Jan Road Levee Setback Project

Existing Conditions Hydraulic Model Update Memorandum

July 2021

KING COUNTY FLOOD CONTROL DISTRICT

King County

A Service Provider of
King County Flood Control District

Prepared for King County by
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1.0 INTRODUCTION

The King County River and Floodplain Management Section, as service provider to the King County Flood Control District, is evaluating alternatives to provide flood risk reduction for the Jan Road neighborhood through the Jan Road Neighborhood Improvements Project. The project area is located on the left and right banks of the Cedar River between River Miles 12.9 and 13.4, approximately two miles north of Maple Valley, and is shown in Figure 1. The project lies within the King County regulatory floodplain based on the review of the FEMA Flood Insurance Rate Map. This memorandum provides supporting information for the completed King County Flood Hazard Certification.

This project will include elements to improve floodplain functions and provide ancillary benefits to salmon spawning and rearing habitat. The following elements are included in the Project.

- Removal of the Jan Road levee from RM 13.13 to 13.36 (1,400 feet) to allow for channel migration into the project area
- Construction of a new setback levee approximately 2,500 feet long with scour protection.
- Excavation of a primary (low-flow) side channel
- Excavation of a secondary (high-flow) side channel
- Recontour the floodplain
- Installation of a larger culvert under SE 197th Place to convey overbank flows from Taylor Creek
- Placement of large wood at strategic locations to deflect bank flows, improve habitat, and provide floodplain roughness (left bank Cedar River, entrances to side channels, floodplain)
- Revegetation and riparian enhancement with site appropriate native plant materials.

This technical memorandum documents the hydrologic and hydraulic analyses conducted for the Project, including discussion of flood hazards, sediment transport, and scour potential, related to existing conditions. Additionally, the King County Ecological Restoration and Engineering Services Unit (ERES) is also, currently in the planning stages of the Rutledge-Johnson Floodplain Reconnection (Rutledge-Johnson) project, located directly upstream of the Jan Road Project (approximately between RM 13.1 and 13.6) (**Error! Reference source not found.**).

2.0 HYDROLOGIC ANALYSIS

Design flows used in the hydraulic model were developed for the Phase I model development analysis to derive the peak flows for the 1-, 2-, 5-, 10-, and 50-percent annual chance of flooding (ACF) events. Flow records from USGS gage 12117600 (Cedar River Below Diversion at Landsburg, WA), and King County gage 31i (Taylor Creek above 236th Ave SE) were used to develop inflow records for the one-dimensional CRIMP model, which was then used to route flows to the project site. Flood frequency analyses and flow durations were developed using the routed CRIMP flows to determine the hydrologic inputs to the hydraulic model. The hydrologic analysis is described in greater detail in the Hydrologic and Hydraulic Modeling Technical Memorandum (King County, 2020). Peak flow rates used in the analysis are listed in Table 1. Additionally, lower flows critical to habitat were also developed and described in King County, 2020. For his phase of the project, the Rearing flow—equal to the 50% exceedance discharge of 482 cfs in the Cedar River and 10 cfs in Taylor Creek during January-June--was included in the analysis.

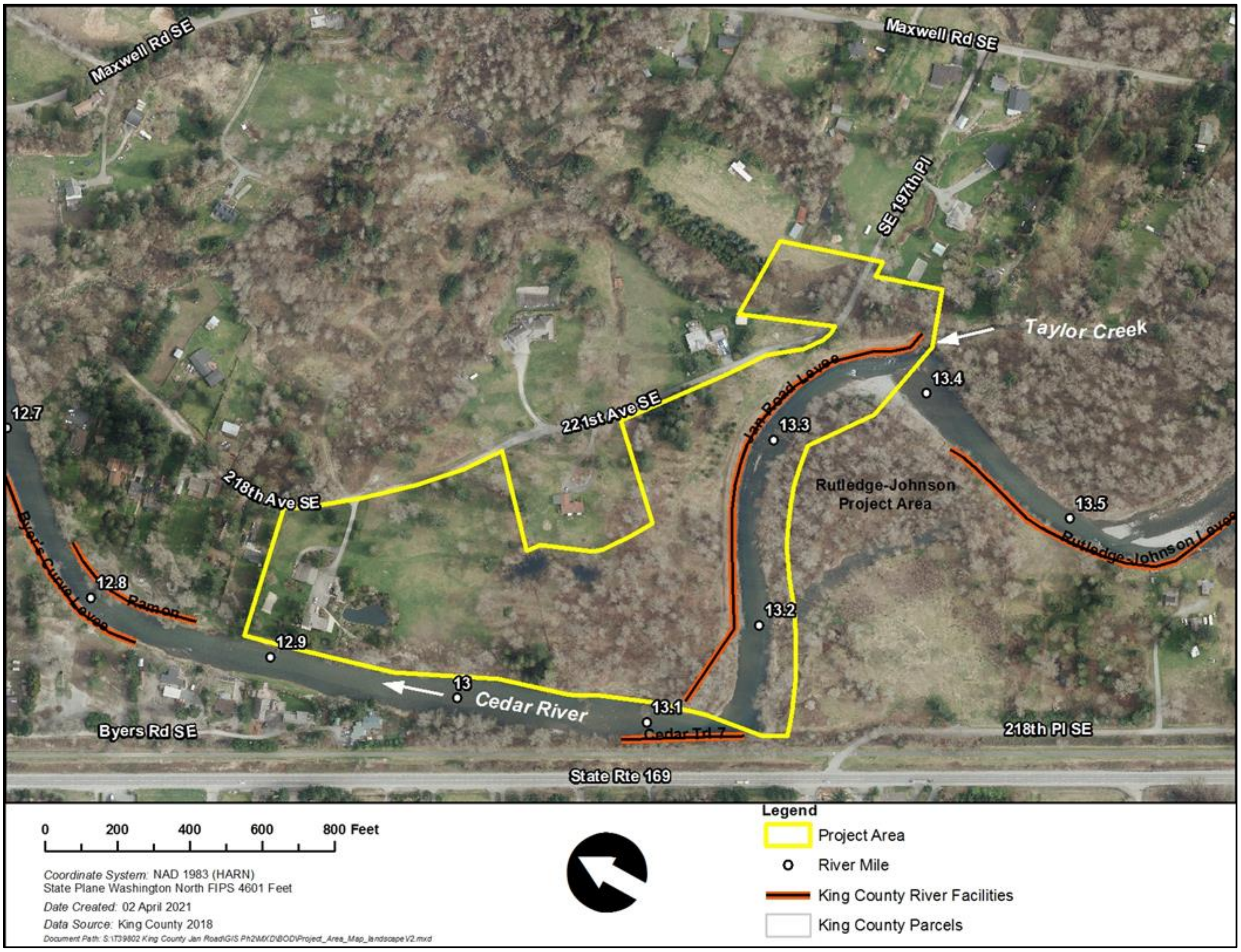

Figure 1. Project Area.

Table 1. Flood Flows and Annual Exceedance Probability for the Cedar River and Taylor Creek

Cedar River		Taylor Creek	
Annual Exceedance Probability	Estimated Flood Flow (cfs)	Coincident Annual Exceedance Probability	Coincident Flood Flow (cfs)
50%	2,419	59%	122
10%	4,817	17%	202
5%	6,016	8%	242
2%	7,420	3%	290
1%	9,443	0.75%	356

3.0 HYDRAULIC MODEL DEVELOPMENT AND CALIBRATION

Two-dimensional (2-D) hydraulic modeling was performed using HEC-RAS version 5.0.7 (USACE, 2016). Selection and development of the 2-D hydraulic model is discussed in greater detail in Hydrologic and Hydraulic Modeling Technical Memorandum (King County, 2020). The following section describes updates to the model for Phase 2 of the project.

3.1 HIGH FLOW CALIBRATION

The Phase I model topography as described in King County, 2020 was developed and calibrated using green LiDAR acquired in March of 2019. The model had been calibrated and validated using the 2019 topography. In February 2020, a large event approximately equal to a 2-percent annual exceedance probability occurred on the Cedar River. The range of flow from the February 2020 event encompasses most of the high flow design events for the Jan Road project and using this event for calibration ensures a good comparison between observed and predicted water surface elevation over the range of high flow events.

Green LiDAR was again acquired for the project area following the February 2020 event in late March/early April of 2020. To assess the impact of morphological changes from the February 2020 event, the model topography was updated using the newly acquired 2020 green LiDAR. The Phase I model mesh was updated to account for changes to the channel morphology caused by the February 2020 flood. No other changes were made to the model. This update was used to check the performance of the model and its ability to predict river stage at river stage monitoring locations. Results are shown in Figure 2 and summary statistics are shown in Table 2.

The model had been previously calibrated using the 2019 “pre-scoured” LiDAR surface. Predicted water surface elevations from the model were compared to observed data from King County gages, with the 5 minute gage data smoothed using a 15-minute moving average to reduce spikes in water surface elevation which are not indicative of typical flow characteristics. The 2019 calibration showed a good match between the rising limb and the peak of the hydrograph at Gages H1 and H2, but a poor match on the falling limb. Conversely, the 2020 calibration using the “post-event scoured” LiDAR surface showed a good match on the falling limb, but a poor match between the rising limb and the peak. Based on the 2019 and 2020 model performance, (2019 good match on the rising limb, 2020 good match on the falling limb) it was concluded that scour occurred in the channel near or slightly after the peak of the February 2020 event, likely sometime between midday on 2/8 and midday on 2/9, and that this scour is generally captured in the 2020 LiDAR. These results are consistent with the changes in the channel morphology between 2019 and 2020, as scour was observed from RM 13.8 to RM 14.1, and from RM 13.1 to RM 13.5. Some aggradation in mid channel bars was noted at the locations of both Gage H1 and H2, but the overall trend within the channel was degradation from scour.

Table 2. Summary Statistics and Error of Prediction for the High-flow Calibration Flood (February 2020)

King County Gage	Observed Peak Water Surface Elevation[a]	Simulated Peak Water Surface Elevation (feet NAVD88	Simulated Peak Water Surface Elevation (feet NAVD88	Average Error (feet)	**Maximum Error (feet)**
H1	283.52	282.83	0.29	-0.69	0.74
H2	267.67	267.19	0.37	-0.49	0.67
H4	254.49	254.39	-0.04	-0.10	0.37

a. Observed water surface elevations derived from 15-minute moving average.

Gage H1 is located at the upstream opening of the SR 169 bridge crossing along the Cedar River at approximately RM 14.04, within a region that showed overall scour through the bridge and in the upstream channel between 2019 and 2020. Peak stage was estimated to be about 0.5 feet below observed stage at this location using the 2019 LiDAR, and about 0.7 feet below observed stage using the 2020 LiDAR.

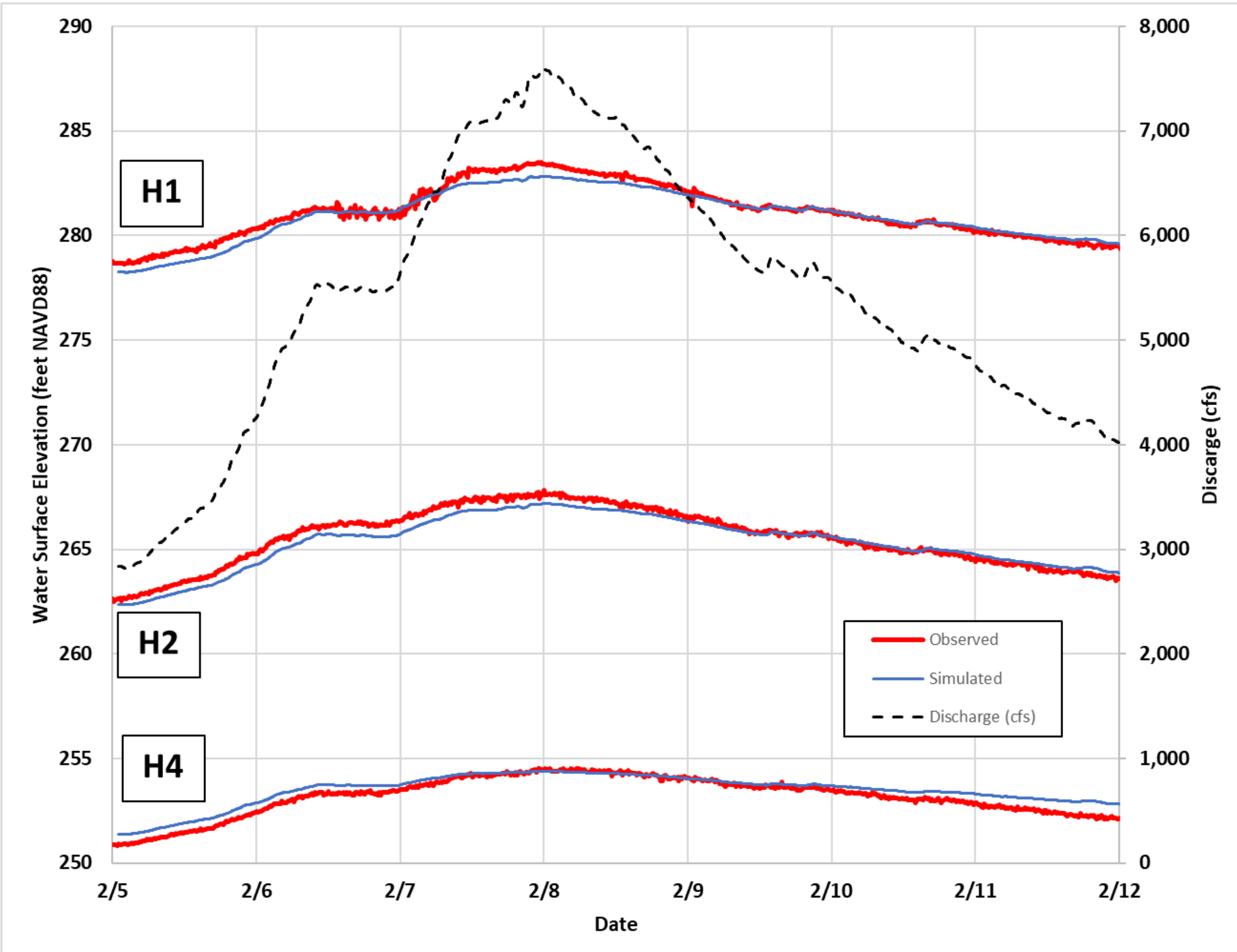

Figure 2. Predicted vs Observed flow at Gages H1, H2, and H4

The same behavior was noted at Gage H2, located along the Jan Road levee at approximately RM 13.38 within a region that showed overall scour in the channel between 2019 and 2020, with the rising limb and peak matching well using the 2019 LiDAR and the falling limb matching well using the 2020 LiDAR. Peak stage was estimated to be about 0.1 feet below observed stage at this location using the 2019 LiDAR, and about 0.5 feet below observed stage using the 2020 LiDAR. This indicates that scour or other changes in the channel bathymetry occurred during the event, but that the defined roughness values are appropriate for each surface.

Gage H3, at RM 13.1 along the CRT7 revetment, failed early in the hydrograph and so there was very little data to use for calibration. The model results computed using the 2020 LiDAR show a drop of about 0.2 feet for the peak stage of the hydrograph. The stage hydrograph for the 2020 model update was lower than the 2019 predicted hydrograph throughout the hydrograph, further indicating a change in the channel bathymetry consistent with the overall pattern of channel changes observed between the 2019 and 2020 surfaces in this vicinity.

Gage H4, at approximately RM 12.83 shows a different trend using the 2020 LiDAR. Predicted stage using the 2020 model was slightly higher than the predicted stage using the 2019 LiDAR surface on the rising and falling limbs of the hydrograph. The LiDAR surfaces show that the reach from RM 12.7 to RM 12.9 was generally slightly aggradational between 2019 and 2020, which may account for this difference. Additionally, a possible explanation for this behavior is that flows overtopped the left bank, approximately 200 feet upstream of the H4 gage, during the February 2020 event. Damage to the bank at this location was repaired prior to the 2020 LiDAR acquisition, thus the LiDAR elevations are representative of the repaired bank condition. Given the repaired ground condition,

overtopping does not occur at this location in the model, resulting in more flow within the channel compared to observations, ultimately leading to higher estimates of water surface elevations than observed. The gage still shows a good match at the peak of the hydrograph for both the 2019 and 2020 LiDAR surfaces.

In summary:

- The changes in the model calibration are generally consistent with the changes seen in the channel morphology from 2019 to 2020.
- Gages H1 and H2 showed a better match on the falling limb using the 2020 LiDAR, which is consistent with the overall general trend towards scour in their respective reaches between 2019 and 2020.
- At H4, the simulated water surface elevations are slightly higher using the 2020 LiDAR, which is consistent with the overall trend towards aggradation in the channel from RM 12.5 to RM 12.7 between 2019 and 2020.
- Considering the good match between observed and predicted WSE on the falling limb of the hydrograph at Gages H1 and H2, and the good match at the peak of the hydrograph at H4, it was determined that the defined roughness values for the high flow model were still appropriate for use with the 2020 LiDAR surface.

3.2 LOW-FLOW VALIDATION

The low-flow HEC-RAS model was also updated with the 2020 Lidar to account for changes to the channel morphology caused by the February 2020 flood and evaluated for its ability to predict river stage under low flow conditions at three river stage monitoring locations (Stage Gage H1 was not working during this event). Similar to the calibration effort described above, the only difference between the models for these simulations using the 2019 LiDAR and 2020 LiDAR was the topography. The one-week period starting on September 21, 2020 was selected as the representative hydrograph for the low-flow validation. Streamflow ranged from 250 cfs to 650 cfs during this period. This range includes the spawning flow (350 cfs) and rearing flow (482 cfs). Results are shown in Table 3 and Figure 3.

Table 3. Summary Statistics and Error of Prediction for the Low-flow Validation Period (September 2020)

King County Gage	Observed Peak Water Surface Elevation	Simulated Peak Water Surface Elevation (feet NAVD88	Simulated Peak Water Surface Elevation (feet NAVD88	Average Error (feet)	**Maximum Error (feet)**
H2	258.65	258.60	0.18	-0.05	0.31
H3	254.53	254.51	0.11	-0.03	0.23
H4	248.43	248.60	0.00	0.17	0.21

In summary:

- The predicted stage computed by the low-flow HEC-RAS model showed good comparison between the predicted and observed stage at all gage locations.
- The predicted stage was slightly higher prior to the peak of the event compared to existing conditions for all gage locations.
- The low-flow HEC-RAS model slightly underpredicts stage at gage location H2 and H3 when flows are below 600 cfs but is able to performs well when flow is at 600 cfs.
- At gage location H4, the model slightly overpredicts stage when flow is below 300 cfs and slightly underpredicts stage above 500 cfs. The low-flow model is able to accurately predict stage between 300 and 500 cfs.
- Stage predicted by the low-flow HEC-RAS model is within calibration tolerance at all gage locations and flow levels evaluated.

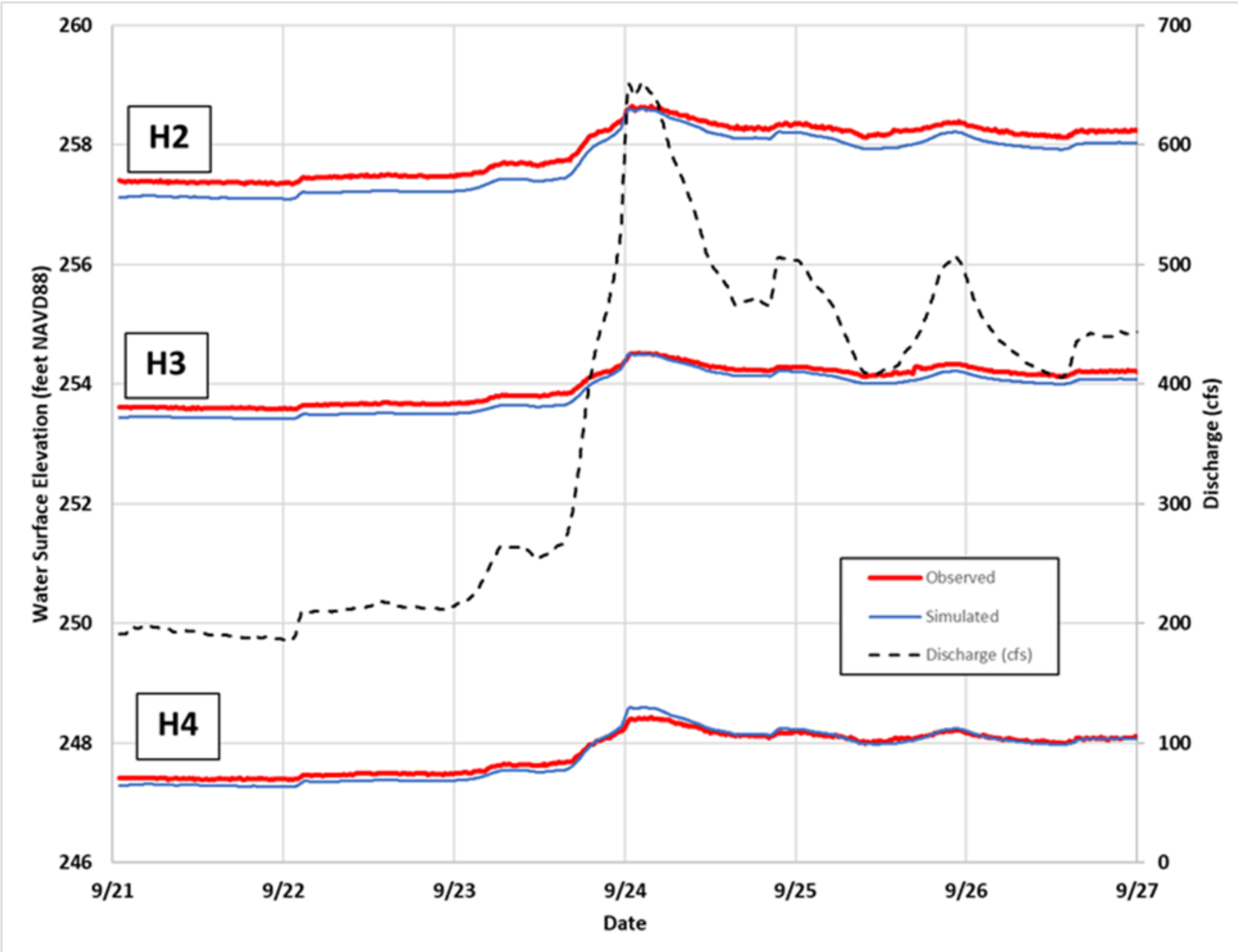

Figure 3. Predicted vs Observed flow at Gages H2, H3, and H4

4.0 EXISTING CONDITIONS HYDRAULIC MODEL RESULTS

The updated HEC-RAS model was used to simulate the five flow conditions described in Section **Error! Reference source not found.**. The resulting maximum depth, velocity, and critical grain size for each event were processed. Depth and velocity are discussed in this report. Critical grain size is presented and discussed in the *Jan Road Existing Condition Geomorphic Assessment Report*. Complete model results are shown in Appendix A of this technical memorandum.

4.1 DEPTH AND VELOCITY

The range of flow conditions analyzed is bracketed by the 50 percent and 1 percent ACF where the 50 percent flow is contained within the Cedar River channel and the 1% percent flow results in extensive overbank flooding over the majority of the river corridor. The computed flood inundation depth for the 50 percent and 1 percent ACF event is shown in Figure 4 and Figure 5. Maximum velocities for the 50 percent and 1 percent ACF event are shown in Figure 6 and Figure 7. A comprehensive discussion of the hydraulic characteristics for existing conditions is provided in Section 4.4. Plots of inundation depth and velocity for all events analyzed are provided in Appendix A of this technical memorandum, 2020.

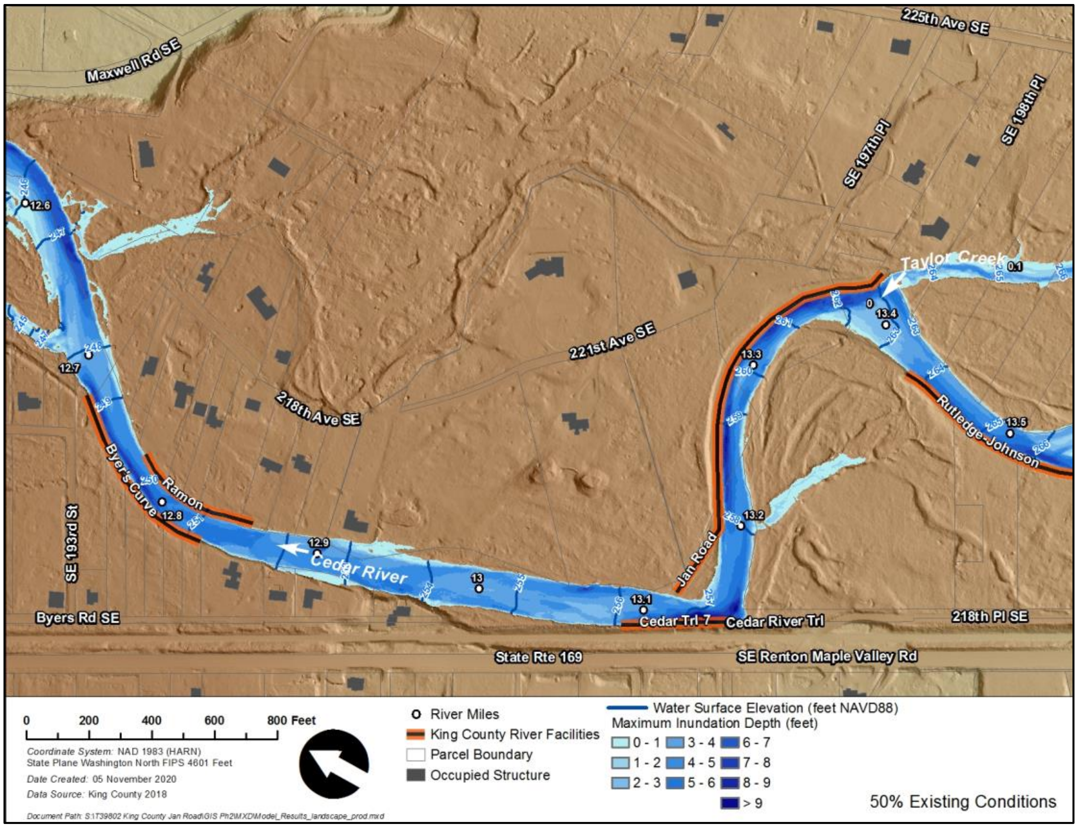

Figure 4. Existing Conditions, 50 Percent ACF Maximum Depth

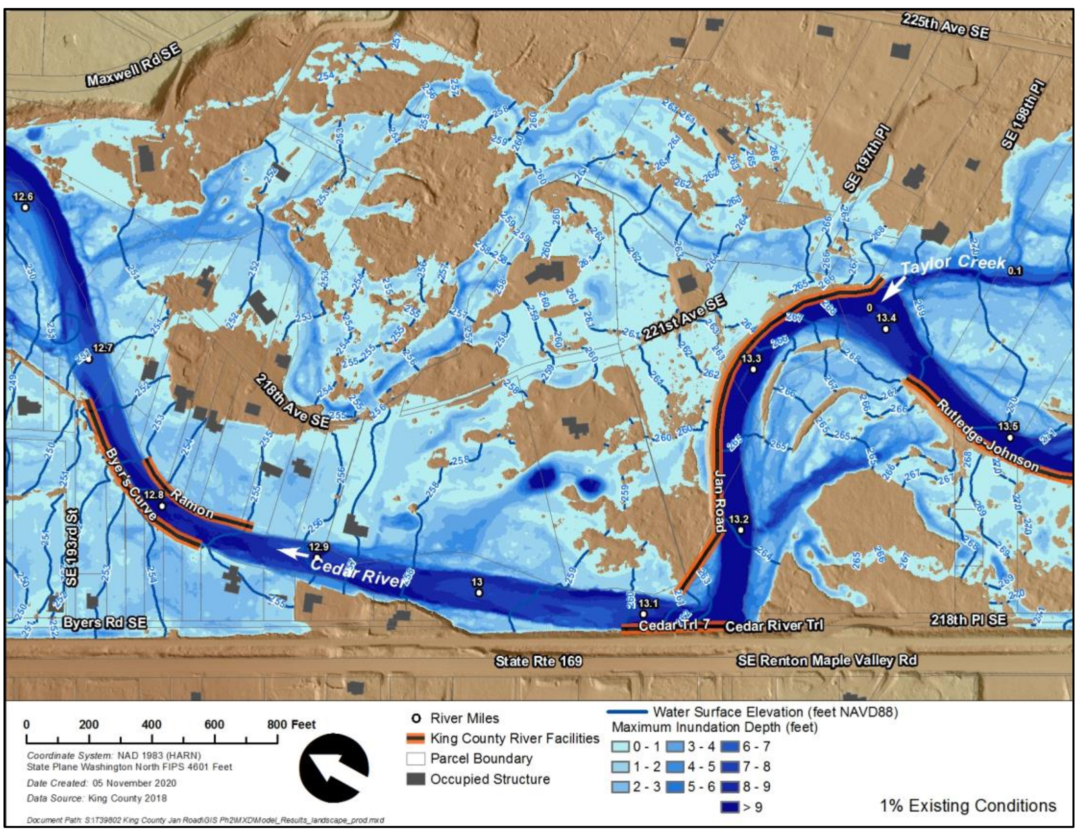

Figure 5. Existing Conditions, 1 Percent ACF Maximum Depth

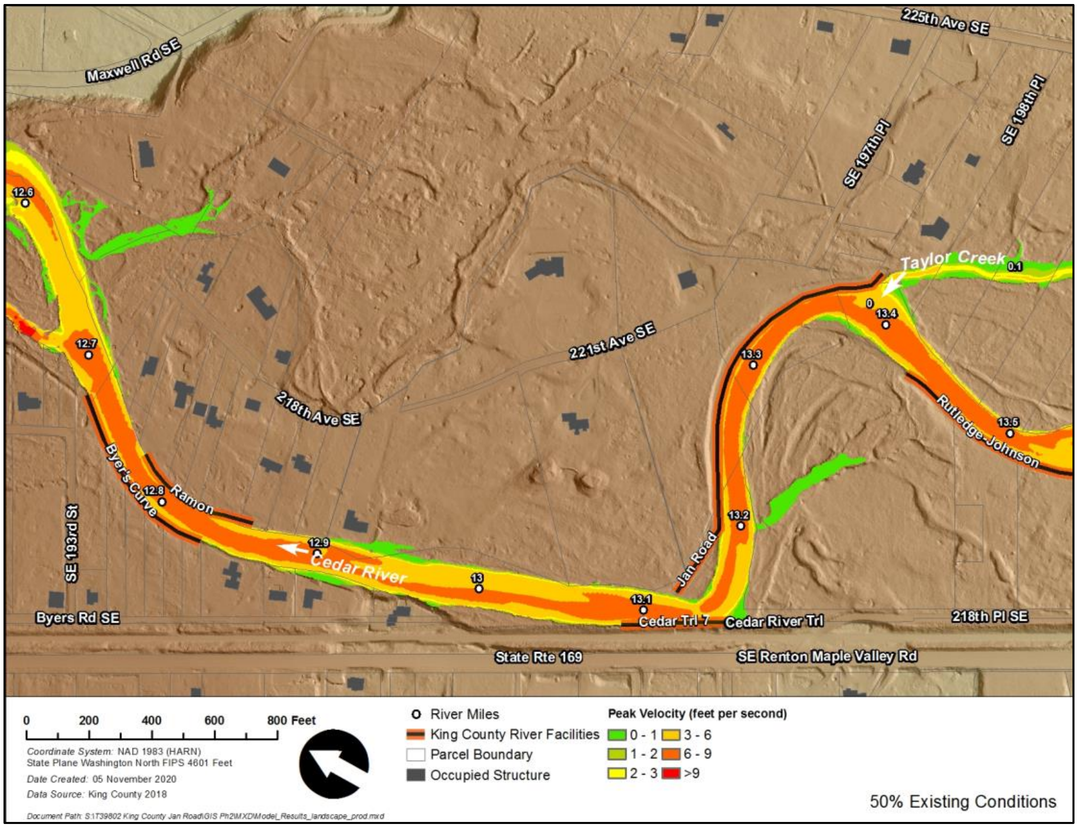

Figure 6. Existing Conditions, 50 Percent ACF Maximum Velocity

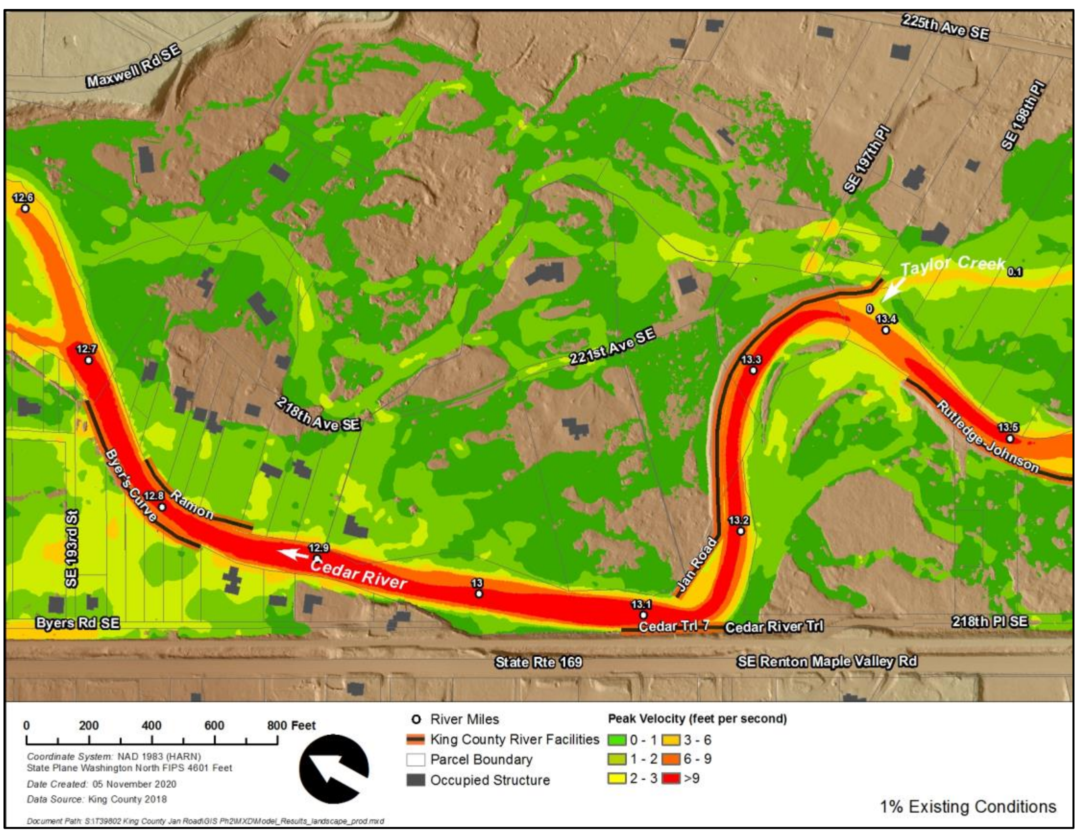

Figure 7. Existing Conditions, 1 Percent ACF Maximum Velocity

4.2 FLOW DISTRIBUTION

The flow distribution through the project area was computed at the locations identified in Figure 8 to characterize the flow distribution in the Cedar River channel and floodplain during flood conditions. The flow distribution is quantified in Table 4.

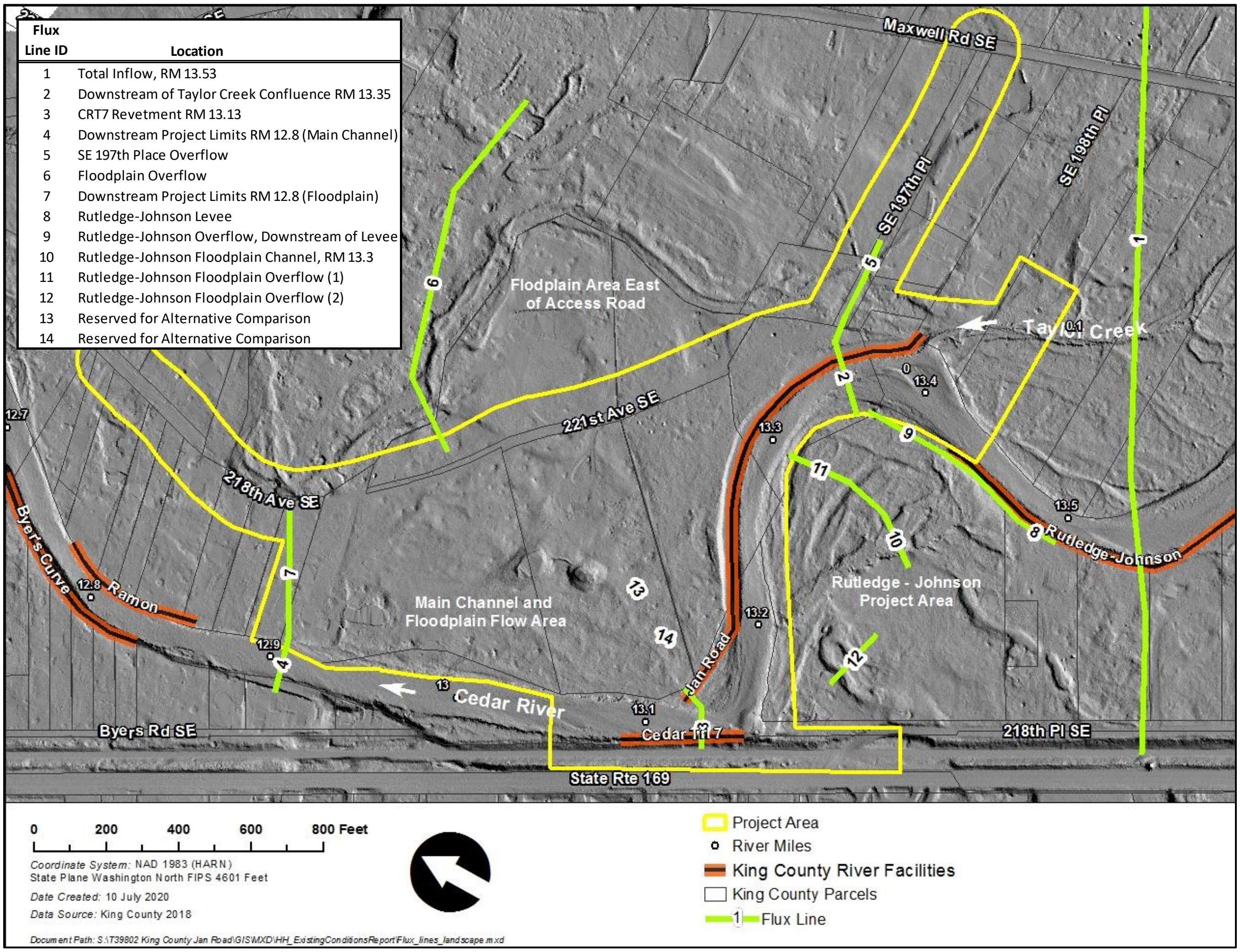

Figure 8. Flow Flux Line Locations for Peak Flow Distribution.

Table 4. Flow Distribution For Peak Flood Events

Flow Flux Line Location[a]	Peak Flow (cfs) 50% ACF	10% ACF	5% ACF	2% ACF	1% ACF
Inflow					
1. Total Inflow, RM 13.53	2,541	5,109	6,256	7,823	9,721
Main Channel and Floodplain Flow					
2. Downstream of Taylor Creek Confluence RM 13.35	2,541	5,018	6,102	7,047	7,803
3. CRT7 Revetment RM 13.13	2,541	5,018	6,221	7,615	9,203
4. Downstream Project Limits RM 12.8 (main channel)	2,541	5,018	6,187	7,415	8,623
7. Floodplain at Downstream Project Limits RM 12.8 (floodplain)	0	0	34	186	528
Floodplain East of Access Road					
5. SE 197th Place Overflow	0	0	35	208	592
6. Floodplain Overflow	0	0	34	203	561
Rutledge- Johnson					
8. Rutledge-Johnson Levee	0	0	0	0	1
9. Rutledge-Johnson Overflow Downstream of Levee	0	0	119	568	1,219
10. Rutledge-Johnson Floodplain Channel RM 13.3	0	0	21	206	690
11. Rutledge-Johnson Floodplain Overflow (1)	0	0	93	320	583
12. Rutledge-Johnson Floodplain Flow RM (2)	0	0	0	0	11

a. See Figure 8 for peak flow reporting locations.

4.3 WATER SURFACE ELEVATION

The average water surface elevation was computed at each of the CRIMP sections for evaluating project impacts during the alternatives analysis phase. and is shown in in the no-rise analysis. The average water surface elevation is computed as the incremental summation of the weighted water surface elevation over the wetted surface of the CRIMP section divided by the length of wetted surface. Average water surface elevation increase would be limited to 0.005 feet for the no-rise analysis that would be performed as part of the flood certification performed during the design phase. Table 5 shows the average water surface elevation at each section.

Table 5. Average Water Surface Elevation at CRIMP Section

CRIMP Section[a]	Average Water Surface Elevation (feet NAVD88)		CRIMP Section[a]	Average Water Surface Elevation (feet NAVD88)
1475.7	299.882		1370.4	274.113
1467.1	296.577		1355.9	270.934

CRIMP Section[a]	Average Water Surface Elevation (feet NAVD88)		CRIMP Section[a]	Average Water Surface Elevation (feet NAVD88)
1457.5	295.545		1346.3	268.528
1444.3	291.421		1338.9	266.338
1430.3	289.155		1330.1	263.321
1420.5	288.033		1318.6	261.281
1411.8	286.336		1307.3	259.940
1406.3	286.300		1296.9	257.307
1404.3	283.105		1286	253.532
1401.2	282.409		1278.1	252.133
1389	278.687		1266.6	250.085
1381.7	276.940		1247.9	248.738

4.4 SUMMARY OF INSTREAM AND FLOODPLAIN IMPACTS

Overflow to SE 197th Place at TM 13.35 and Floodplain Area East of Access Road

- Floodplain inundation, leading to the greatest depths of roadway overtopping, occurs starting at the 5 percent ACF event, with high river stage at the Taylor Creek confluence (upstream end of Jan Road Levee) causing flow within Taylor Creek to overtop its banks, leading to overtopping of SE 197th Place. These flows are then conveyed northerly via an existing floodplain channel and return to the Cedar River at about RM 12.65.
- At about the 10% ACF, backwater from the Cedar River begins to inundate the floodplain landward of the Access Road, near the outlet of the existing floodplain channel (RM 12.65).
- For the 1% ACF, a portion of the overflow returns back to the Cedar River floodplain at the south end of 220th Avenue SE (near the SE 197th Place culvert crossing) over the Access Road and in the area between the Access Road and the landward side of the Jan Road Levee.
- At the location of overtopping of SE 197th Place, maximum flow velocities are about 5 feet per second (fps), during the 1% ACF.
- Total volume of overflow across SE 197th Place ranges from 1 to 6 percent of the flow in the Cedar River for the 5 and 1% ACF, respectively.

Cedar River Channel and Right Bank floodplain within the Project Limits (RM 12.9 to RM 13.45)

- The Jan Road levee restricts flow in the Cedar River resulting in a backwater condition upstream of the levee which extends to Taylor Creek causing the creek to overflow its banks at about the 5% ACF.
- For the 1% ACF, flow on the right bank floodplain landward of the levee is primarily due to a return of a portion of the overflow at SE 197 Place from the landward side of the Access Road.
- Downstream of the Jan Road levee, the Cedar River starts to inundate the right bank floodplain in the project area at about the 10% ACF. However, conveyance of flood flows through the floodplain does not occur until about the 5% ACF when floodwaters breakout of the Cedar River channel immediately downstream of the Jan Road Levee. Overbank flows at this location are the primary source of flow in the right bank floodplain at the downstream project limits (RM 12.8).

- Floodwaters from the Cedar River floodplain overtop the Access Road at about RM 12.9 for the 5% and larger ACF.

CRT7 Revetment

- Streamflow diversion at the SE 197th Place reduces the flow rate (and velocity) at the CRT7 Revetment however this reduction is offset for larger events by additional flow contribution from the Rutledge Johnson area.
- Flow velocities for the 1% ACF typically range from 6 to 10 fps in the main channel, but increase to greater than 10 fps at the bend at the CRT7 Revetment (RM 13.15). The flow velocities at this location are the highest within the model domain, with peak velocities over 14 fps.
- Flow velocities in the pool at the bend immediately upstream of the CRT7 revetment are below 1 foot per second for all events. However, this buffering effect on approach flows is limited to the pool itself and does not extend downstream along the CRT7 revetment.

Rutledge-Johnson Project Area

- At the 5% ACF, flow depth exceeds the bank height and overflow into the left bank floodplain downstream of the Rutledge – Johnson Levee.
- Total volume of Cedar River flows through the Rutledge – Johnson site range from 2 to 13 percent for the 5 and 1% ACF, respectively.

5.0 SCOUR ANALYSIS

An analysis was conducted to quantify scour depths for existing conditions along the Cedar River in the project area between RM 12.8 and RM 13.4. This reach includes the bend along the Jan Road Levee at RM 13.31 and the bend at the Cedar River Trail at RM 13.16 (see Figure 1**Error! Reference source not found.**). The scour analysis results will be used to support the design of in-channel features. An initial scour analysis was performed in Phase I that considered long-term scour trends, general scour and local scour (bend, contraction, abutment and pier scour).

A description of that approach, including the calculated scour elevation, as well as equations and variables used in the scour analysis, are documented in Appendix B of King County, 2020. In this assessment, bend scour was re-calculated due to the changes in the channel resulting from the February 2020 event. The resulting scour elevations are shown in Table 6 and Table 7. Table 6 excludes the Safe Design Curve method of determining scour elevation, as they can be overly conservative for a slope stability analysis. Table 7 include the Safe Design Curves, which will be used to inform the levee design. The results from the Phase I analysis are also included, and show that despite the channel changes, the estimated scour elevation changed by no more than 0.2 ft.

Table 6. Estimated Bend Scour Depth for Slope Stability Analysis

Bend ID	Location (RM)	1% ACF WSE (feet NAVD88)	Channel Invert Elevation (feet NAVD88)	Scour Depth / Scour Depth Below Channel Bed (feet) [a]			Phase 2 Scour Elevation[b] (feet NAVD88)	Phase 1 Scour Elevation[b] (feet NAVD88)
				Minimum	Maximum	Average		
B1	Jan Road Levee (13.31)	266.1	253.5	12.2 / 0.0	17.4 / 4.8	14.4 / 1.8	251.7	251.5
B2	CRT 7 Revetment (13.16)	263.8	248.4	17.9 / 2.5	24.4 / 9.0	21.1 / 5.7	242.7	242.8

a. Excludes SDC equations.

b. Scour elevation based on average scour depth value.

Table 7. Estimated Bend Scour Depth Analysis for Facility Design

Bend ID	Location (RM)	1% ACF WSE (feet NAVD88)	Channel Invert Elevation (feet NAVD88)	Scour Depth / Scour Depth Below Channel Bed (feet)			Phase 2 Scour Elevation[a] (feet NAVD88)	Phase 1 Scour Elevation[a] (feet NAVD88)
				Minimum	Maximum	Average		
B1	Jan Road Levee (13.31)	266.1	253.5	17.4 / 4.8	23.4 / 10.8	20.5 / 7.9	245.6	245.6
B2	CRT 7 Revetment (13.16)	263.8	248.4	21.3 / 5.9	37.9 / 22.6	30.4 / 15.0	233.4	233.5

a. Scour elevation based on average scour depth value.

APPENDIX A. HYDRAULIC MODEL RESULTS

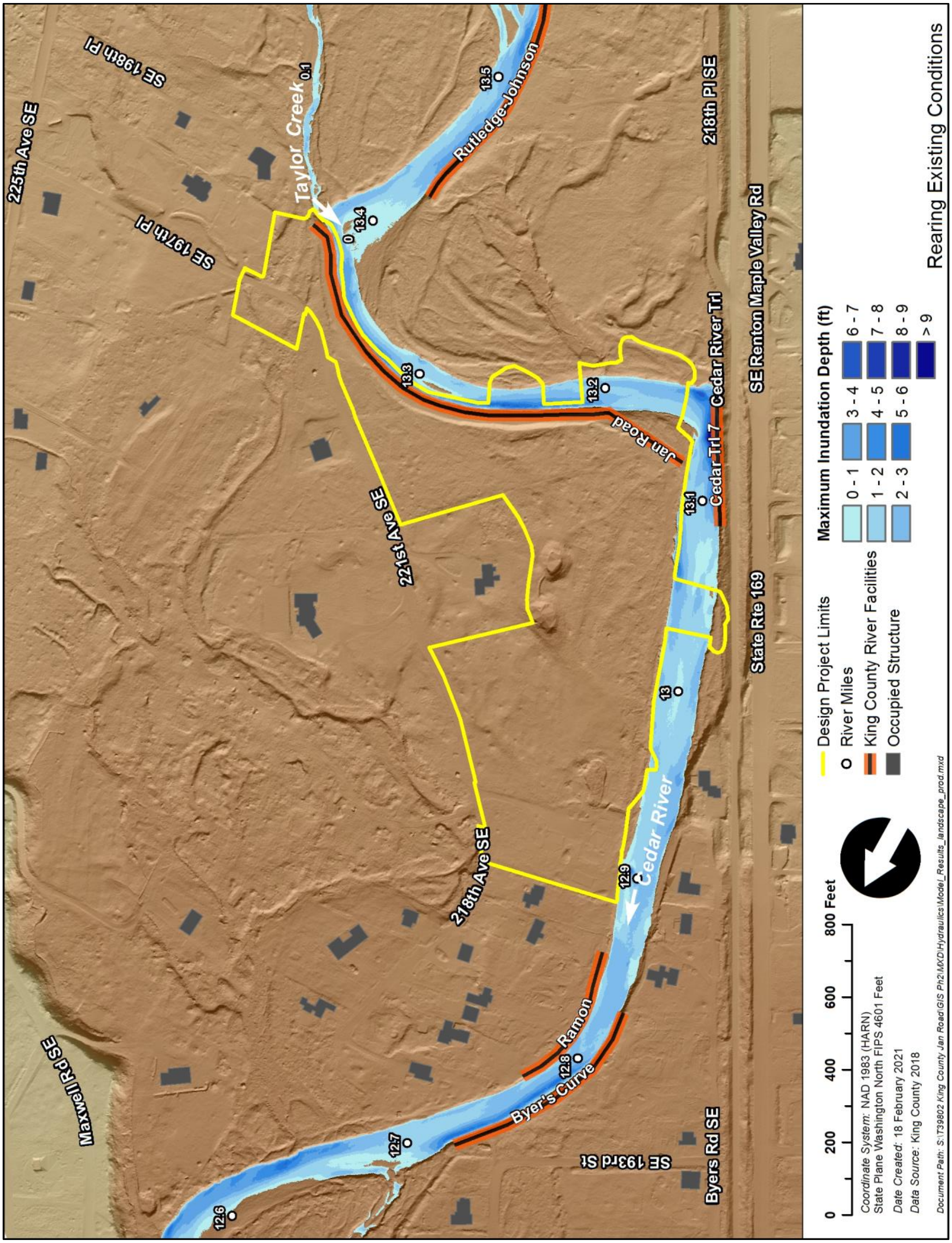

Figure A-1. Existing Conditions, Rearing Flow, Maximum Depth (Q = 492 cfs)

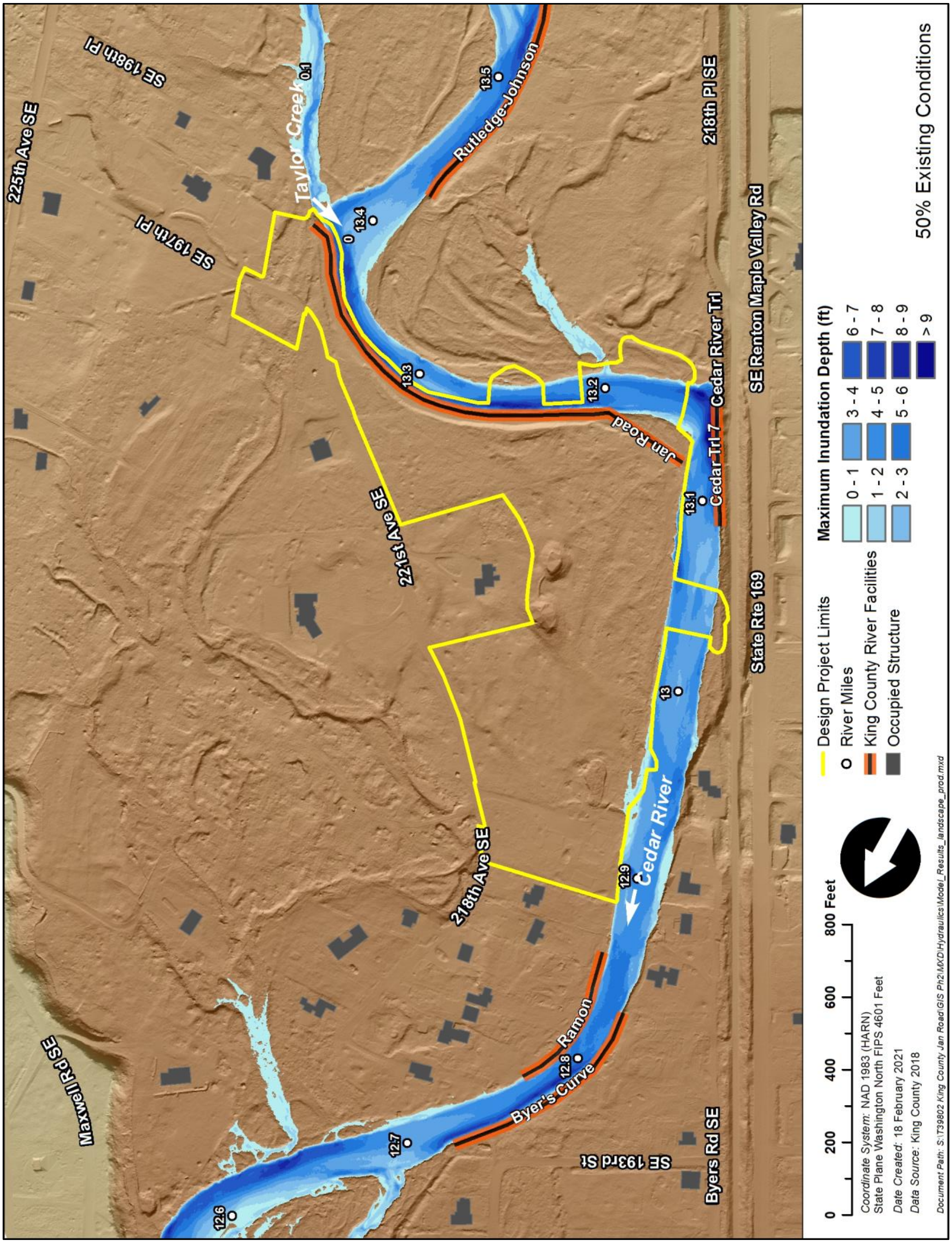

Figure A-2. Existing Conditions, 50% ACF, Maximum Depth (Q = 2,540 cfs)

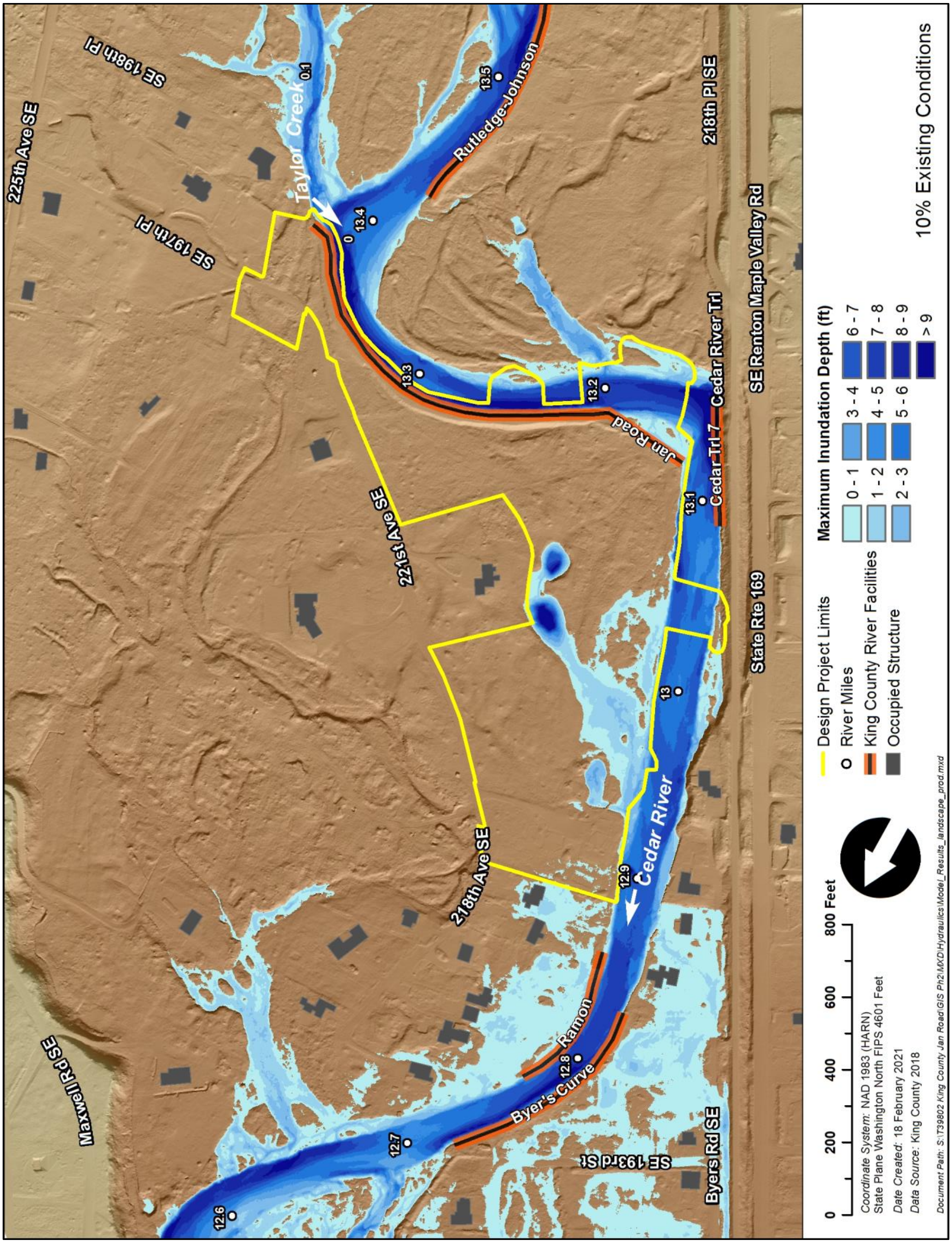

Figure A-3. Existing Conditions, 10% ACF, Maximum Depth (Q = 5,020 cfs)

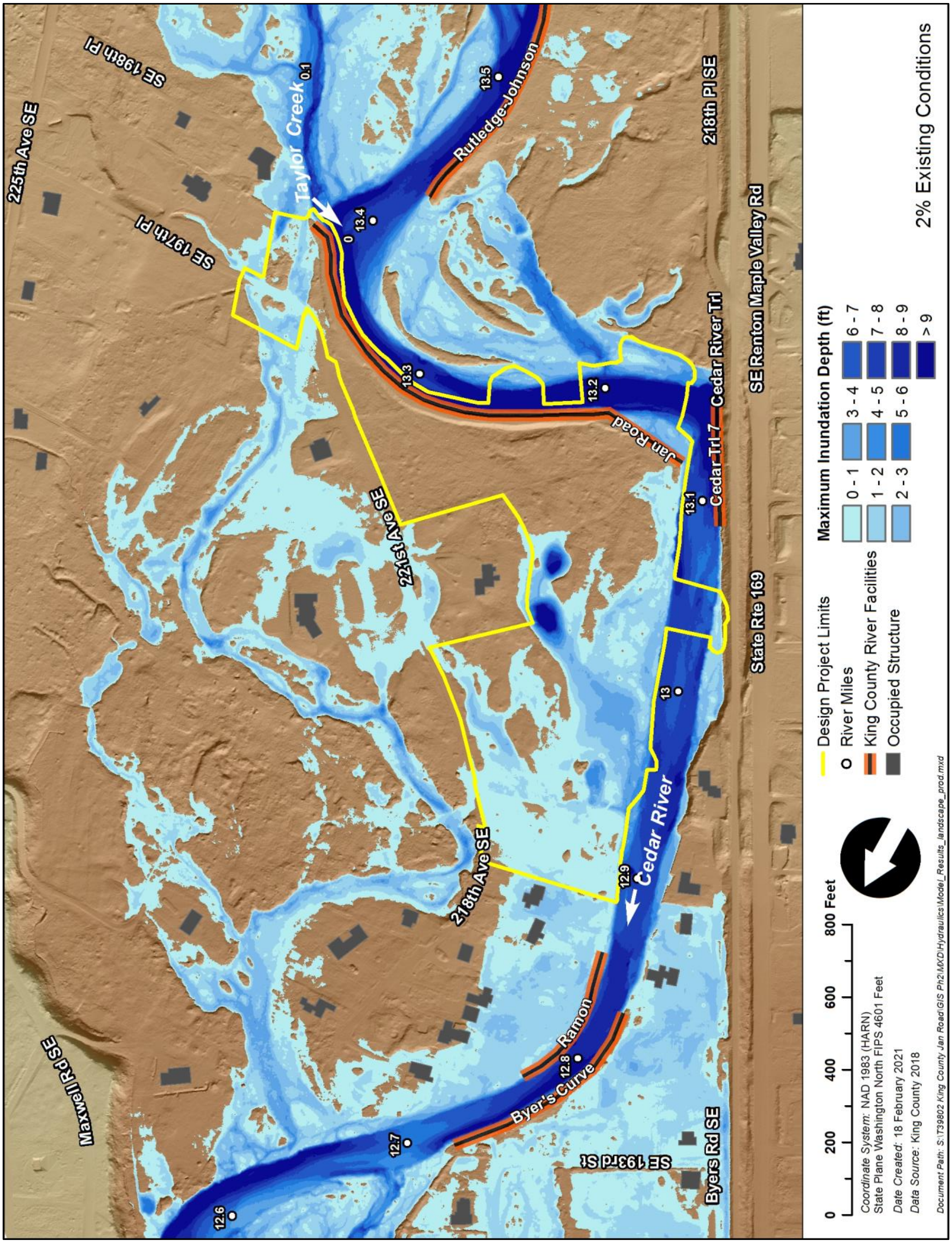

Figure A-4. Existing Conditions, 2% ACF Maximum Depth (Q = 7,820 cfs)

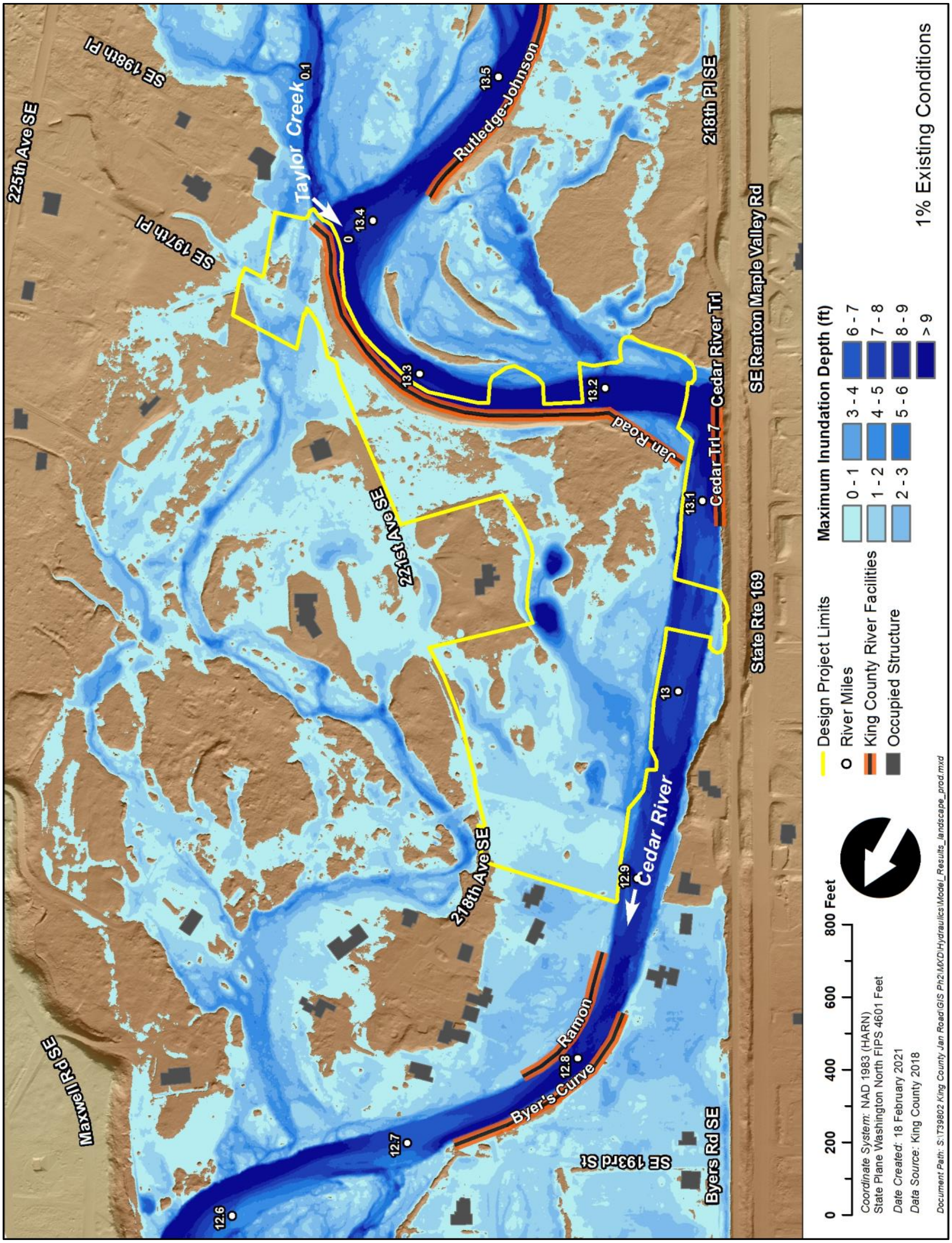

Figure A-5. Existing Conditions, 1% ACF Maximum Depth (Q = 9,440 cfs)

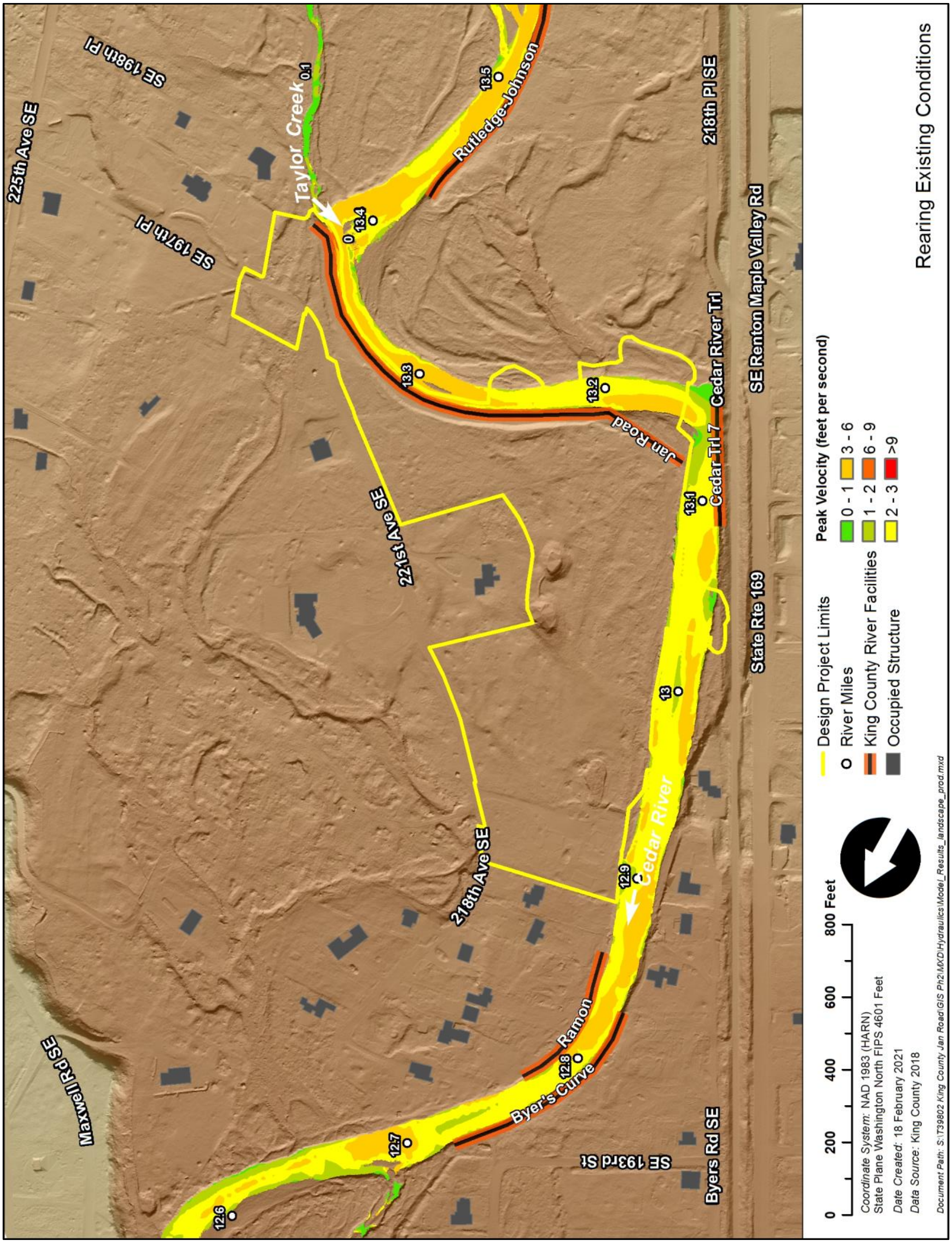

Figure A-6. Existing Conditions, Rearing Flow (Q = 492 cfs), Maximum Velocity

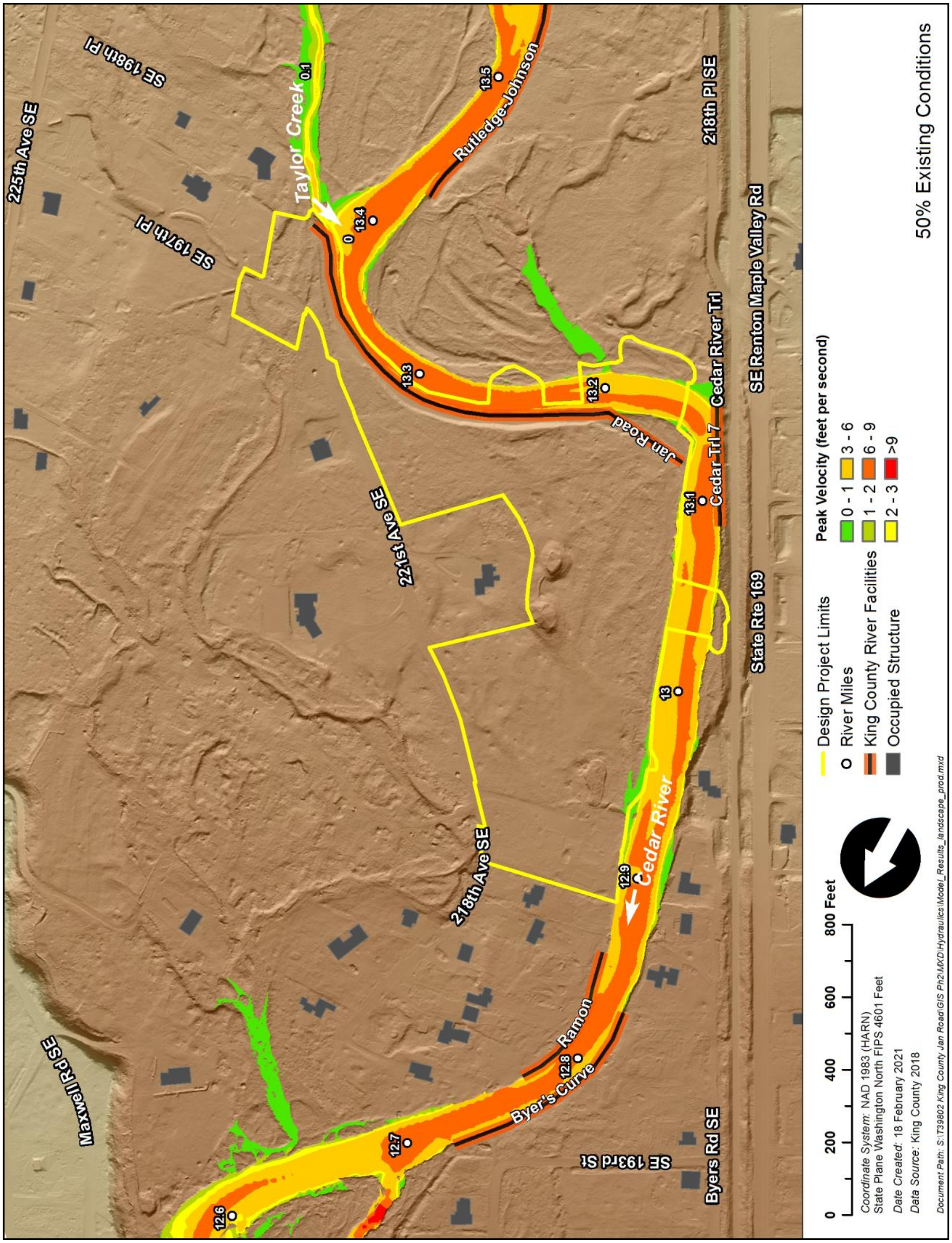

Figure A-7. Existing Conditions, 50% ACF (Q = 2,420 cfs), Maximum Velocity

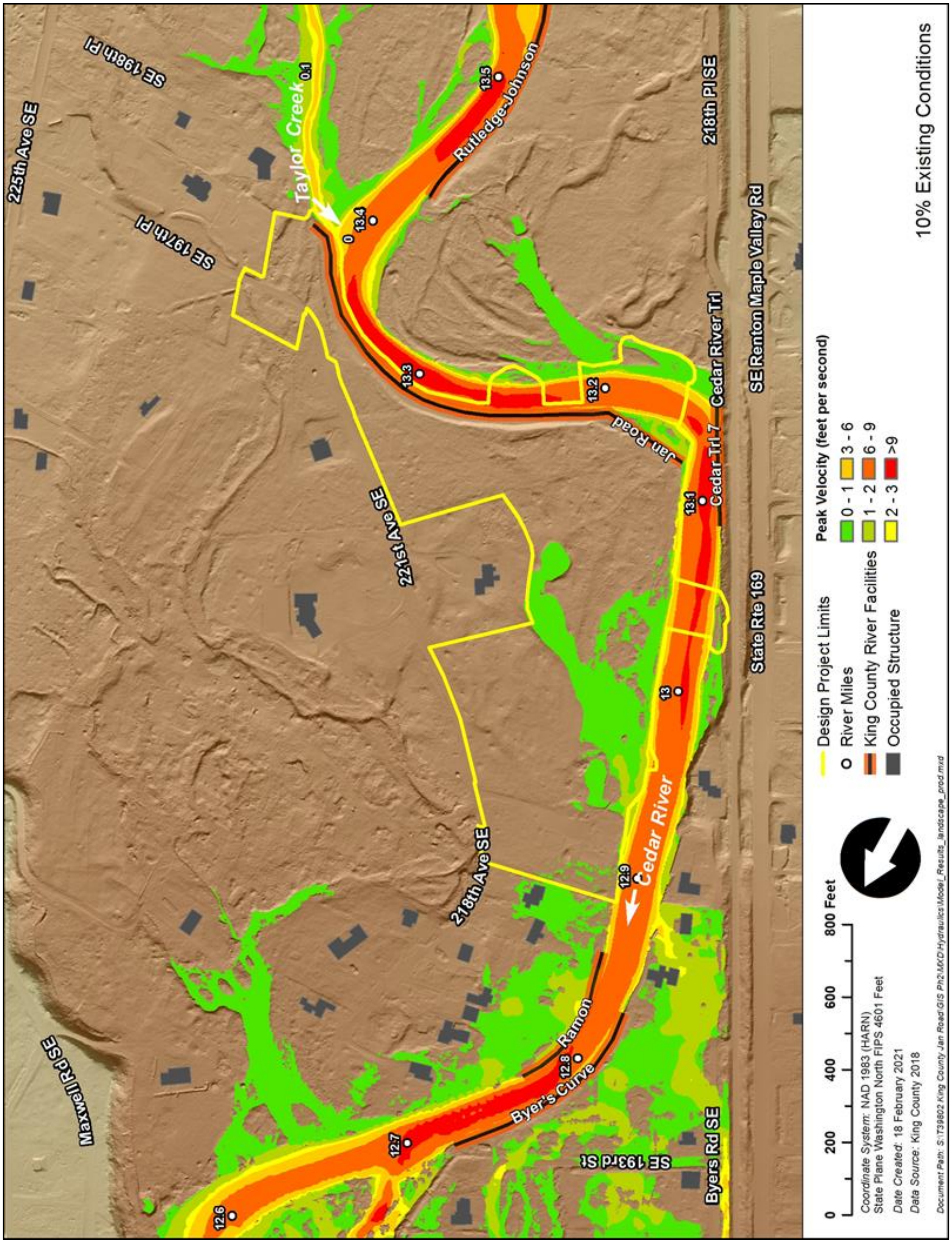

Figure A-8. Existing Conditions, 10% ACF (Q = 4,820 cfs), Maximum Velocity

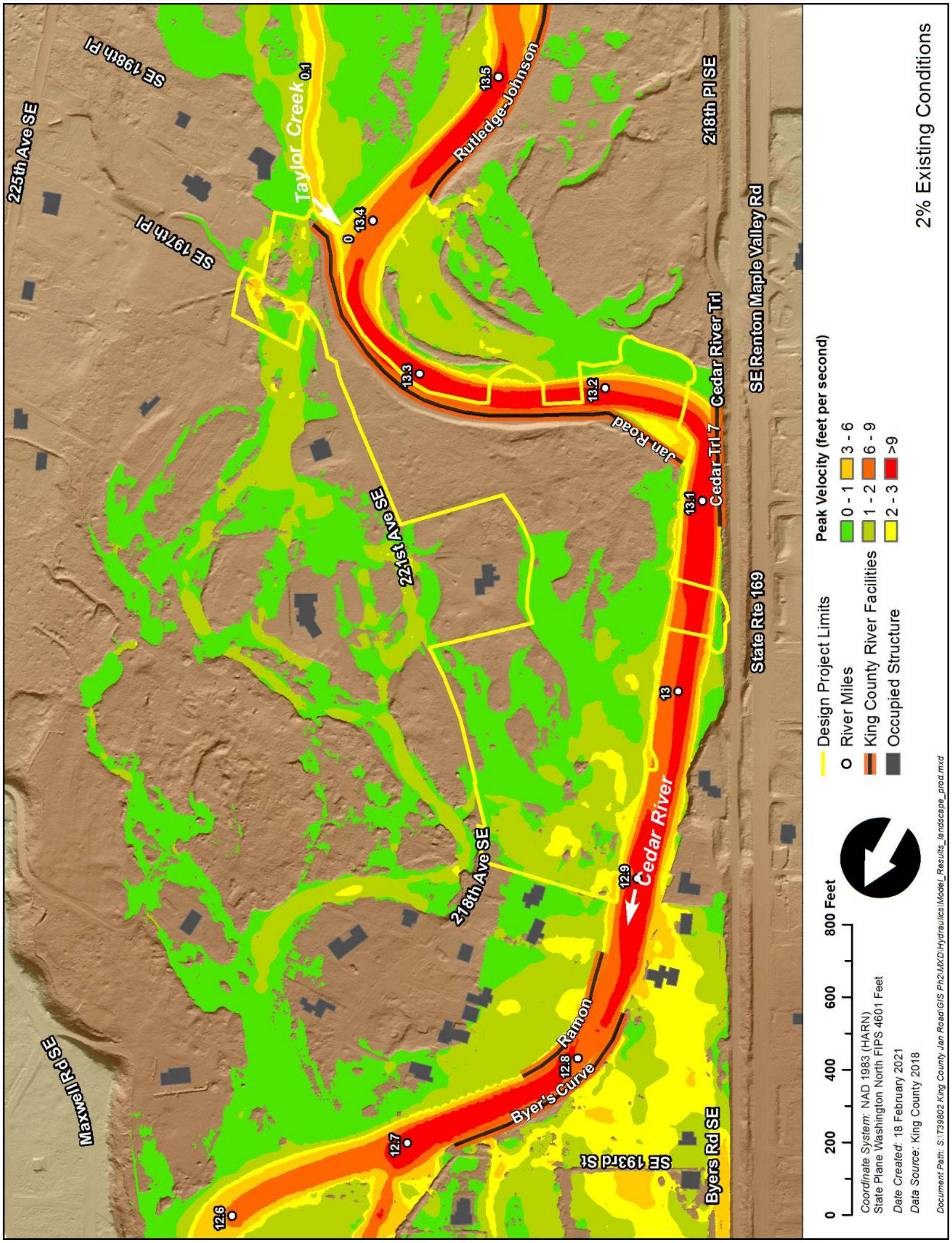

Figure A-9. Existing Conditions, 2% ACF (Q = 7,820), Maximum Velocity

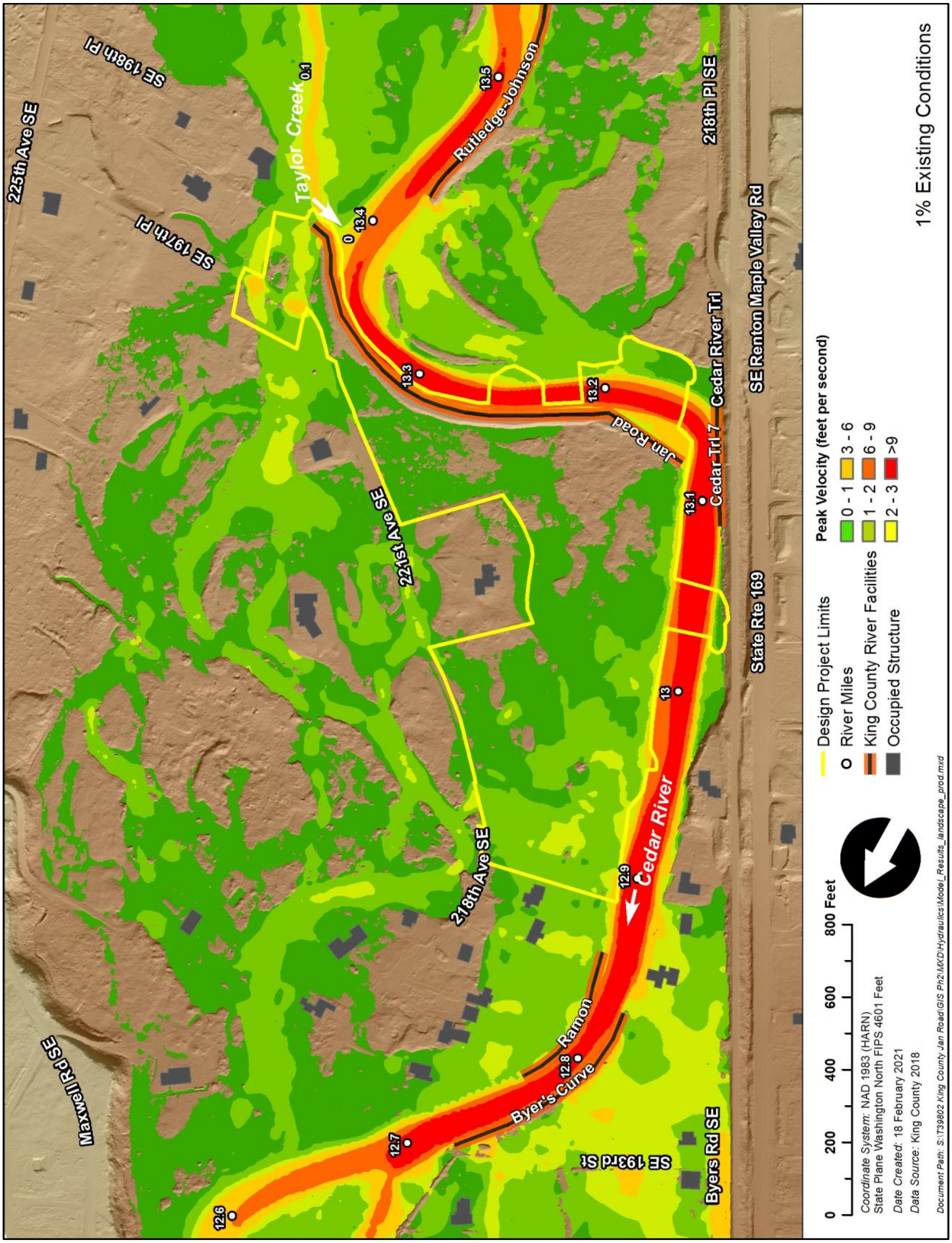

Figure A-10. Existing Conditions, 1% ACF (Q = 9,440 cfs), Maximum Velocity