural channels (WDFW, 2013). This design approach was jointly developed with the Fish Passage Technical Subcommittee in 2016 and differs from traditional fish passage criteria as individual target fish species are not governing specific velocity and depth criteria.

4.0 Model Extent

The 2D hydraulic model extents begin approximately 1,500 feet upstream of the confluence of Crim Creek and the Chehalis River and terminate approximately 3,800 feet downstream of the confluence (Figure 1). The model boundaries encompass the floodplain of the existing rivers, bypass channels, and reference reaches. The model extents were conservatively extended beyond the Project location to certify influences of the boundary conditions were negligible.

Figure 1. Locations of Reference Reaches and Model Boundaries



5.0 Surface Roughness

In the model, surface roughness (Manning's) was assigned to corresponding land surface characteristics approximated from aerial photographs and site visit photos from September 2022. In the modeling process, surface roughness was adjusted within standard tolerance limits (U.S. Geological Survey [USGS] 1989; Yochum et al. 2014). Table 1 summarizes the Manning's roughness coefficients as they relate to land surface designations.

Table 1. Summary of Modeled Manning's Roughness Coefficients

Land Cover	Manning's Value
Forested Overbank	0.1
Gravel Road	0.03
Exposed Bedrock	0.025

Land Cover	Manning's Value
Existing Chehalis River Channel[a]	0.032 – 0.065
Existing Crim Creek Channel	0.045
Rip Rap	0.07
Bypass Channels	0.05

[a] Chehalis River channel composition varies significantly from primarily small gravel and cobbles to very large boulders.

6.0 Inflow Boundary Conditions

The model was developed to evaluate flow depths and velocities within the bypass channels at five flows. For fish passage, this was at the lowest 95 percent (16 cubic feet per second [cfs]) and highest 5 percent (2,200 cfs) exceedance probability during target species fish migration periods at the FRE location. The exceedance flows are based on the best available, previous hydrology estimates (USGS 2018). The remaining flows are annual exceedance probability (AEP) flows for the Chehalis River at the Project location, developed by HDR via an unregulated flood frequency analysis (HDR 2023) using USGS Bulletin 17C methodology (USGS 2018).

Flows at the Project location were then split between Crim Creek and the Chehalis River to assign inflow boundary conditions, based on their respective drainage basins upstream of the confluence. Crim Creek accounts for 18 percent of the total flow while the Chehalis River accounts for 82 percent.

Table 2 presents the selected design discharges. The peak flows were held constant throughout each simulation (i.e., steady state). A 25-year AEP was selected as the maximum return interval for design of the construction bypass channels, which is typical for design of temporary construction facilities for the expected construction duration.

Table 2. Selected River Flows at the FRE Location

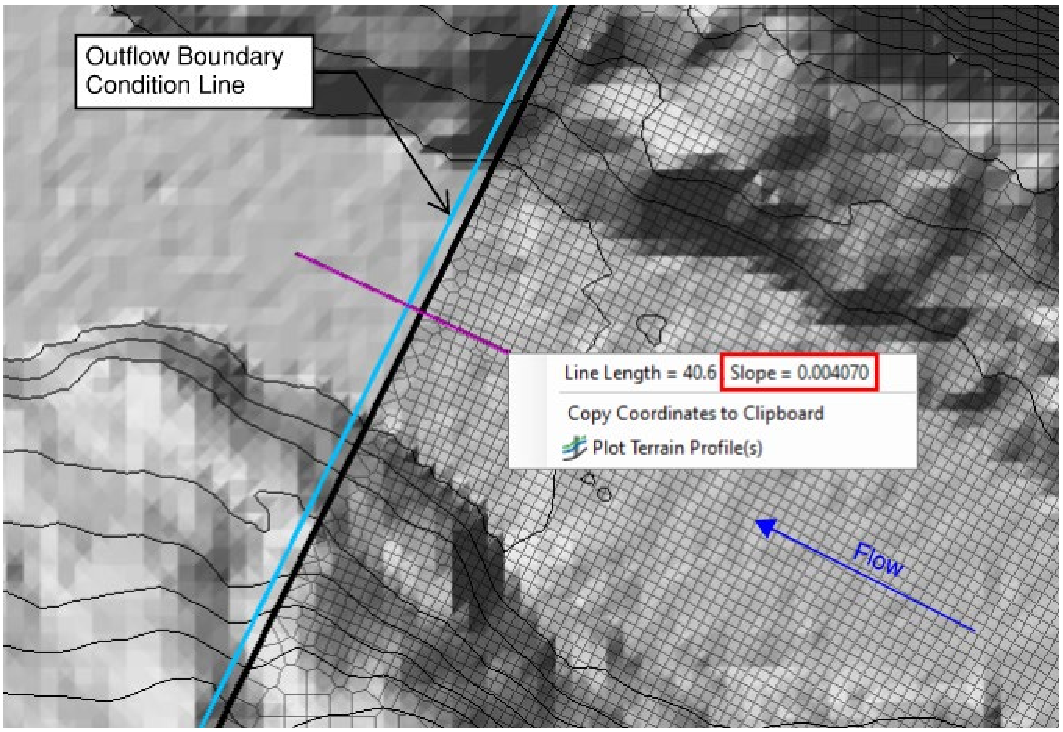
Flow Event	Crim Creek Flow (cfs)	Chehalis River Flow (cfs)	Total Flow (cfs)
Low fish passage design flow[a]	3	13	16
High fish passage design flow[a]	396	1,804	2,200
5-year flood[b]	2,790	12,710	15,500
10-year flood[b]	3,636	16,564	20,200
25-year flood[b]	4,824	21,976	26,800

[a] Low and high fish passage design flows are based on the lowest 95% and highest 5% exceedance flows, respectively, during various fish migration periods (2018 CHTR Preliminary Design Report).
[b] AEP flows were obtained from an unregulated flood frequency analysis (HDR 2023).

7.0 Outflow Boundary Conditions

The 2D model was simulated using the normal depth boundary condition at the sole outflow location. The energy slope assigned to calculate the normal depth water surface elevation was assumed to match the channel slope at the corresponding location of the digital terrain model. Figure 2 illustrates how measurement tools in RAS were used to calculate an approximate channel slope of 0.4 percent at the outflow location.

Figure 2. Outflow Boundary Condition Slope



8.0 Existing Conditions

8.1 Topographic Data Collection

Topographic data currently being used for Project components is Light Detection and Ranging (LiDAR) survey data downloaded from the Washington State Department of Natural Resources' online LiDAR portal. LiDAR data available at the Project location was flown in 2019 and the corresponding digital terrain model has a 3-foot resolution (Quantam Spatial 2019). Table 3 provides the coordinate system in which LiDAR data was collected.

Table 3. LiDAR Coordinate System

Coordinate System Component	Description	Units
Projection	WA State Plane South	n/a
Horizontal Datum	North American Datum of 1983	U.S. Survey Feet
Vertical Datum	North American Vertical Datum of 1988	U.S. Survey Feet

Bathymetric data is currently unavailable for the Project location. Future studies will incorporate bathymetric data as it becomes available.

8.2 Reference Reach

Reference reaches that are naturally occurring in the river channel were selected to closely represent a selected design feature. The reference reaches are intended to verify design features have the same or better fish passage capabilities than the existing river channel. The selected reference reaches are identified in Figure 1 and described below:

- **Chehalis River Bypass Reference Reach:** This reach is approximately 650 feet long and located about 400 feet downstream of the confluence with Crim Creek. It is comprised of medium cobbles and some boulders with an approximate slope of 0.5 percent.
- **Crim Creek Bypass Reference Reach:** This reach is located approximately 200 feet upstream of the confluence with the Chehalis River. It is comprised of small cobbles and some small boulders with an approximate slope of 1.3 percent.

9.0 Proposed Conditions

The proposed terrain was developed by merging the existing LiDAR topography with the proposed channel (developed in AutoCAD Civil3D). The channel is essentially burned into the existing topography and does not reflect the full grading that would be required to construct the channels sized to contain the 25-year AEP flow.

The proposed Chehalis River and Crim Creek channels were modeled based on the typical cross sections illustrated in Figure 3. These channel cross sections and slopes are conceptual based on the Washington Department of Fish and Wildlife's design approach to mimic observed geomorphology. Bankfull widths, visually approximated bathymetry, and reference reaches for the Chehalis River and Crim Creek channels were obtained during September 2022 field work, with cross sections and slopes obtained from LiDAR. Refinement of the bypass channels and tie-in points will occur during preliminary design, after bathymetry data has been incorporated. Table 4 compares the proposed slopes to the reference reach slopes.

Figure 3. Typical Bypass Channel Cross Sections

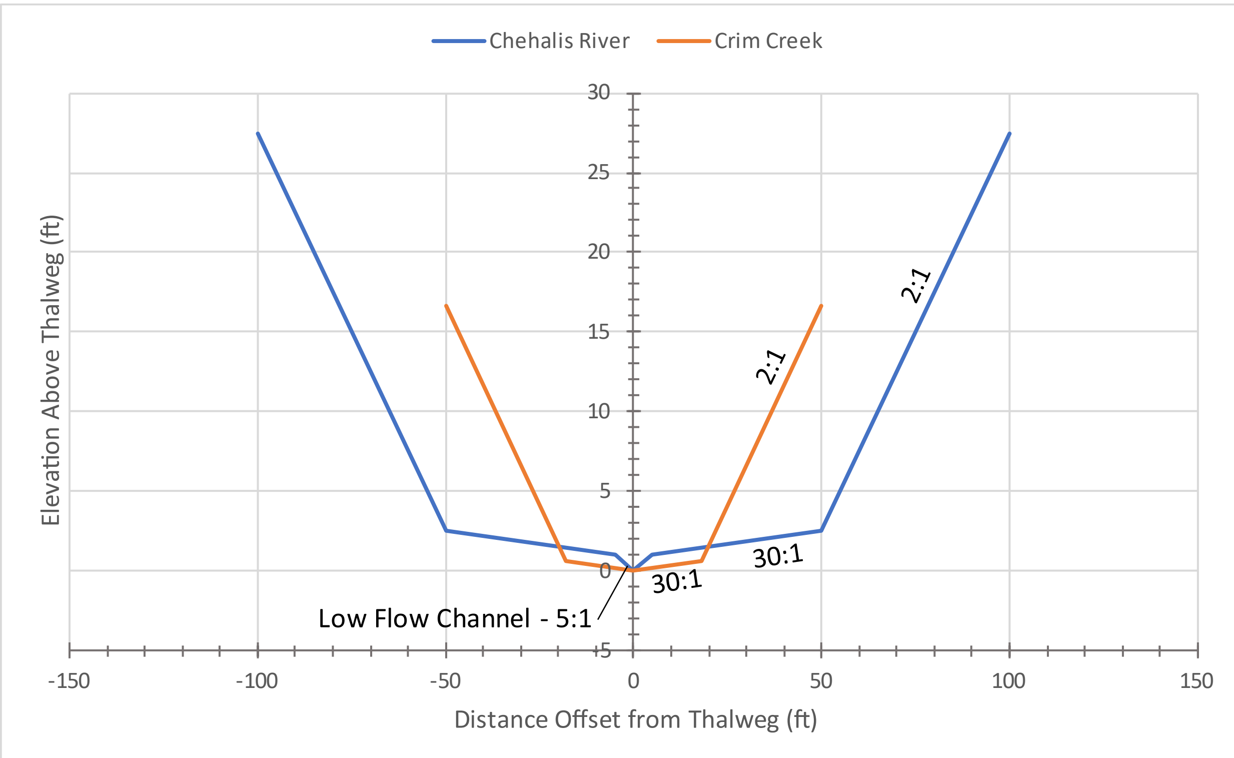


Table 4. Summary of Proposed Slopes

Proposed Channel	Proposed Slope (%)	Reference Reach Slope (%)
Crim Creek Bypass	1.2	1.3
Chehalis River Bypass	1.2	0.5

10.0 2D HEC-RAS Model Results

10.1 Constructability

The 5-year, 10-year, and 25-year AEP flow results are presented in Figures A1 through A6 in Attachment 1 and briefly summarized below.

10.1.1 5-year AEP (15,500 cfs)

Depth in the Chehalis River bypass channel generally increases moving upstream to downstream under all flows. This appears to be caused by the addition of flow from Crim Creek discharging into the bypass channel, and some backwater effects from the narrow rock canyon downstream of the bypass channel that constricts flow. Depths at the thalweg range from about 8 to 15 feet in the Chehalis River bypass and 7 to 11 feet in the Crim Creek bypass.

Velocity under the 5-year flow is predominantly 8 to 13 feet per second (ft/s) in the Chehalis River bypass and 5 to 8 ft/s in the Crim Creek bypass. Velocity is relatively constant across the channel cross section, with a thin low-velocity area along the channel margins. Overall, velocities in the bypass channels are attenuated compared to the immediate upstream and downstream reaches.

10.1.2 10-year AEP (20,200 cfs)

Under the 10-year flow, depths increase to approximately 10 to 18 feet in the Chehalis River bypass and 9 to 13 feet in the Crim Creek bypass.

Velocity along the channel centerline increases to 10 to 15 ft/s in the Chehalis River bypass and 6 to 10 ft/s in the Crim Creek bypass. The low-velocity margins widen compared to the 5-year flow and include small recirculating eddies in some locations.

10.1.3 25-year AEP (26,800 cfs)

Under the 25-year flow, depths increase to approximately 12 to 22 feet in the Chehalis River bypass and 10 to 16 feet in the Crim Creek bypass.

Velocity along the channel centerline increases to 11 to 16 ft/s in the Chehalis River bypass and 7 to 11 ft/s in the Crim Creek bypass. The low-velocity margins further widen compared to the 5-year and 10-year flows. The recirculating eddies visible in the 10-year flow become more prominent, most notably on the right bank of the Chehalis River bypass and downstream of its main bend.

Under existing conditions, the overbank areas upstream of the confluence with Crim Creek (the Chehalis River left overbank and Crim Creek right overbank) are activated under the 25-year flow. Design and construction of the upstream portion of the bypass channels should include sufficiently sized and located berms to contain flows at or below the 25-year flow within the proposed bypass channels.

10.2 Fish Passage

The results for the high and low fish passage design flows are presented in Figures A7 through A10 in Attachment 1 and briefly summarized below.

10.2.1 High Fish Passage Design Flow (2,200 cfs)

Under the high fish passage design flow, the maximum depth in the bypass channels is similar to the reference reaches, generally ranging from 3 to 4 feet in the Chehalis River bypass and 2 to 3 feet in the Crim Creek bypass. Some small portions of the reference reach experience depths greater than 5 feet.

Bypass channel velocities are generally at or below velocities in the reference reaches. Maximum velocities are located in the center of the Chehalis River bypass, with low-velocity regions along the margins of the channel. Preserving these low-velocity regions during future design phases is important for juvenile fish passage through the bypass channels.

10.2.2 Low Fish Passage Design Flow (16 cfs)

Under the low fish passage design flow (16 cfs), channel depths are consistent with reference reach depths. Depths in the reference reaches are less than 1 foot in the Chehalis River and less than 0.3 feet in Crim Creek. The proposed bypass channels meet or exceed these depths. Depths are shallowest at the upstream tie-in of the Chehalis River bypass channel, where flow transitions from the natural channel to the low flow channel. The preliminary design should carry the full low-flow channel up to the start of the channel grading.

Maximum velocities in the bypass channels are generally at or below the maximum velocities in the reference reaches. One notable difference in the Chehalis River bypass is that flow in the reference reach is spread over a much wider channel than the low-flow channel, which distributes velocity asymmetrically and provides multiple paths for fish to navigate the channel. In contrast, the proposed low-flow channel is thin and uniform and concentrates flow/velocity.

11.0 Considerations for Preliminary Design

For this conceptual design modeling effort, the bypass channels were roughly cut into the existing topography and sized to contain the flow. Preliminary design should further refine the channel grading to provide a continuous low-flow channel for fish passage and extend the channel slopes to provide adequate freeboard. Furthermore, berms or other structures required to contain high flows in the channel should be investigated and designed.

The channel experiences high velocities under the modeled peak flood events. Some form of bank and streambed protection should be included to mitigate damage to the bypass channels if high-flow events occur while they are operational. The streambed material should be sized to mitigate damage at flows at or below the 25-year flow, while simulating the native streambed material through the reference reach to the maximum extent practicable.

12.0 References

HDR

2017 Combined Dam and Fish Passage Conceptual Design Report. June 2017.

2018 Combined Dam and Fish Passage Supplemental Design Report FRE Dam Alternative Report. September 2018.

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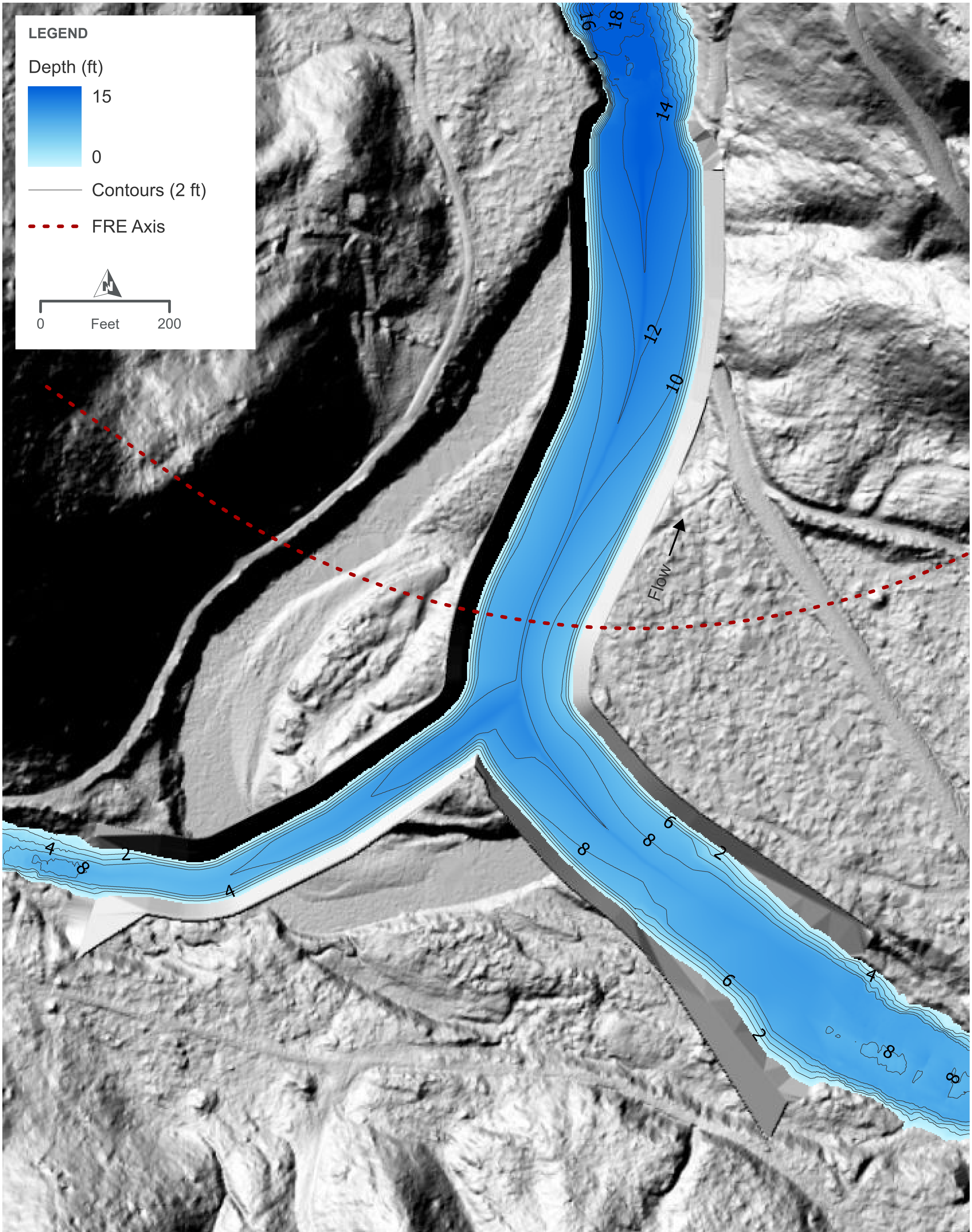
2014 Photographic Guidance for Selecting Flow Resistance Coefficients in High-Gradient Channels. Gen. Tech. Rep. RMRS-GTR-323. Fort Collins, CO: U.S. Department of Agriculture, Forest Service, Rocky Mountain Research Station. 91 p. 2014.

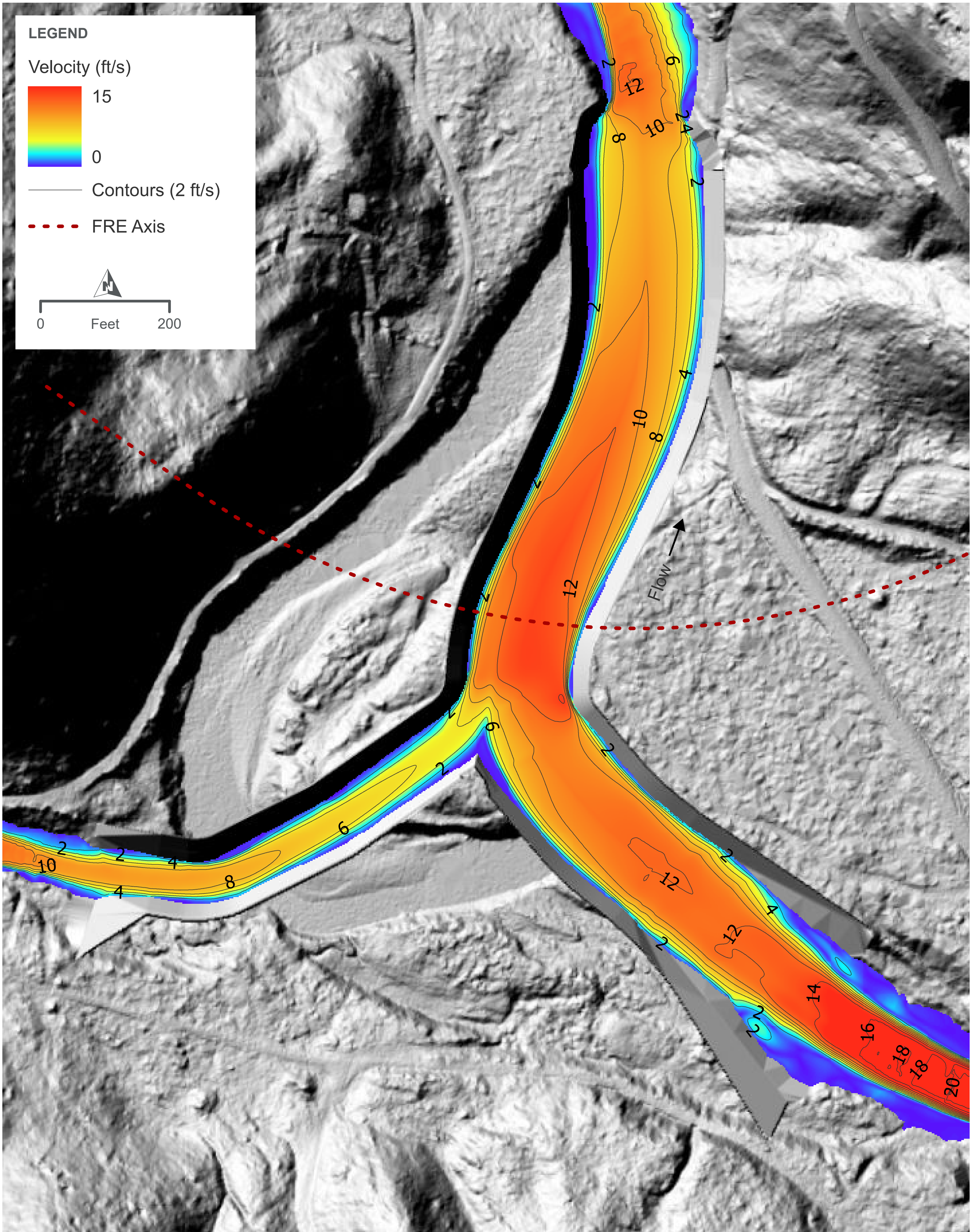
13.0 Abbreviations List

2D	two-dimensional
AEP	Annual Exceedance Probability
cfs	cubic feet per second
FRE	flood retention expandable
ft/s	feet per second
LiDAR	Light Detection and Ranging
Project	proposed Chehalis River Basin Flood Damage Reduction Project
RAS	HEC-RAS Version 6.3
SWE-ELM	Shallow Water Equation, Eulerian-Lagrangian Method
USGS	U.S. Geological Survey

Attachment 1. 2D HEC-RAS Modeling Results



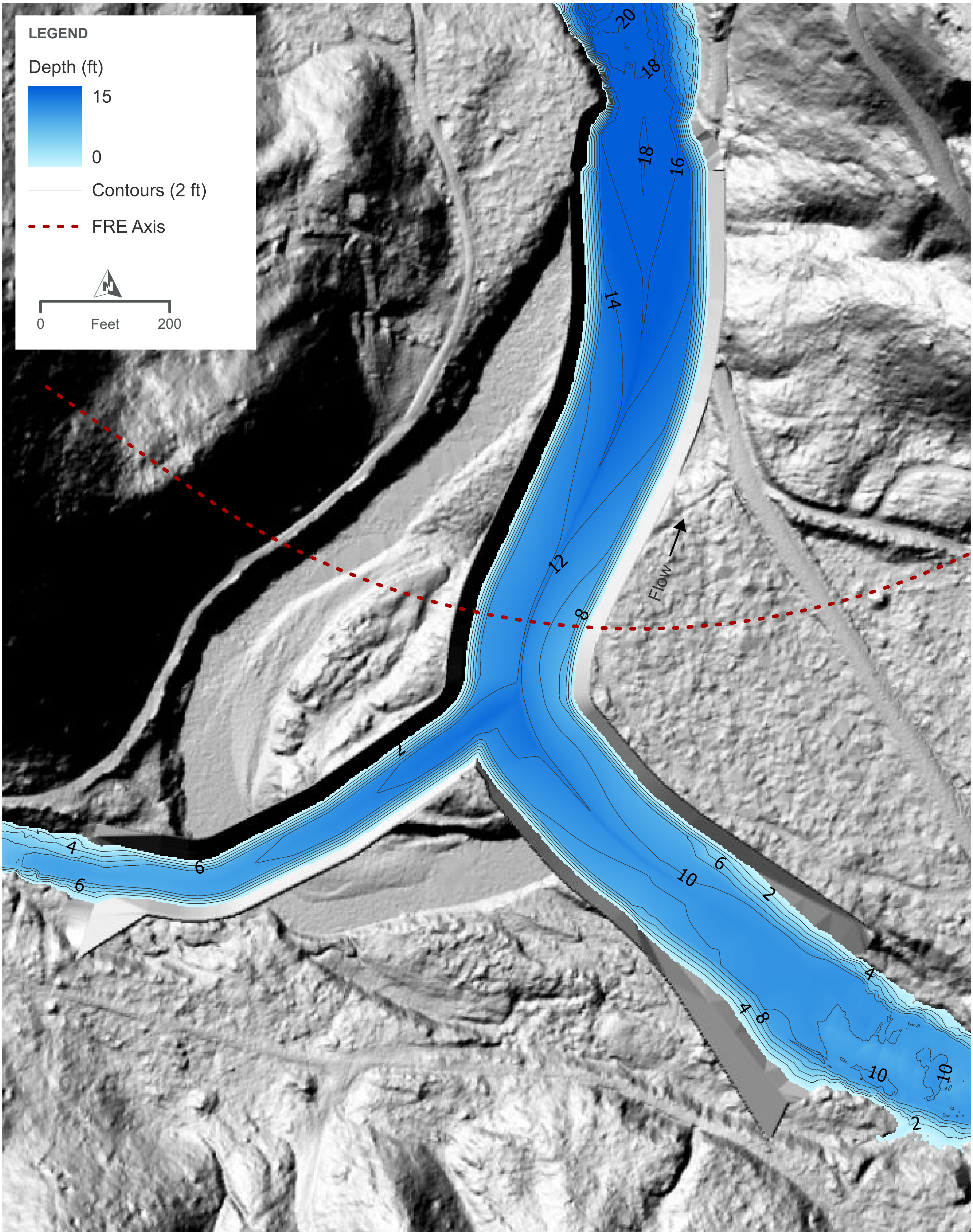
PROPOSED CONDITIONS - 5-YEAR
DEPTH
FIGURE A1



PROPOSED CONDITIONS - 5-YEAR

VELOCITY

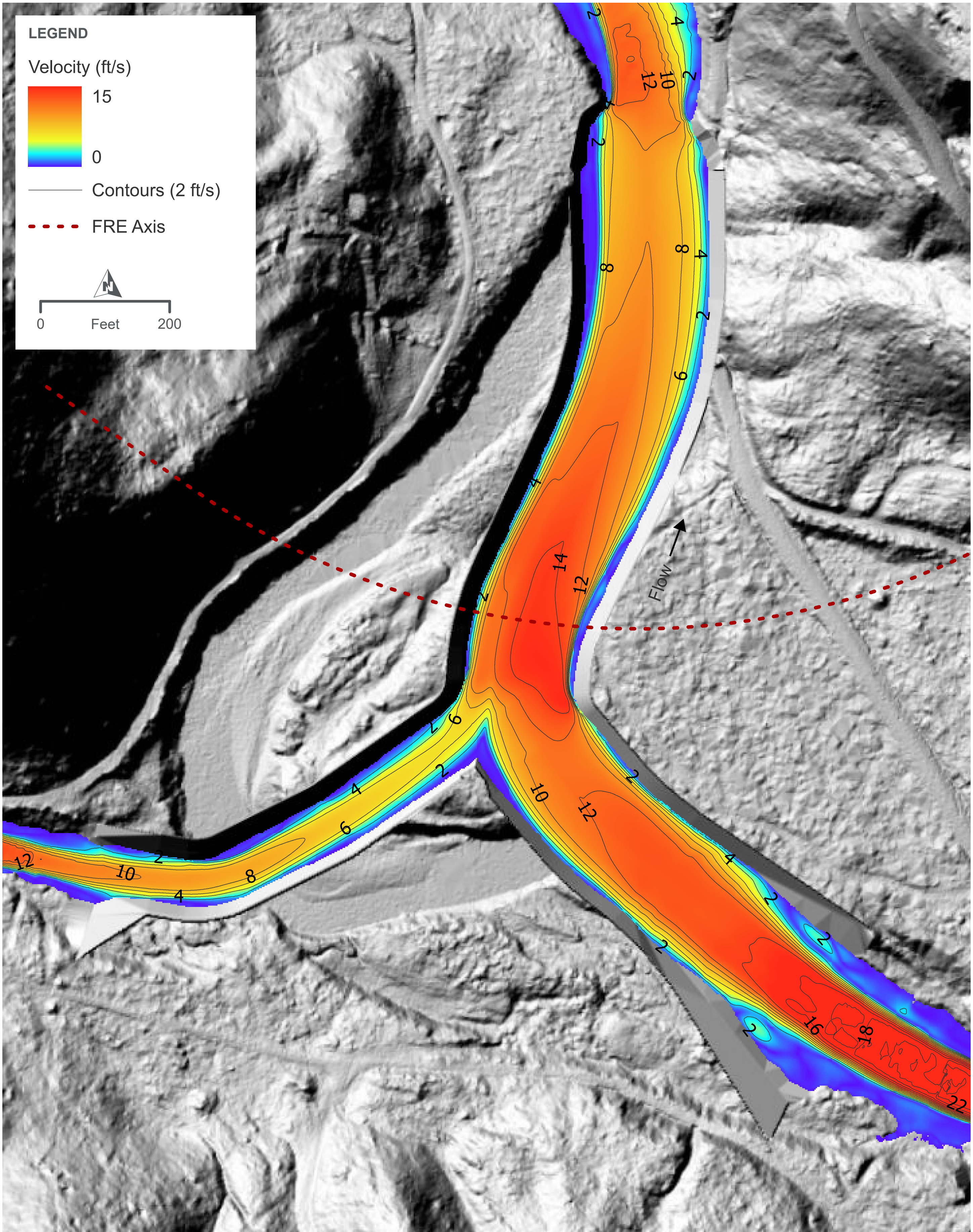
FIGURE A2



PROPOSED CONDITIONS - 10-YEAR

DEPTH

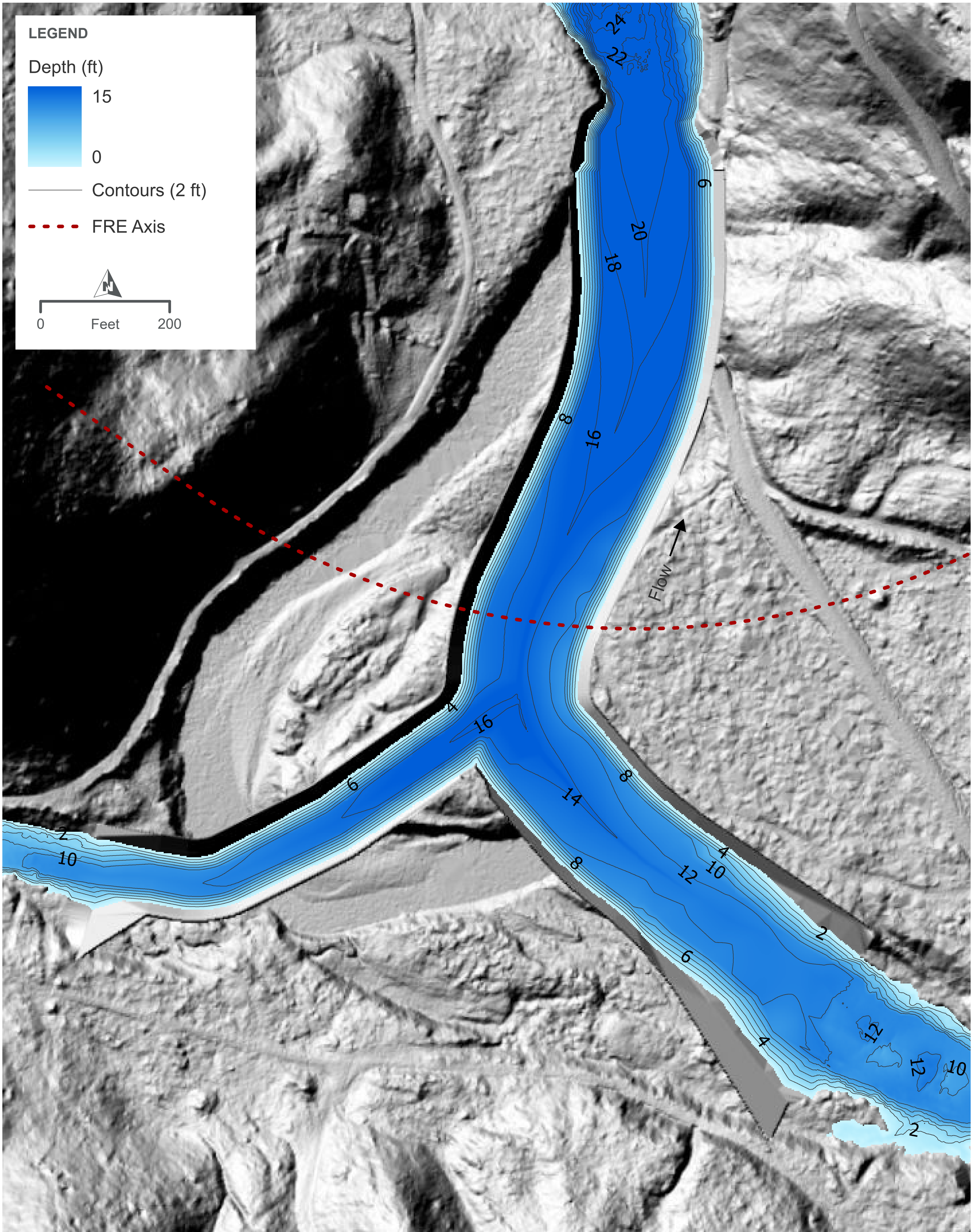
FIGURE A3



PROPOSED CONDITIONS - 10-YEAR

VELOCITY

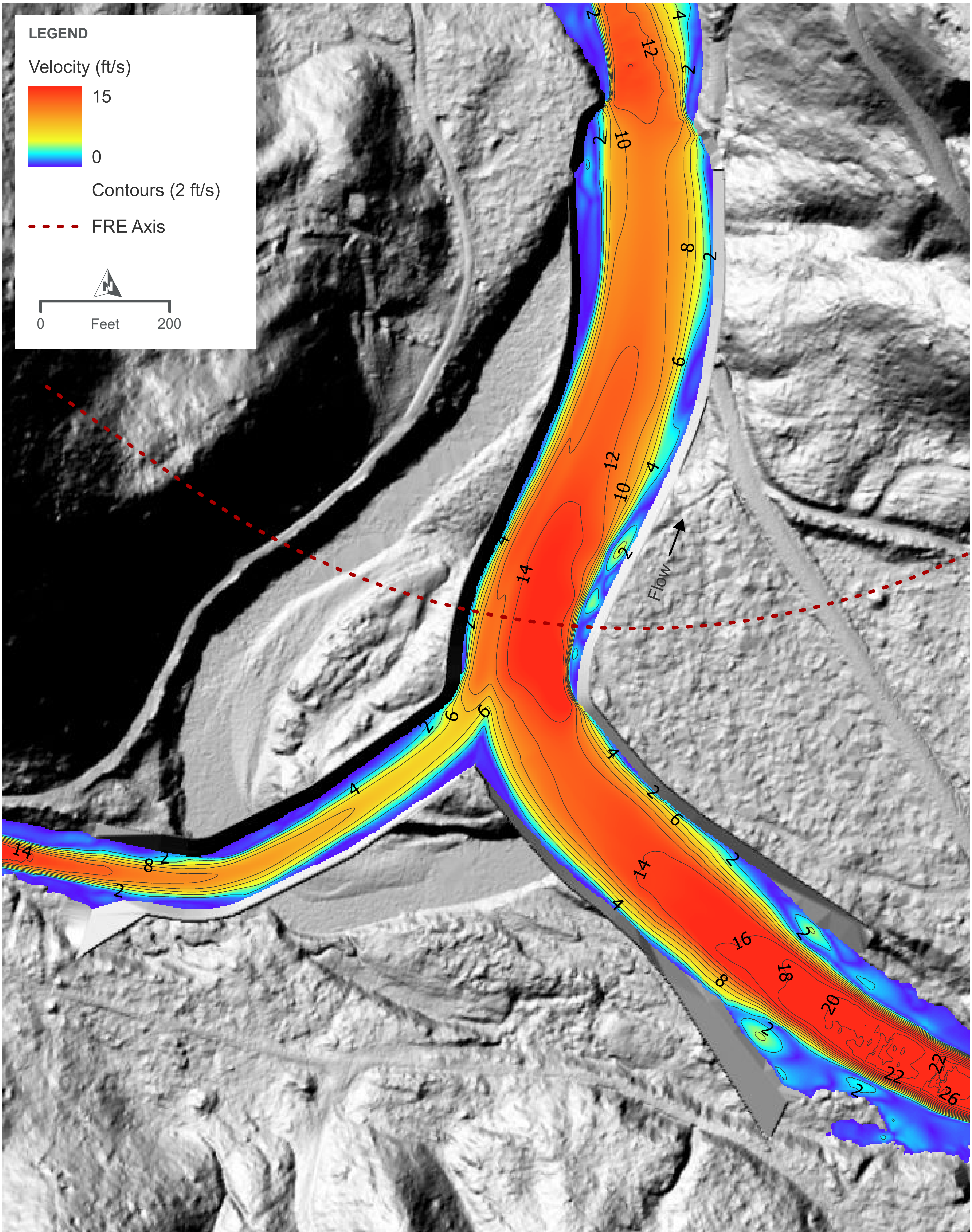
FIGURE A4



PROPOSED CONDITIONS - 25-YEAR

DEPTH

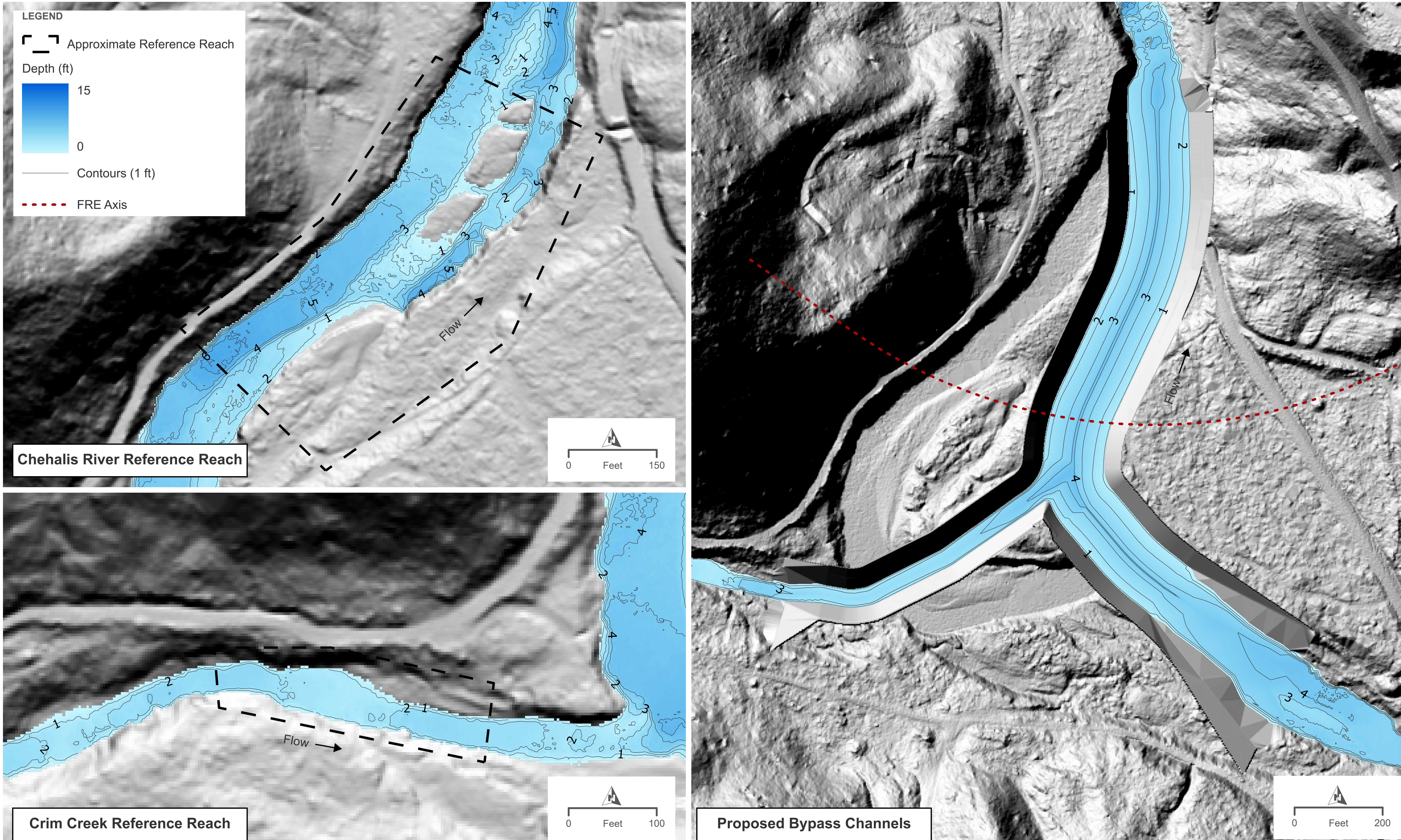
FIGURE A5



PROPOSED CONDITIONS - 25-YEAR

VELOCITY

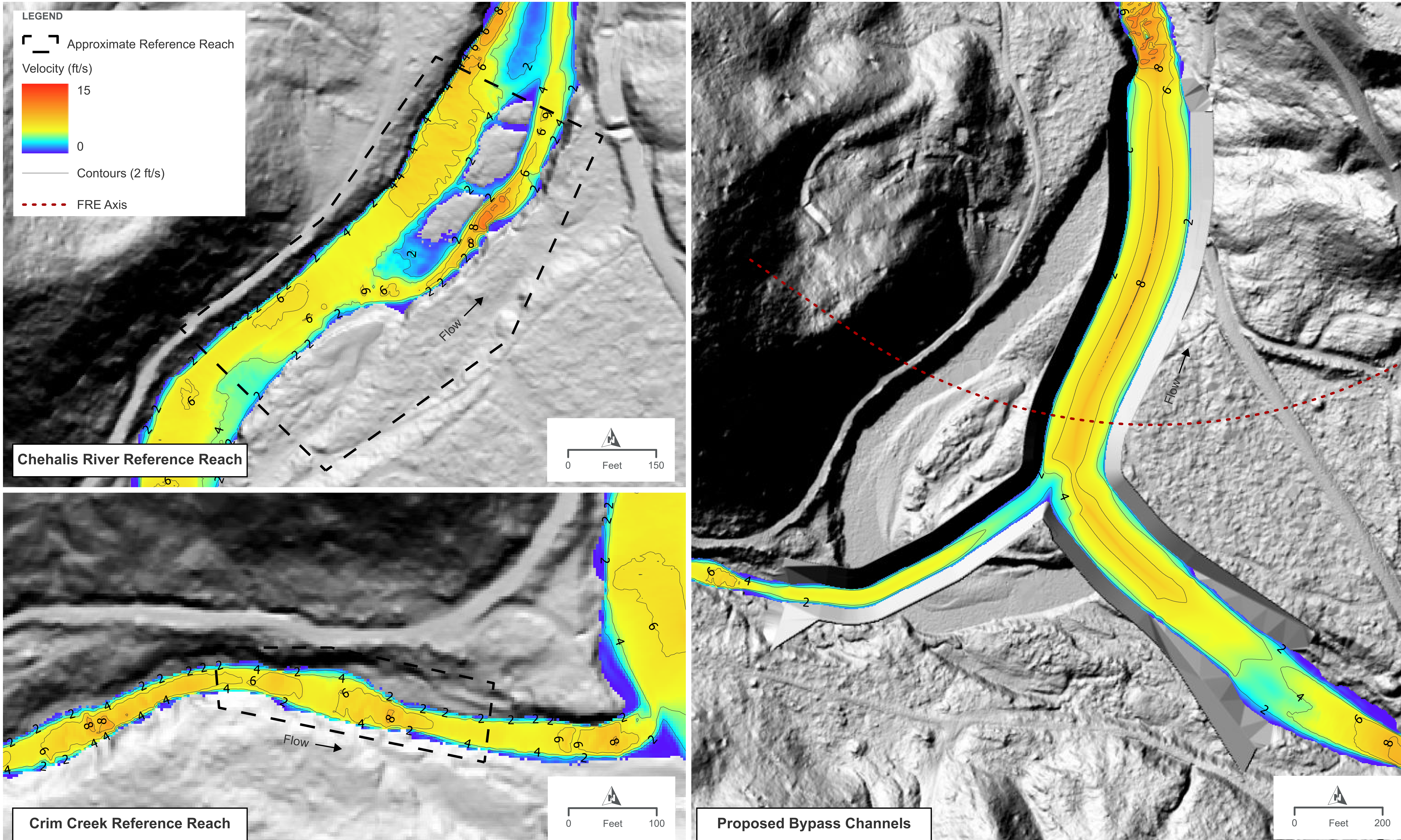
FIGURE A6



EXISTING VS PROPOSED - FISH PASSAGE HIGH DESIGN FLOW

DEPTH

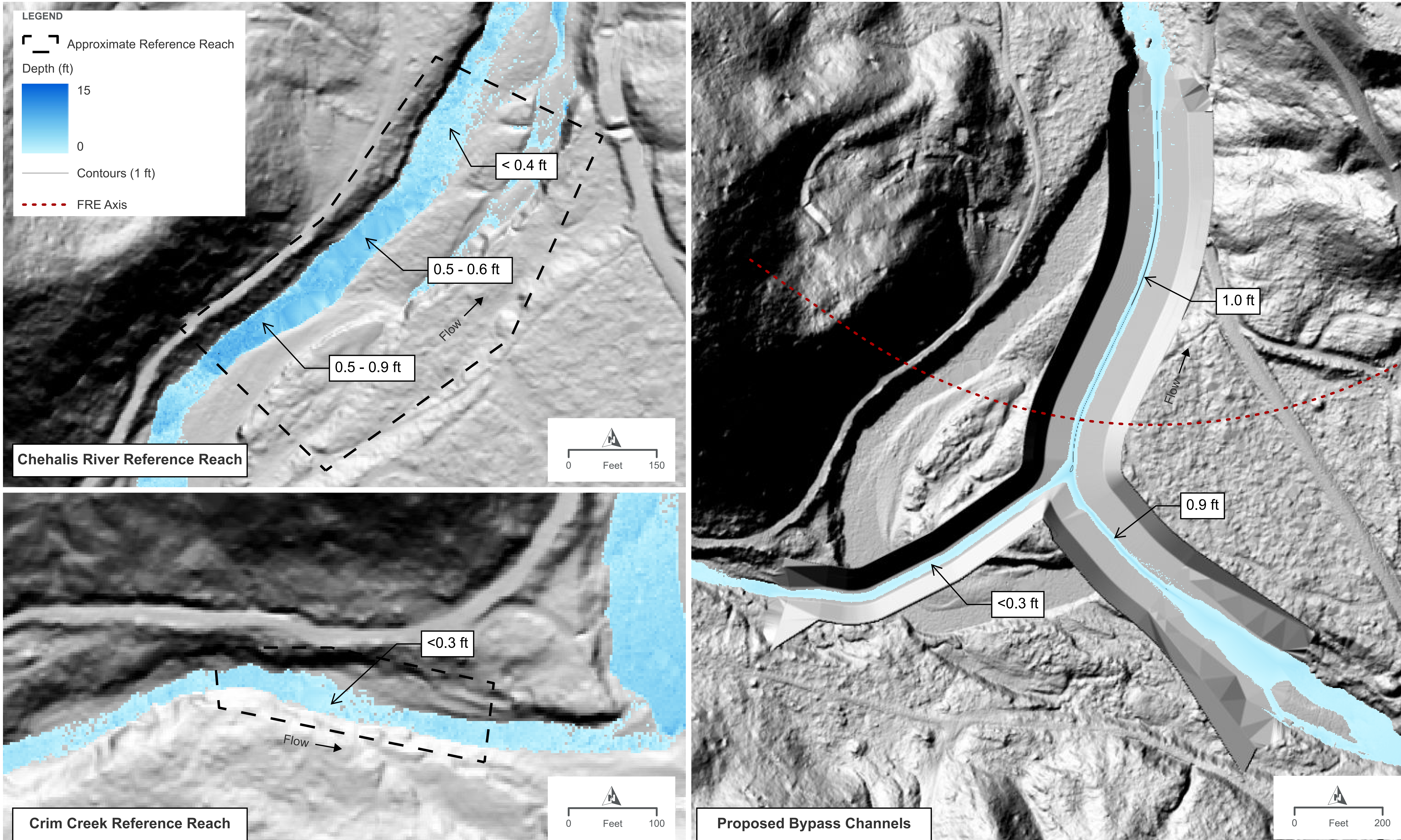
FIGURE A7



EXISTING VS PROPOSED - FISH PASSAGE HIGH DESIGN FLOW

VELOCITY

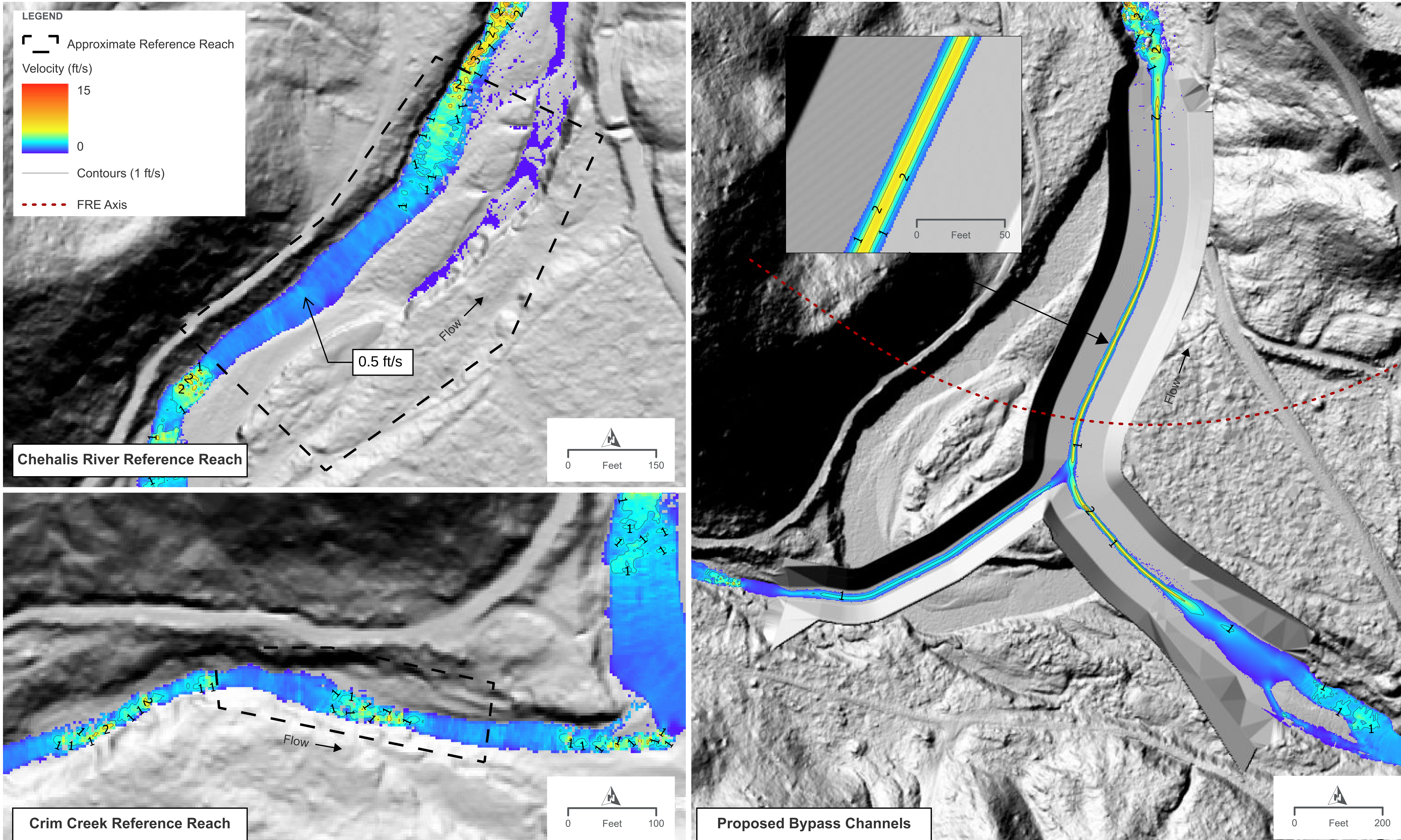
FIGURE A8



EXISTING VS PROPOSED - FISH PASSAGE LOW DESIGN FLOW

DEPTH

FIGURE A9



HDR

EXISTING VS PROPOSED - FISH PASSAGE LOW DESIGN FLOW

VELOCITY

FIGURE A10