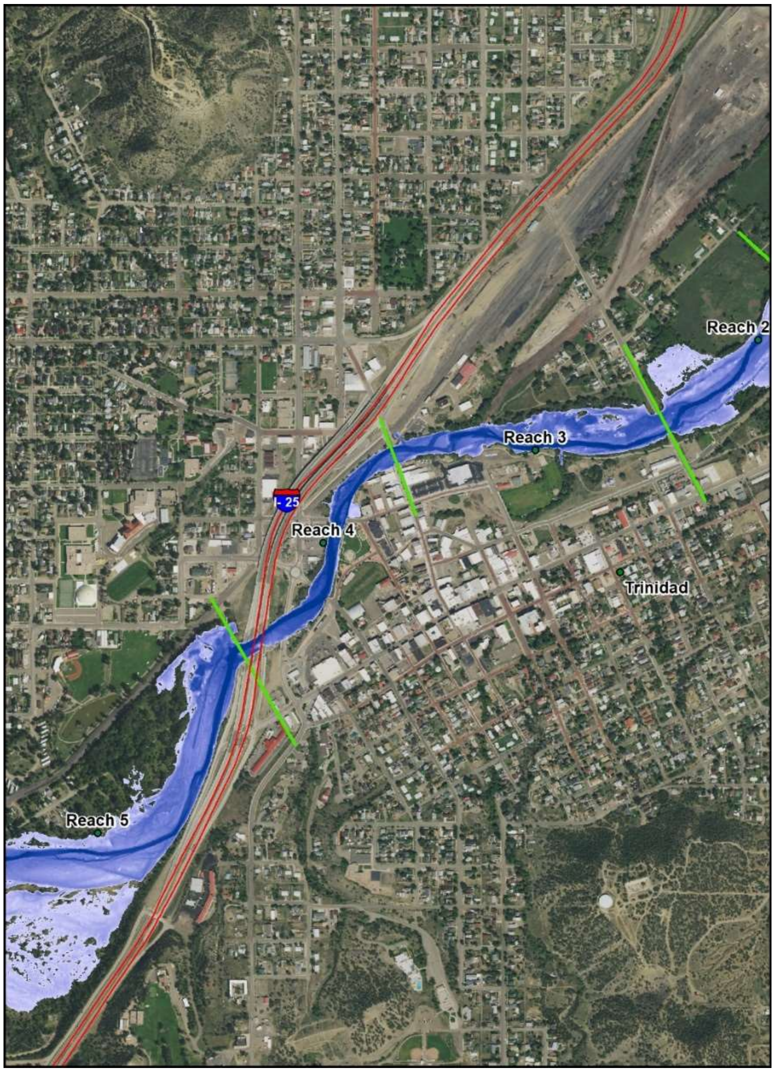
PURGATOIRE RIVER ASSESSMENT REPORT: PART 2

STORM WATER RELEASE AND MUNICIPAL INFRASTRUCTURE ASSESSMENT

2019-2020



Prepared By:

Kevin Pilgrim, Water Resource Engineer, PE
Coalitions & Collaboratives, Inc

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EXECUTIVE SUMMARY

This report documents the hydrology and hydraulics portion of the Purgatoire River Assessment. The primary goal of this assessment is to provide recommendations for river improvements to protect infrastructure through the City of Trinidad in the event a storm water release is required to satisfy water release obligations under the Operating Rules of the Trinidad Lake Dam.

Survey data was collected and two hydraulic models were used to assess the Purgatoire River. A 1D model and a 2D model were both created using the US Army Corps of Engineers Hydrologic Engineering Center River Analysis System (HEC-RAS). A sediment stability analysis was completed to assess the stability of the bed of the river.

The primary recommendation from the hydraulics analysis is to prepare a river monitoring plan to monitor cross sections throughout the study area near key locations of interest. Benchmarks could be established and used in the event of a high water (greater than 1,500 cubic feet per second (cfs)) storm water release. These cross-section profiles and high-water marks could be used to fine tune (calibrate) the hydraulic models.

Recommendations for improvements in the river corridor are discussed in detail in section 7. The primary locations of concern are located upstream of the Interstate 25 bridge in reach 5 and downstream of the Baca-Picketwire diversion in reach 3.

1. In reach 5, the river constricts considerably as it flows under Interstate 25. In the event of a larger stormwater release this will likely cause local scour and backwater effect that would likely inundate the floodplain area to the north of the bankfull channel. The floodplain upstream of I-25 and the channel below are priority locations for improvements to protect the infrastructure.
2. In reach 3, the Baca-Picketwire diversion is a location with the potential for a lot of energy dissipation after flows pass through Trinidad. The channel should be protected upstream and downstream of the diversion as it is a key location that is holding grade for the river through the city. The floodplain downstream of the diversion should be maintained and enhanced in order to provide an area where flood debris and sediment can be allowed to deposit once it passes through the City reaches.

Opportunities for river improvements to improve safety, recreation, fish habitat and resilience exist throughout the river corridor. A collaborative approach is recommended. Communication among the various stakeholders is important as the common goals are better understood. A healthy and resilient river corridor can provide multiple benefits to stakeholders.

1. Introduction

This report documents the hydraulic models built for the 2019/2020 Purgatoire River Assessment. Coalitions & Collaboratives (CO-CO), under contract with the Colorado Water Conservation Board and at the request of the Purgatoire Watershed Partnership (PWP), was tasked with completing a Hydrologic/Hydraulic Assessment for a segment of the Purgatoire River corridor to determine how to best manage winter flows to create a self-sustaining fishery and determine how to best manage controlled release of storm water from Trinidad Reservoir. The determination of a non-damaging flow release is of interest to many stakeholders and the focus of the work for this grant under the Kansas-Colorado Arkansas River Compact. A flow rate of 5,000 cfs is a target and the impacts of flows up to this magnitude is of interest to many stakeholders.

In 2010, Reclamation completed a hydraulic assessment of the Purgatoire River (Klump and Garcia, 2010). A HEC-RAS 1D (one dimensional) steady state model of the Purgatoire River through Trinidad, CO was obtained from the U.S. Bureau of Reclamation. For purposes of clarity, the model produced by Reclamation will be referred to as the 2010 model and the current modeling effort will be referred to as the 2020 PWP Model. The hydraulic study associated with that modeling effort was completed on the Purgatoire River below the Trinidad Dam by Reclamation's Sedimentation and River Hydraulics Group in collaboration with the Army Corps of Engineers, Albuquerque's District Office in 2010. Reclamation created the 2010 1D (one dimensional), steady state, backwater model using HEC-RAS to model a range of flows up to 21,000 cfs. That model was calibrated to a flow of 250 cfs using data collected by Reclamation and Colorado Department of Water Resources. Reclamation's 2010 model begins at the base of the Trinidad Dam and extends downstream approximately 42 miles to the end of the Purgatoire River Water Conservancy District Boundary upstream of Purgatoire Canyon (**Figure 1-1**).

Note on the model limitations: This modeling effort focused on reaches 1-6. The author assumed for this study the majority of infrastructure that stakeholders are interested in assessing lie in reaches 1-5. The study reaches for this hydraulic assessment are shown in **figure 1-2**. Reach 7 was not assessed or mapped due to its proximity to the base of the Trinidad Lake Dam. In reach 7 the floodplain map and hydraulics will be influenced by the level of the lake (the potential energy) and the outlet works of the dam with the associated energy dissipation measures. The 2020 PWP model does not account for these factors and, therefore, no results will be presented at this time. A hydraulic assessment of reach 7 would require close coordination with stakeholders of the Trinidad Lake Dam.

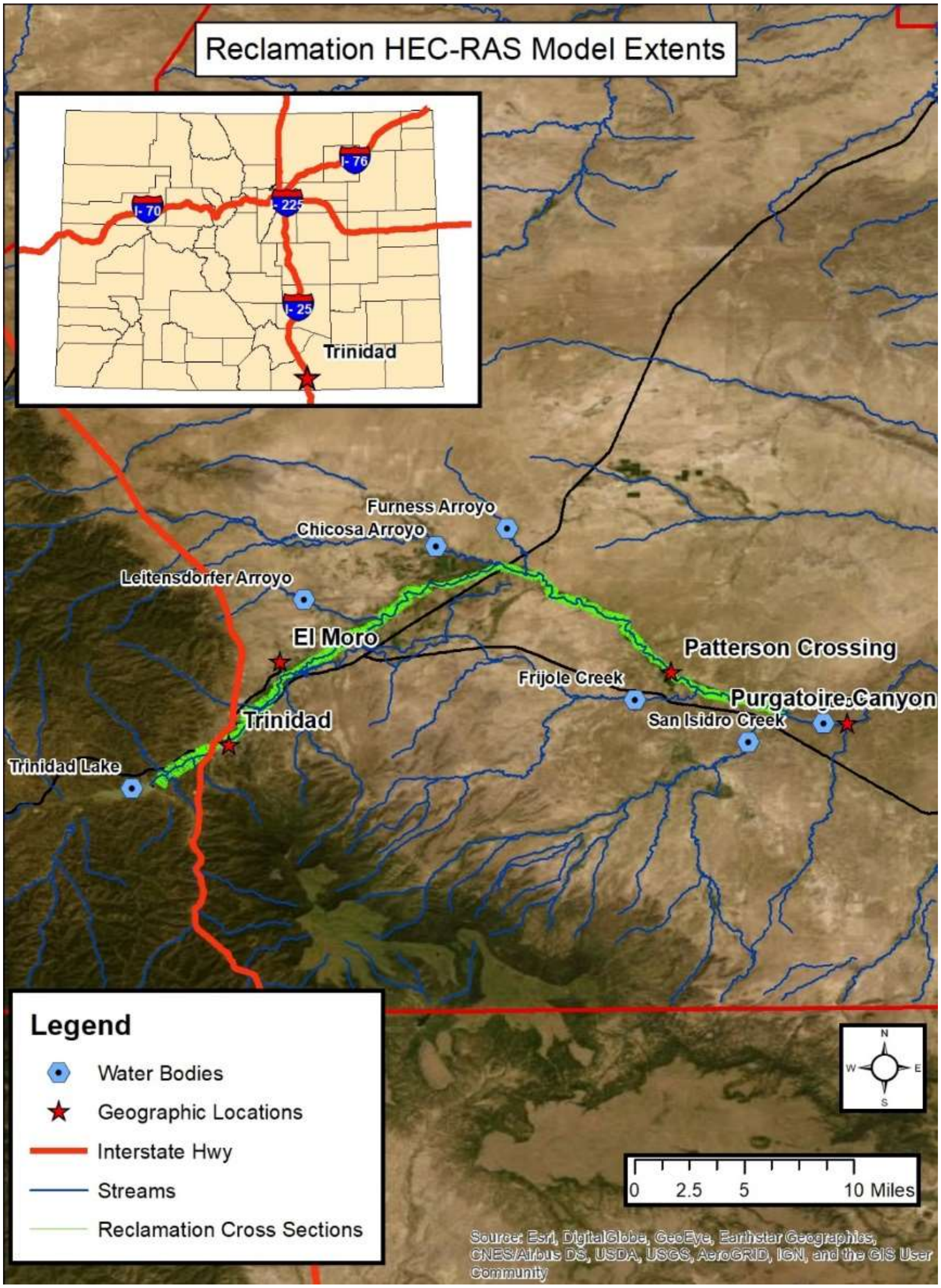

Figure 1-1. HEC-RAS Model extents by Reclamation (Klump and Garcia, 2010)

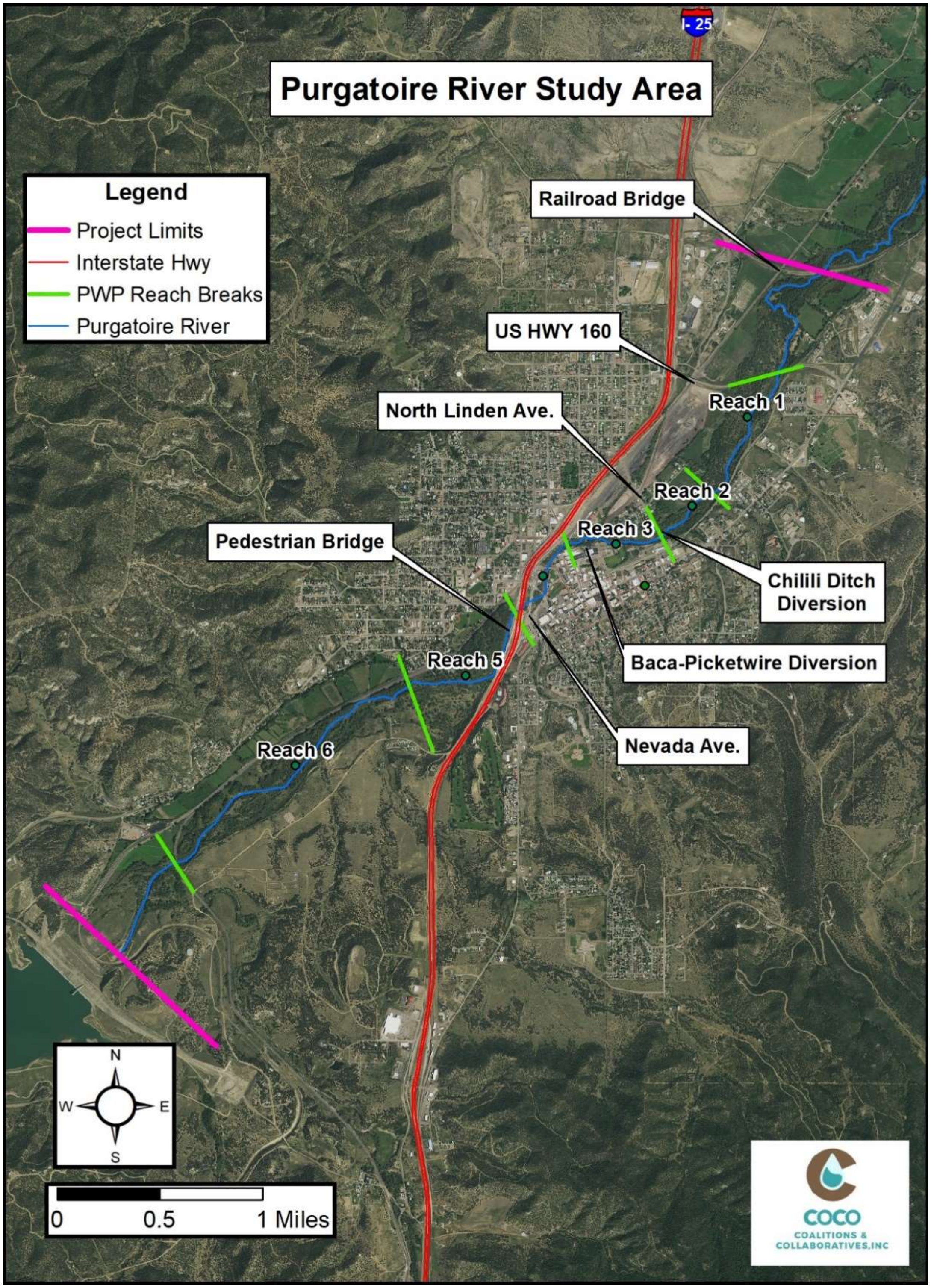

Figure 1-2. Map showing the study reaches used for the 2019/2020 PWP Assessment.

2. Hydrologic Analysis

Reclamation's 2010 model used a single flow throughout the reach. The 2020 PWP model included flows from the calibration flow of 250 cfs up to 21,000 cfs. A peak flow analysis was completed and is presented in section 2.1. Discussion of the high flow release in May 2017 is presented in section 2.2. A flow frequency analysis was also completed and is presented in section 2.3.

2.1. Peak Flow Analysis

Peak flow data for the Purgatoire River at Trinidad stream gage (PURTRICO) was downloaded from the State of Colorado Department of Water Resources (CO DWR) website (http://www.dwr.state.co.us/) and USGS gage #07124410 Purgatoire River Below Trinidad Lake, CO. The USGS gage below Lake Trinidad is near the outlet of the dam. The CO DWR gage at Trinidad is located downstream from the dam and captures additional inflow from side channels below the dam. Peak flow estimates for the post-dam period from 1977 to 2018 for both gages are shown in **Figure 2-1**.

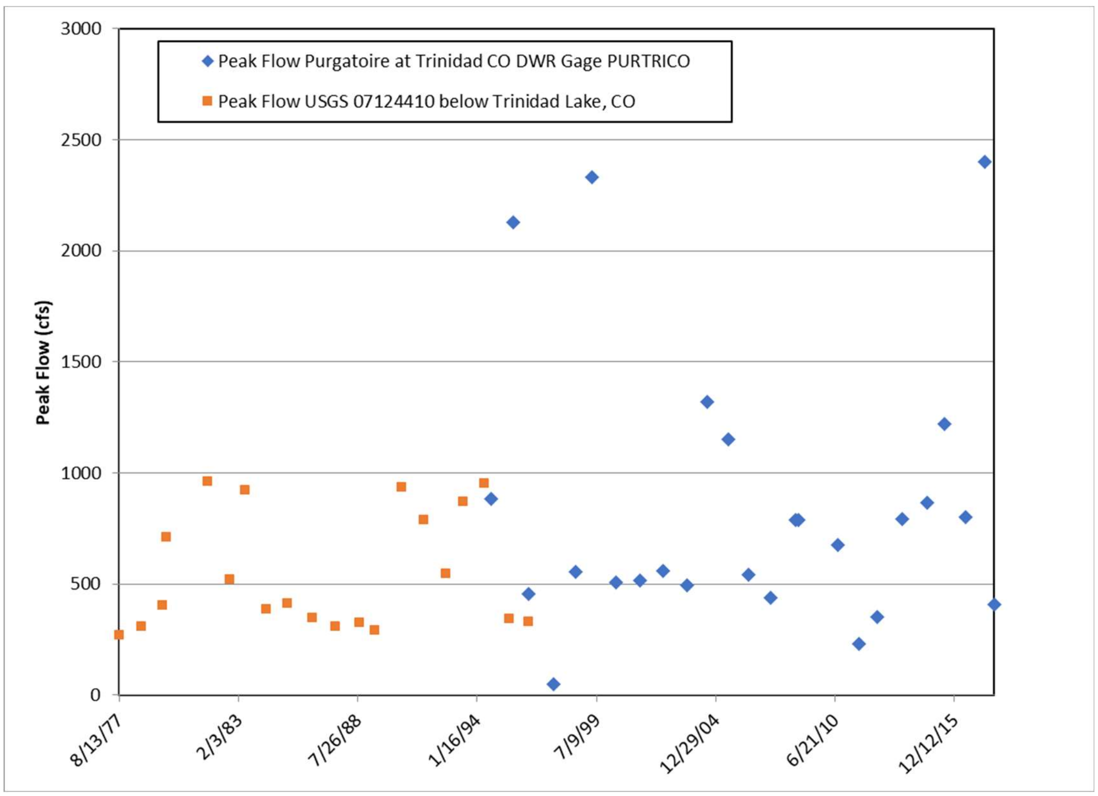

Figure 2-1. CO DWR Peak flow data for the Purgatoire River at Trinidad

Peak flows rarely exceed 1,000 cfs since the dam closure in 1977. Prior to the construction of Trinidad Lake Dam flood peaks were estimated as high as 45,400 cfs in 1904 and 28,000 cfs is 1955. Three events over 2,000 cfs were recorded for the PURTRICO gauge. The Hydrologic Engineering Center Statistical Software Package (HEC-SSP) was used to analyze the pre- and post-dam peak flows within the City of

Trinidad. The results of the bulletin 17B analysis are shown in **figure 2-2**. Pre-dam peak flows range from 500 cfs to 45,500 cfs. Pre-dam a 2-yr flow was 4,463 cfs and a 5-yr flow was 9,600 cfs. Since the dam was completed peak flows have not exceeded 2,400 cfs. The 2-yr and 5-yr flow have been reduced by an order of magnitude to 570 cfs and 954 cfs respectively.

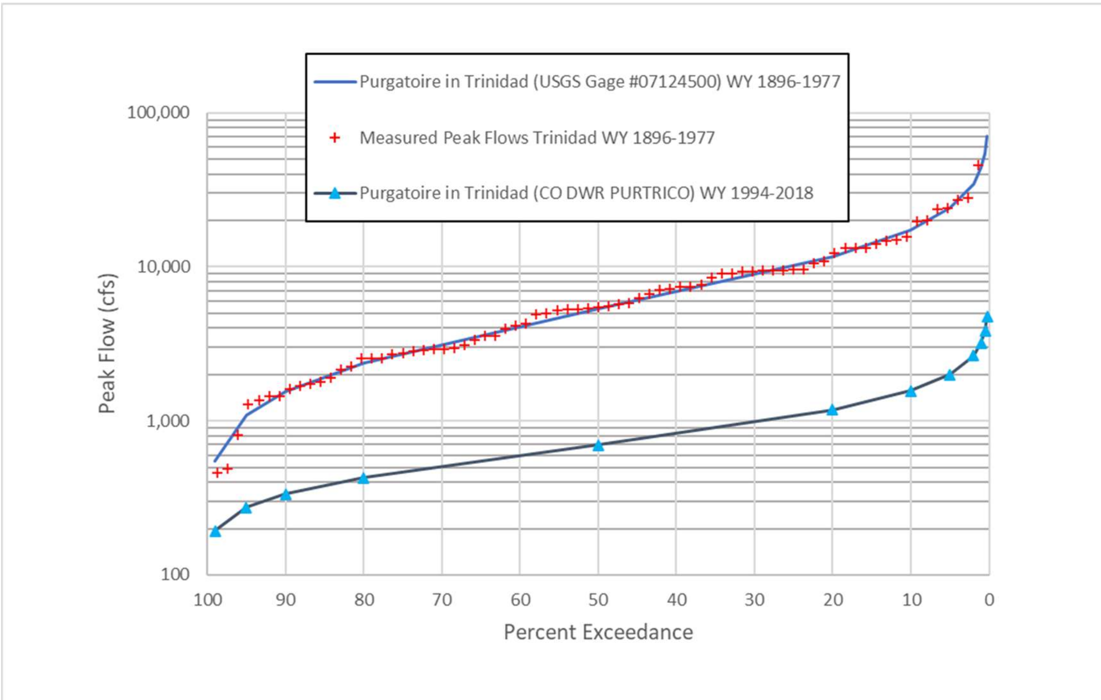

Figure 2-2. Peak flow analysis (Bulletin 17B) below Trinidad Lake pre- and post- dam closure in 1978.

2.2. May 2017 High Flow Event

In May 2017, a flow of up to 2,040 cfs was released from the Trinidad dam (**Figure 2-3**). Based on anecdotal evidence, this high flow event caused some concern for stakeholders. The event is a good example of what could be considered a calibration flow. It is standard practice when using hydraulic models to collect data such as velocity, cross sectional area and water surface elevations to compare to the hydraulic model. If another higher flow event was released from the Trinidad Lake Dam it would be advantageous to collect high water mark information. Measurement of distance to a fixed point such as a bridge deck could be collected. Photographs of the measurements would be very helpful as well. When the opportunity arises, survey grade GPS data could be collected at locations throughout the reach upstream and downstream of hydraulic control points such as the bridges through town and the Baca-Picketwire diversion dam to use for future calibration efforts.

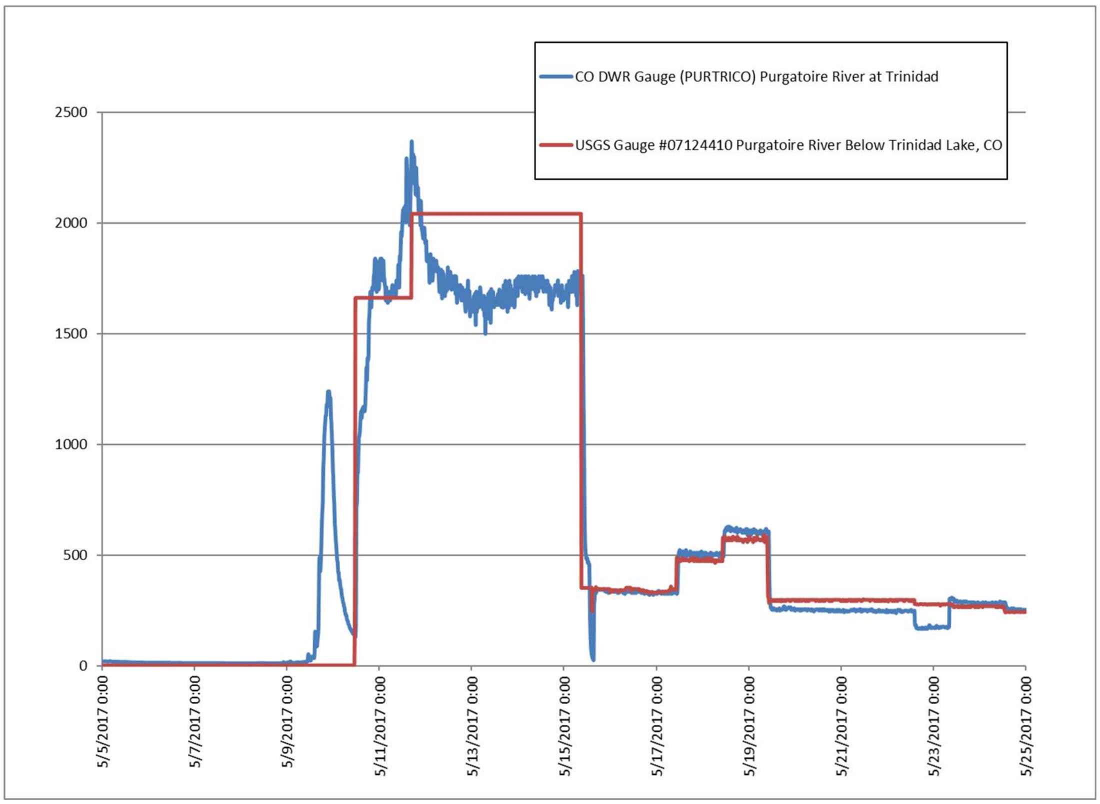

Figure 2-3. Hydrograph showing a comparison of the USGS gauge #07124410 Purgatoire River below Trinidad Lake and the CO DWR PURTRICO gauge at Trinidad.

2.3. Average Annual Hydrology

The post-dam hydrology was evaluated for this study by analyzing the gage data upstream and downstream of Trinidad Lake. The data for USGS gage #07124200 Purgatoire River at Madrid and USGS gage #07124410 Purgatoire River below Trinidad Lake was used. A flow duration curve is created by statistical analysis of the mean daily flow record. The Hydrologic Engineering Center Statistical Software Package (HEC-SSP) was used to create an average annual hydrograph using the mean daily flow for these gages. The pre- and post-dam flow duration curves shown in **Figure 2-4** illustrates the decrease in flow following the closure of the dam. In an average year, the flow only exceeds 1 cfs 50% of the time.

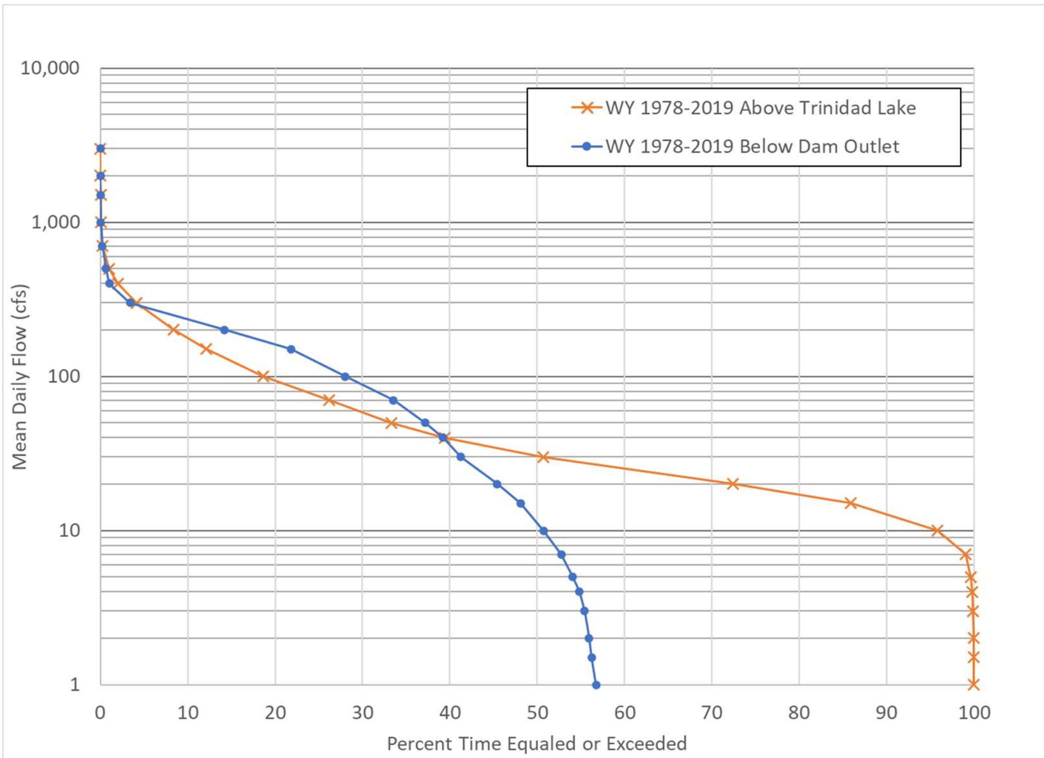

Figure 2-4. Flow duration analysis above and below the Trinidad Lake dam for water years 1978-2019.

3. Hydraulic Model Setup

3.1. 1D HEC-RAS Model

3.1.1. Cross Section Geometry

The 2010 model obtained from Reclamation is a single thread model steady state approximately 42 miles long. The overbank geometry is based on LiDAR data assumed to be collected in 2009 and the in-channel geometry is based on cross section surveys collected by Reclamation. The horizontal projection for this project is Colorado State Plane South NAD 1983 (2011) US feet and the vertical datum is NAVD 88 US feet. This applies to the 1D and 2D models and results.

The 2018 LiDAR data was compared with the 2009 LiDAR survey. An example profile was created to compare the in-channel elevations interpolated on the Digital Elevation Model (DEM) (**Figure 3-1**). LiDAR does not give a return on the bed of the channel so the data within the bankfull low flow channel is normally clipped out. The resulting elevation is based on triangulation from the left bank to the right bank and does not represent actual elevation data. The irregularities in both surfaces can be seen along the profile line in the graph shown in **Figure 3-2**. The 2018 data which appears to be showing aggradation of approximately 2 feet is not actual data but an artifact of the location of clipping the water surface out of the LiDAR data set. The Reclamation 2009 data is in green and the 2018 data set is in yellow.

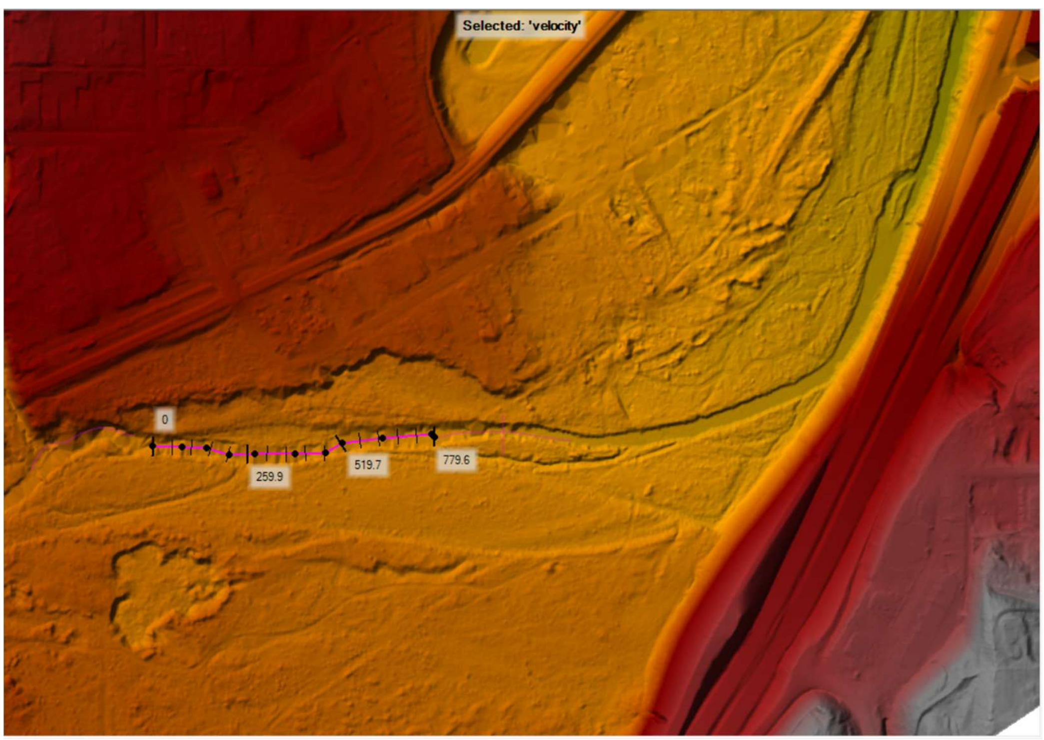

Figure 3-1. A profile was drawn along the channel in reach 5 to compare the 2018 DEM with the 2009 LiDAR surface provided by Reclamation. Interstate 25 can be seen in the LiDAR to the right of the screenshot.

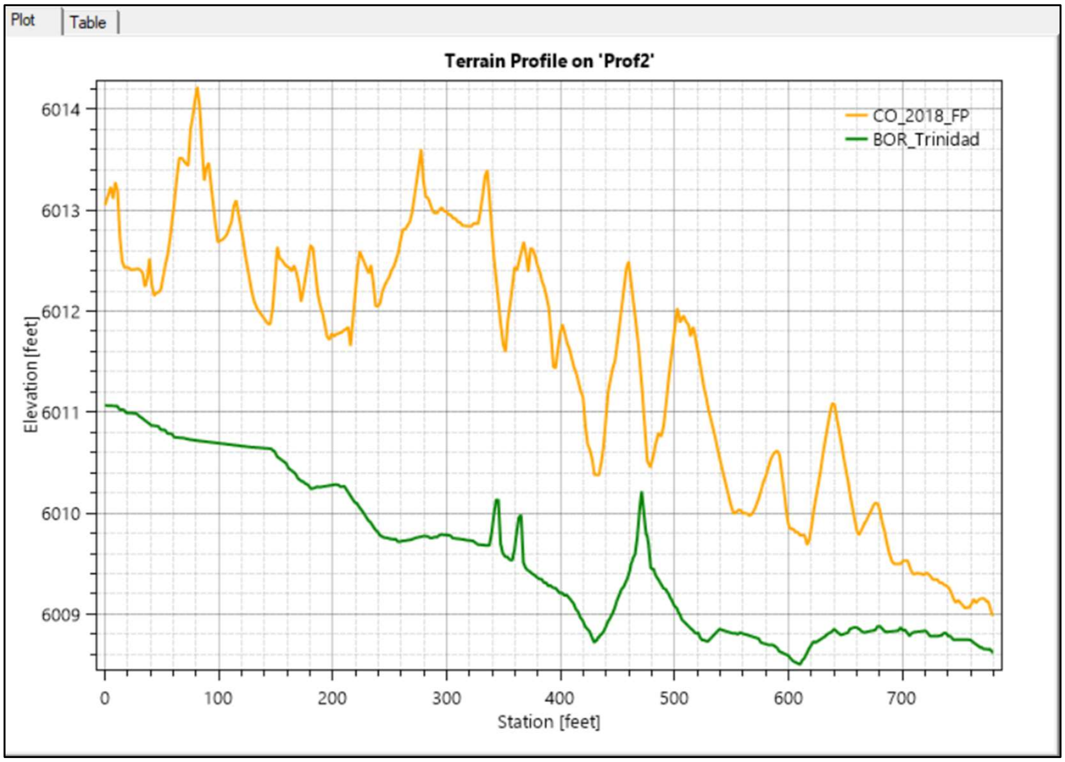

Figure 3-2. A plot showing the comparison of the 2009 surface with the 2018 surface elevation data in the low flow channel.

The 2020 PWP 1D HEC-RAS model was updated with LiDAR data obtained from the State of Colorado Governor's Office of Technology. The LiDAR data was obtained in DEM format and represents bare earth. In channel survey data was collected by Pete Gallagher with Fin-Up Habitat Consultants (Fin-Up) in May 2019 in reach 5 as part of monitoring effort and geomorphic assessment (**Figure 3-3**). The May 2019 survey data was compared to the 2009 Reclamation data and the 2018 LiDAR data. The inconsistencies between the data set could not be explained so the 2009 Reclamation data was used to represent bathymetry (channel bed topography) of the Purgatoire River.

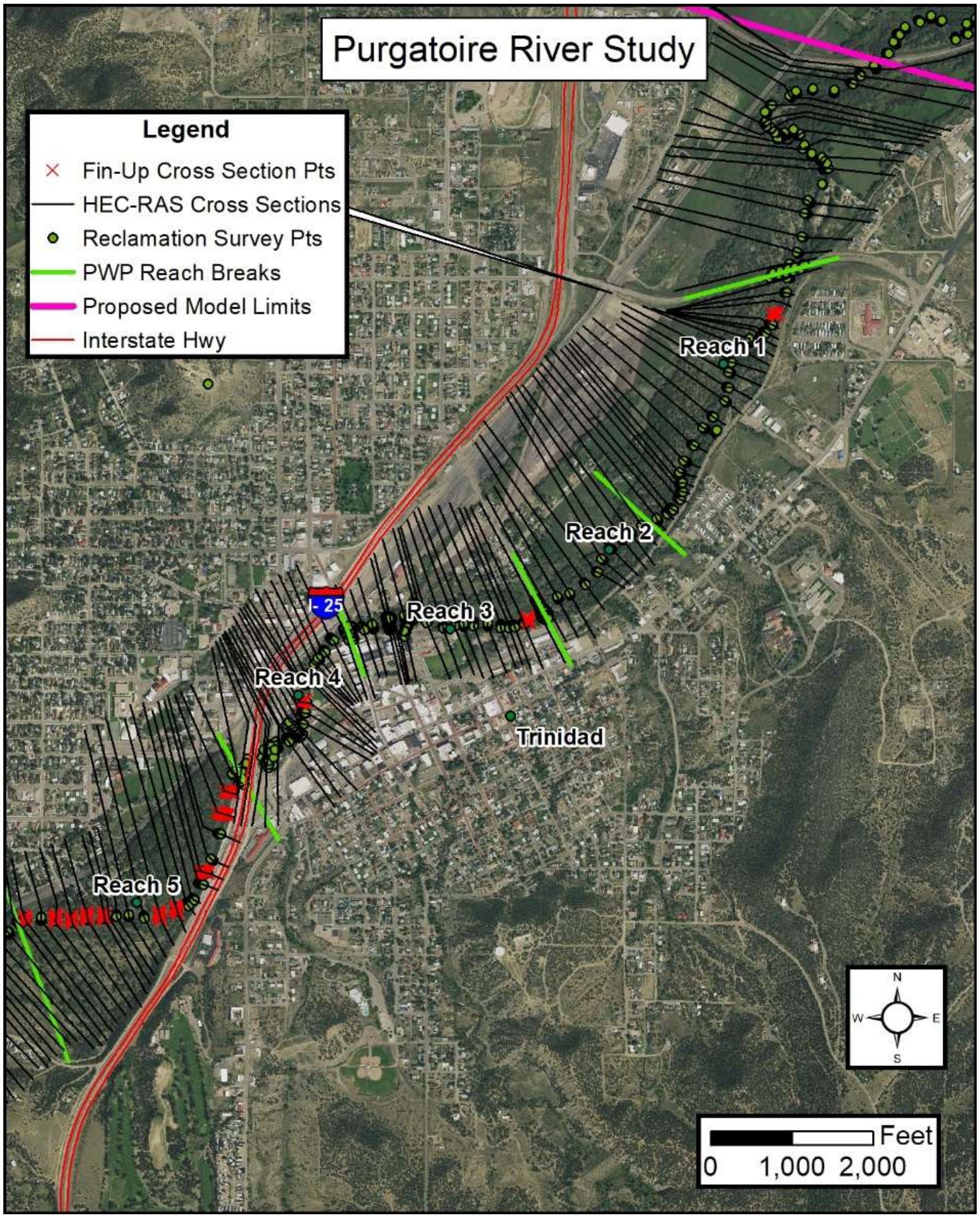

Figure 3-3. Reach 5 survey data is shown by red x collected in May 2019.

Cross sections were drawn according to the location of the in-channel survey data and following standard practice of 1D modeling (Figure 8). Cross sections are drawn in a 1D model perpendicular to the modeler's interpretation of the three flow paths in channel and left and right overbanks. The reach lengths between cross sections are calculated using overbank flow paths. All geometry edits were made using the RAS Mapper tool.

RAS-Mapper has the ability to generate a surface that is representative of in channel bathymetry. The cross sections cut from the LiDAR surface were edited using the 2009 channel cross sections collected with RTK-GPS. The GPS points were projected to the line perpendicular to the assumed flow direction, so all calculations are based on the width of the channel perpendicular to flow. A 3D surface representing the in-channel bathymetry was created within RAS mapper to be an interpolation between the cross sections. This surface was merged with the LiDAR DEM using ARC-MAP add-in 3D Analyst. If additional cross sections are needed the cross section can be cut on the edited surface instead of using the interpolation function in HEC-RAS. Low flow results can be mapped more accurately this way.

3.1.2. Energy loss

Hydraulic models require estimation of energy loss due to turbulence, boundary friction, form friction and sediment transport in order to compute water surface elevations for a given discharge. Manning's n is the primary friction coefficient used in a HEC-RAS model. The 2010 model was evaluated based on the past survey data and flow release of approximately 2,000 cfs in May of 2017. Reclamation calibrated the historical 1D model to the low flow measurements during the 250 cfs release. Based on these low flow measurements a Manning's n value of 0.033 was used for in channel roughness while 0.077 was used for overbank roughness. The use of a low flow calibration limits its value in assessing the energy loss on the floodplain. The n-values appear to be low based on the density of vegetation in the connected floodplains upstream and downstream of the city. It would be beneficial to collect water surface data at higher flow to validate this assumption. This concept of parameter validation or model calibration was discussed in section 1.2 where the May 2017 runoff event was documented.

The existing conditions analysis presented in the current hydraulic assessment are based on the in-channel values of 0.033 and 0.077 for overbanks for ease of comparison with Reclamations results. Additional model versions were created with higher n values to create a range of conditions to evaluate the effect of increased energy loss. Future design efforts could use a range of n values to examine scenarios (a sensitivity analysis) and account for uncertainty with estimates of energy loss.

A uniform n-value may not be appropriate for the changing conditions within the study area on the Purgatoire but the best method for estimating Manning's n in a complex floodplain is using calibration data. If higher than bankfull flows are released (above 1,000 cfs) on the Purgatoire River running through the City of Trinidad, high water marks should be recorded and could be used with flow data from the gage to calibrate the model further.

The n-values were increased to 0.045 and 0.1 for in channel and overbank respectively in order to examine the effect the manning's n-value has on the computed water surface elevations. As previously stated, the best way to determine the appropriate Manning's n-value is to measure water surface elevations over a range of flows and compare the measurements to the model. The model is an "ideal" scenario and the results should be viewed as such. In the real world, the energy loss (Manning's n value) varies across the channel, longitudinally (upstream and downstream) and vertically with increase in water surface elevation. When viewing the results (the inundation maps), the reader should be aware of the limitations of a hydraulic model. Inundation maps for a flow of 5,000 cfs in section 4.

The Purgatoire River flows through multiple bridges through the City of Trinidad. The contraction and expansion coefficients could also be raised to account for energy loss from contraction and expansion of flow. The default coefficient for contraction, 0.1, and expansion, 0.3, were used for the majority of cross sections. The coefficients were increased to 0.3 and 0.5 for cross sections upstream and downstream of bridges to account for additional losses.

3.1.3. Bridges and Structures

The bridges and inline structures within the study reach were reviewed within the Reclamation Model and used in the 2020 PWP Model. The bridge decks were extended to model overbank flows. Ineffective flow areas were checked according to the HEC-RAS manual to properly model the contraction and expansion zones.

Additional topographic survey data was collected for this current modeling effort to account for the changes within the study area since the 2010 modeling effort. For example, a pedestrian bridge was recently built immediately upstream of the I-25 crossing. The right bank appears to be modified to accommodate the sidewalk approaching this bridge (**Figure 3-4**). The channel, sidewalk and pedestrian bridge were surveyed and added to the model. The survey was conducted with a total station. This is a differential measurement that is accurate for local surveys but not tied to a horizontal or vertical datum. There are no survey monuments close to the location, so the data was projected (moved and rotated but not scaled) to fit the other GIS data. The elevation is based on comparison to the LiDAR data. No scaling (projection to a state plane grid) was attempted since it is a small area.

Figure 3-4. View from the pedestrian bridge, looking downstream underneath the I-25 crossing at the sidewalk and embankment modifications.

3.1.4. Boundary Conditions

One flow contribution point is used for the length of the model. The 17 flows from 250 cfs to 21,000 cfs were run based on the previous modeling work by Reclamation. All runs were computed using steady state, subcritical flow option. The downstream boundary condition used the normal depth option with a slope of 0.004 ft/ft.

3.2. 2D HEC-RAS Model Setup

3.2.1. Model Geometry

A two-dimensional (2D) model was also developed, to capture complex overbank flow characteristics that 1D hydraulic models do not model well. In locations such as the transition from a broad floodplain to the constricted channel under the I-25 bridge a 2D model can look at overbank flow and estimate the elevation when water flows north around the bridge. The HEC-RAS 5.0 2D capabilities (Corps, 2016) was chosen for its relative ease of use and very stable computational engine and is freely available. Results from the modeling were used to supplement the results from the 1D modeling in developing a sound basis for the project recommendations.

A key component in the development of a 2D model is the development of a continuous topographic surface (terrain model) to represent the topography of the channel and floodplain in the study area. The 3D surface described in section 3.1 was used for the 2D model.

Unlike 1D hydraulic modeling that is based on cross sections at specific locations to describe the reach topography, 2D modeling relies on a computational mesh composed of 3-sided, 4-sided, 5-sided, etc. computational cells that cover the entire domain of the flow field being modeled. The computational cells are superimposed on the terrain model to define the topography for the model. A mesh is created from the cells using tools within HEC-RAS geometry (**Figure 3-5**). Breaklines were drawn along the top of bank to better define the channel and cell spacing was set to 20 feet. Overbank cell spacing was set to 50 feet.

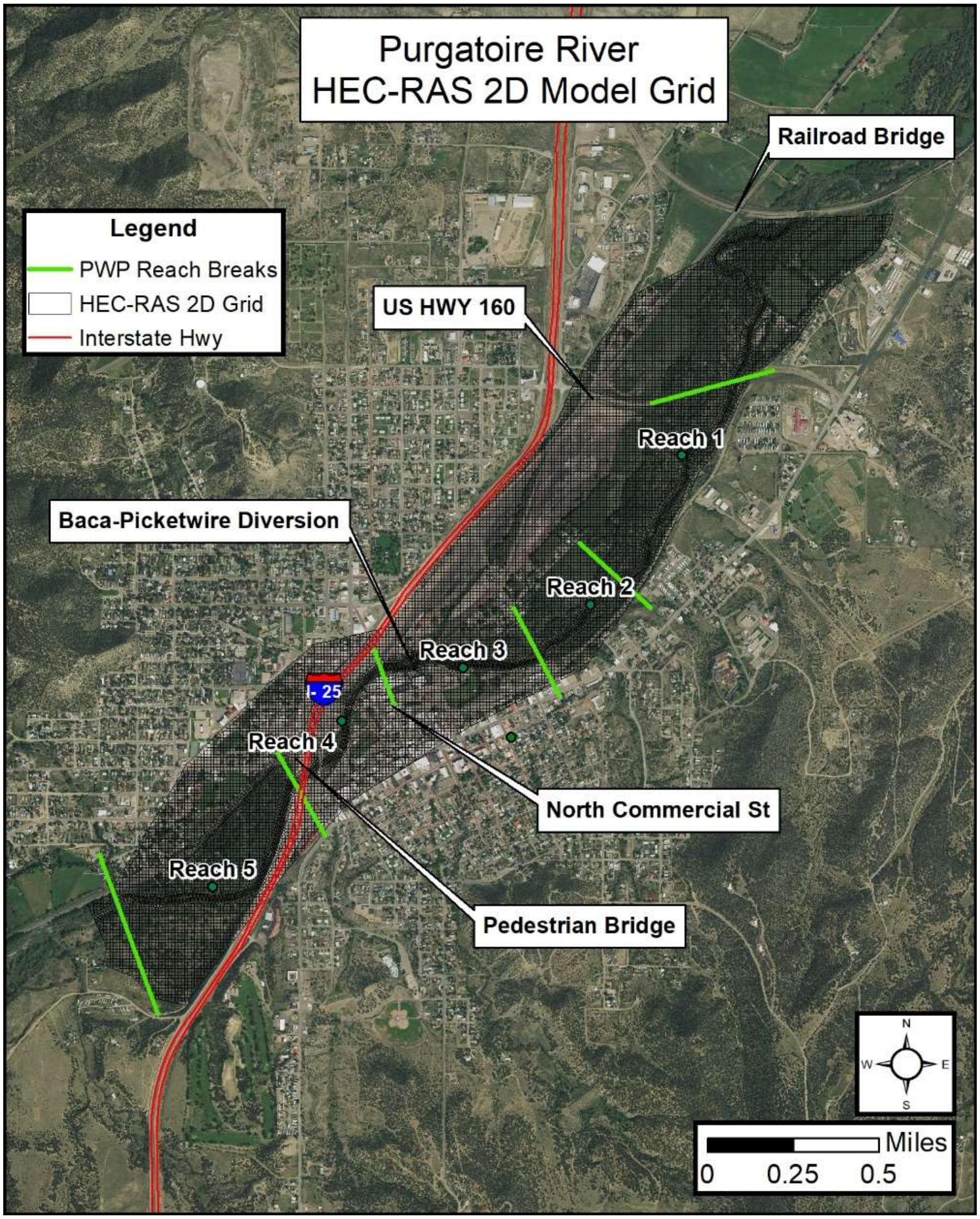

Figure 3-5. HEC-RAS 2D Model Grid

3.3. Energy loss

Similar to 1D models, 2D models like HEC-RAS 5.0 characterize energy losses associated with channel and floodplain roughness using Manning's n values. These are characterized in the model as land use polygons. The Manning's n values used are shown in **Table 3-1**. As discussed previously in this report these energy loss parameters are estimates. A model calibration using measured water surface elevations associated with a known discharge would ensure a more robust estimate of flood hydraulics for flows that exceed the bankfull channel capacity.

Table 3-1. Manning's n values used for 2D model for the various land uses

Land Use	Manning's n
Agricultural Land	0.070
Bankfull Channel	0.045
Low Flow Channel	0.035
Open Space	0.030
Floodplain	0.080
Urban Area (includes effects of homes and buildings)	0.150
Forest	0.100

3.4. Boundary Conditions

The 2D model requires boundary conditions be set at the upstream and downstream end similar to a 1D model. The model is run as an unsteady model meaning the flow changes with time. In order to assess specific flow rates, the hydrograph was entered as a ramp with a series of steady states (**Figure 3-6**). This allows the unsteady model to produce steady state results by examining the hydraulics at the end of the time step. The unsteady 2-D computations in HEC-RAS can be done using the full dynamic equations (2D Saint Venant) or a simplified Diffusion Wave approximation. The 2D runs were done using the full dynamic equations (see the HEC-RAS 2-D modeling user's manual [USACE, 2016]).

The flow steps were arbitrarily chosen apart from the 5,000 cfs and 15,000 cfs. 15,000 cfs is identified as the uncontrolled release rate when the Trinidad Lake Dam spills uncontrolled. The intervening flows were chosen to look more closely at things like when the flow goes north around the I-25 bridge abutments. The normal depth option was chosen for the downstream boundary condition.

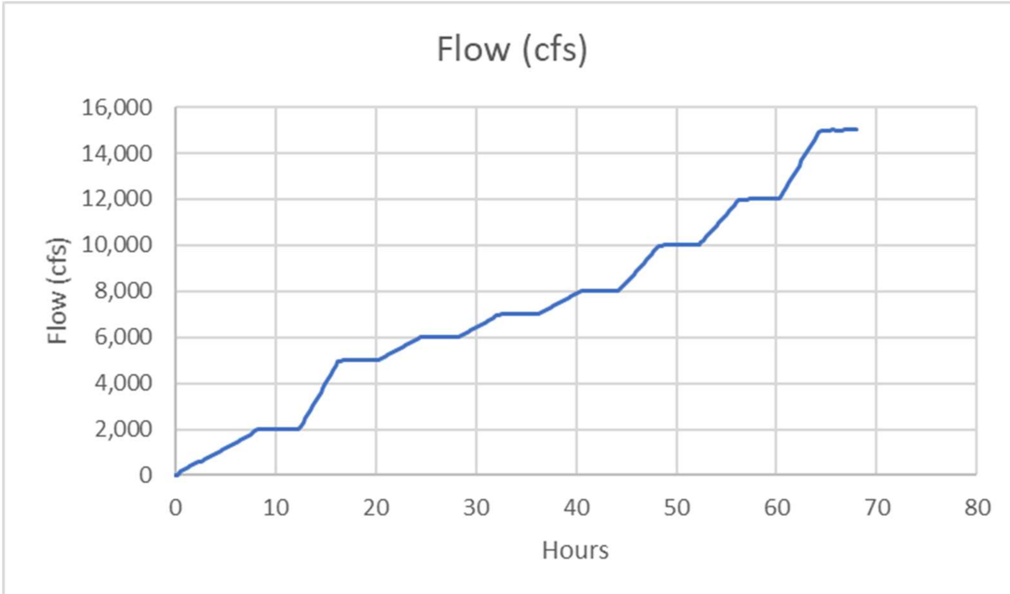

Figure 3-6. Inflow hydrograph at the upstream end of the model. Results are stored for each step that could be extracted to create maps of interest.

3.5. Structures Modeled in 2D

HEC-RAS 2D has the ability to model hydraulic structures within the 2D mesh area. The structures are based on 1D equations, so a connection is created for the structures using a breakline. The pedestrian bridge was modeled since it is overtopped at 4,000 cfs in the 1D model. The Baca-Picketwire diversion dam was also modeled using the inline structure option within the 2D mesh. The diversion is a critical point that controls the water surface elevation upstream and the flows go through critical depth as it falls over the diversion. Using the inline structure option increased model stability through this area as the rapidly changing velocity and water surface makes it difficult to model with a 2D mesh.

3.6. Model Run Parameters

The fixed time step option was used for 2D model runs. The full momentum option was used to account for effects of turbulence. Maximum iterations were set to 40 iterations. The water surface tolerance and volume tolerance were left at the default of 0.01. The eddy viscosity was set to 0.5.

4. Hydraulic Model Results (Storm Water Release Assessment)

4.1. Results Discussion

The primary focus for this effort is to examine the 5,000 cfs flow and the potential impact on the Purgatoire River through Trinidad. The discussion here will focus on the 1D model results. 2D map results showing area of inundation for various flows are shown in figures 4-11 through 4-16.

4.2. 1D Model Results at Bridges and potential impact of 5,000 cfs

The majority of utilities through the City of Trinidad is located at the bridges. The conduits are located on the underside of the bridges, therefore, the focus of the impact of a 5,000 cfs. A scour analysis was not completed because this requires a much larger level of effort that is well beyond the scope and budget of this grant.

Only the pedestrian bridge located under Interstate 25 is overtopped at 5,000 cfs based on the 1D model results (both low and high n-value). A screen shot showing the HEC-RAS model and the computed water surface elevations is shown in **figure 4-1**. The focus of the screen shot is centered on the bridges from Interstate 25 to the railroad bridge crossing below the Baca-Picketwire diversion dam. These bridges are most susceptible to impacts of high flows through town because of the constricted floodplain through town and, therefore, the decreased conveyance. The pedestrian bridge is most susceptible to impacts of higher flows since it will be subject to additional forces when it is overtopped.

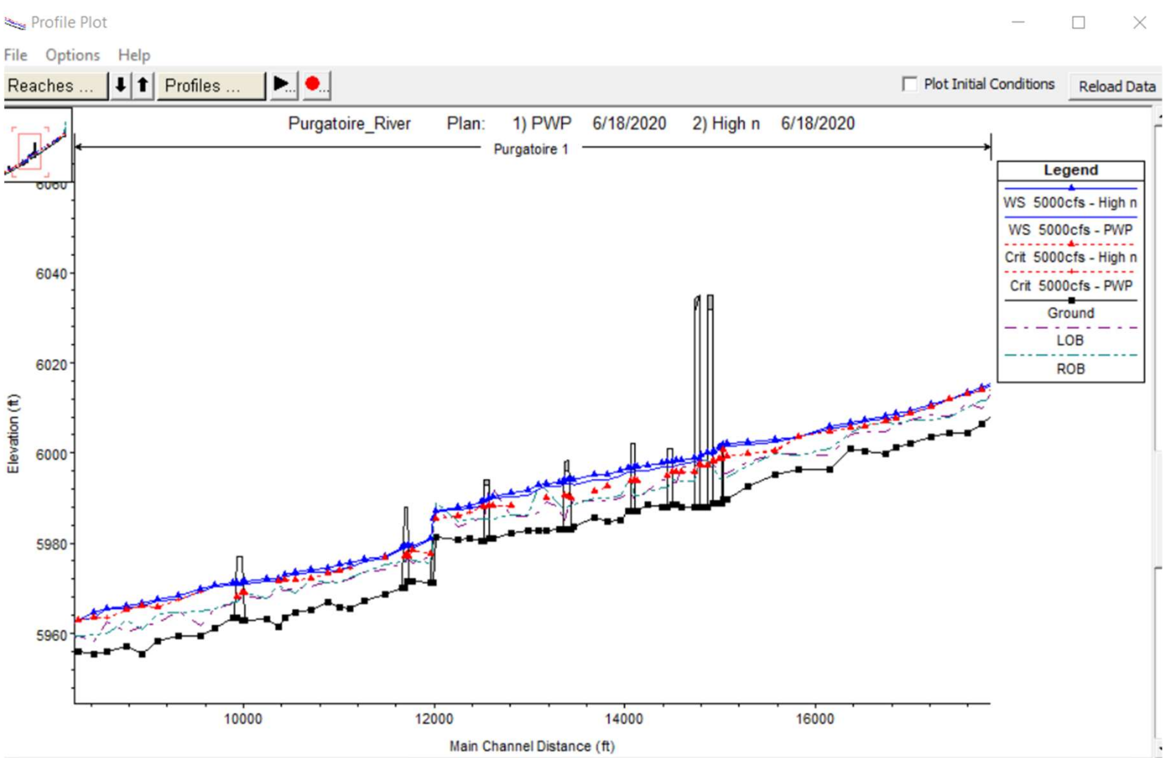

Figure 4-1. Screen shot of the 1D HEC-RAS hydraulic model showing the comparison of the computed water surface elevations at 5,000 cfs.

4.3. 1D Model Results Velocity Profile through the Study Area

The 1D HEC-RAS model velocities profiles through the study area are shown in **figure 4-2**. These results can be used in later investigations or design depending on the engineer's preference. The plot shows generally what one would expect. There are locations of transition from high to low velocity based on changes in channel geometry. Velocities of 12 feet per second are generally high and can cause local scour at bridges, on bends, the base of steep slopes and the floodwalls through the City.

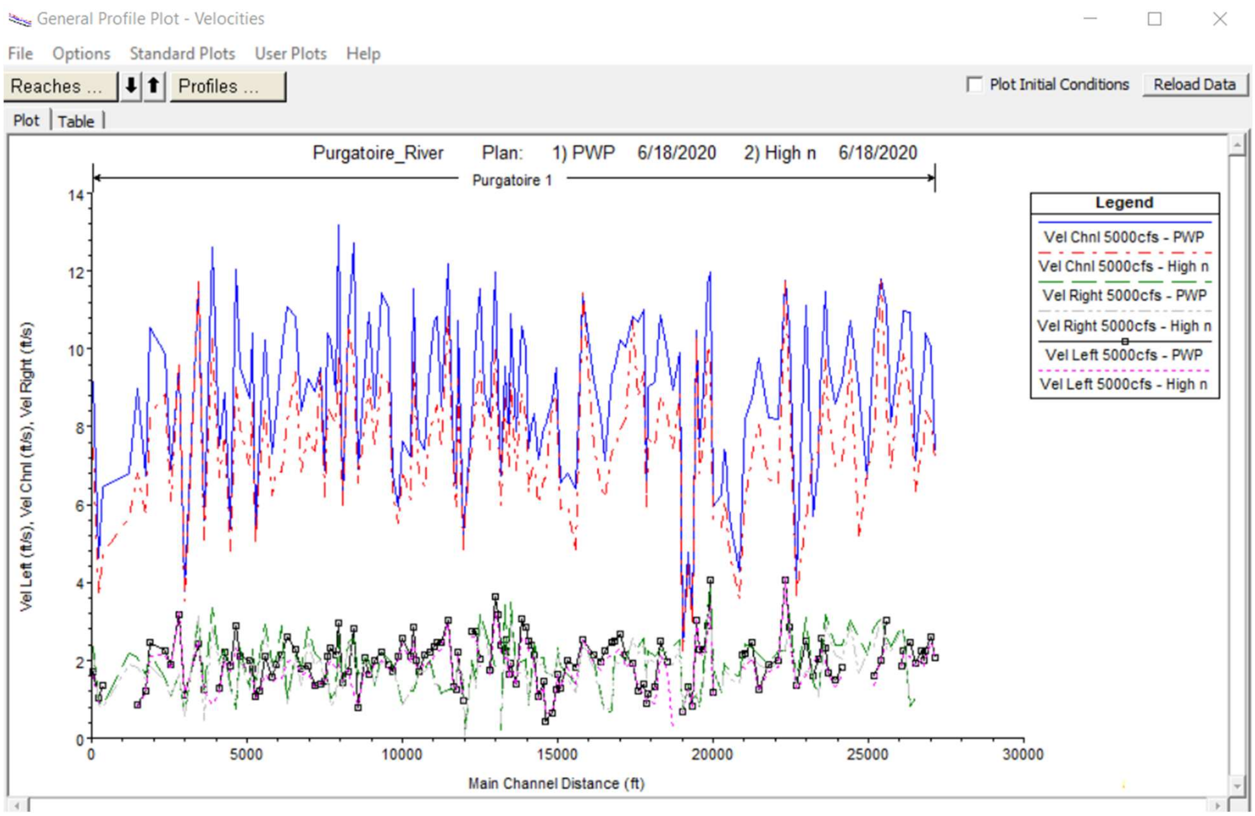

Figure 4-2. Velocity profiles through the study area (Reaches 1-6) for the 1D HEC-RAS model results with both low and high n-values shown.

4.4. 2D Model Results Velocity Map through the City of Trinidad

The 2D model results at 5,000 cfs were mapped using the HEC-RAS mapper tool. A raster was created to show the This calculates the velocity based on the 2D grid and can give a more accurate portrait of the variation in velocity across and longitudinally through a river. **Figure 4-3** shows the velocity maps for Reaches 1-3 and **figure 4-4** shows the computed velocities for reaches 4 and 5. The range of velocities is in good agreement with the range of velocity for the 1D model. The high velocities ranging from 10- 11.5 ft/s are highlighted by the red color. The high velocities are focused in the low flow channel where sediment transport will also be high. The low velocities are shown in blue and, as expected, show in the overbank areas. These low velocity areas are locations where sediment deposition will occur. More discussion of sediment transport will be provided in section 5.0.

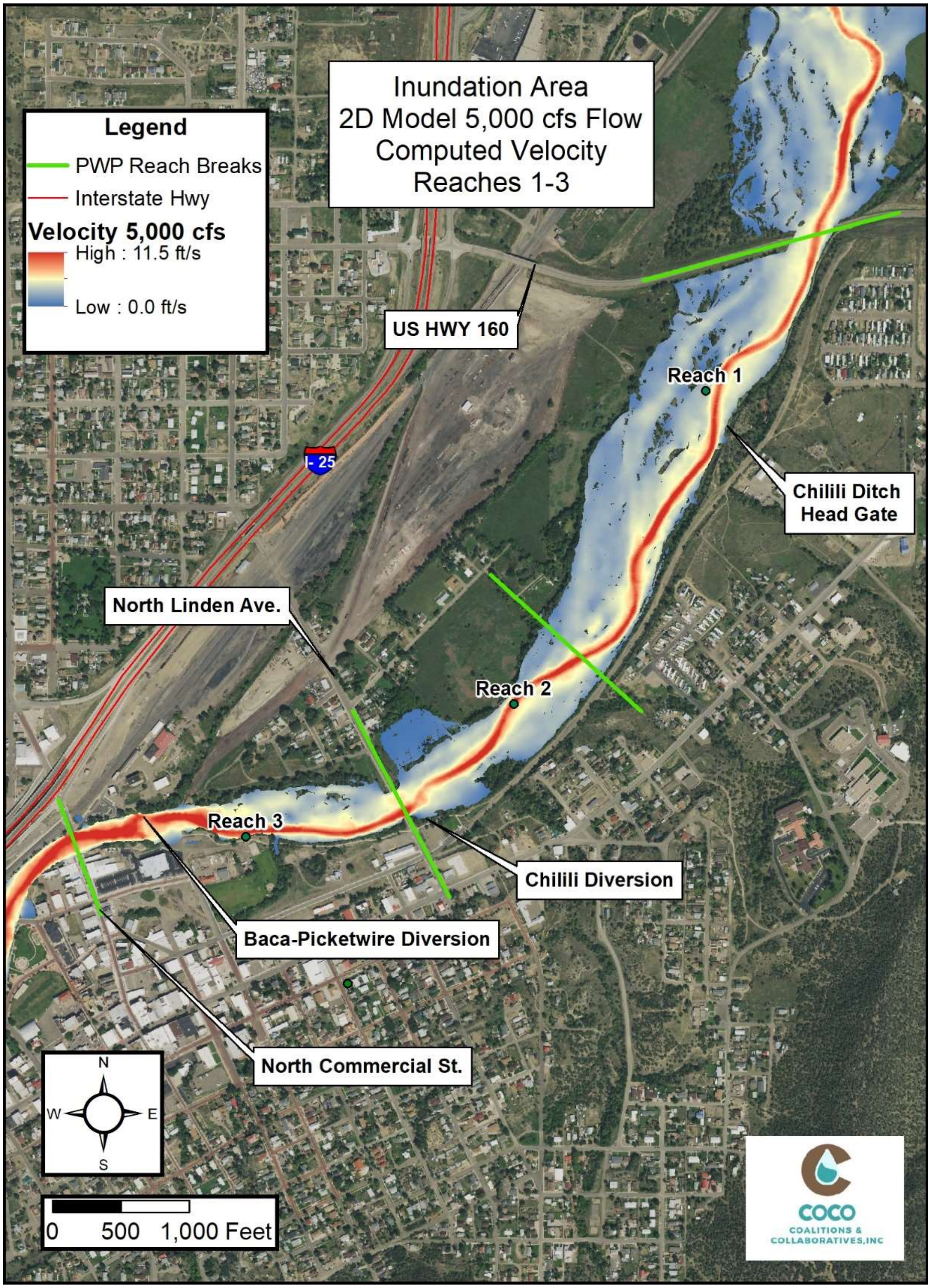

Figure 4-3. 2D HEC-RAS model computed velocities through reaches 1-3.

Figure 4-4. 2D HEC-RAS model computed velocities through reaches 4-5.

4.5. 1D Area of Inundation Maps through the Study Area.

The area of inundation for flows ranging from 500 cfs to 5,000 cfs using the 1D model are shown in figures 4-5 to 4-30.

Figure 4-5. Map showing the depth of inundation for a 1D model run at 500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 1-3.

Figure 4-6. Map showing the depth of inundation for a 1D model run at 500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 4-5.

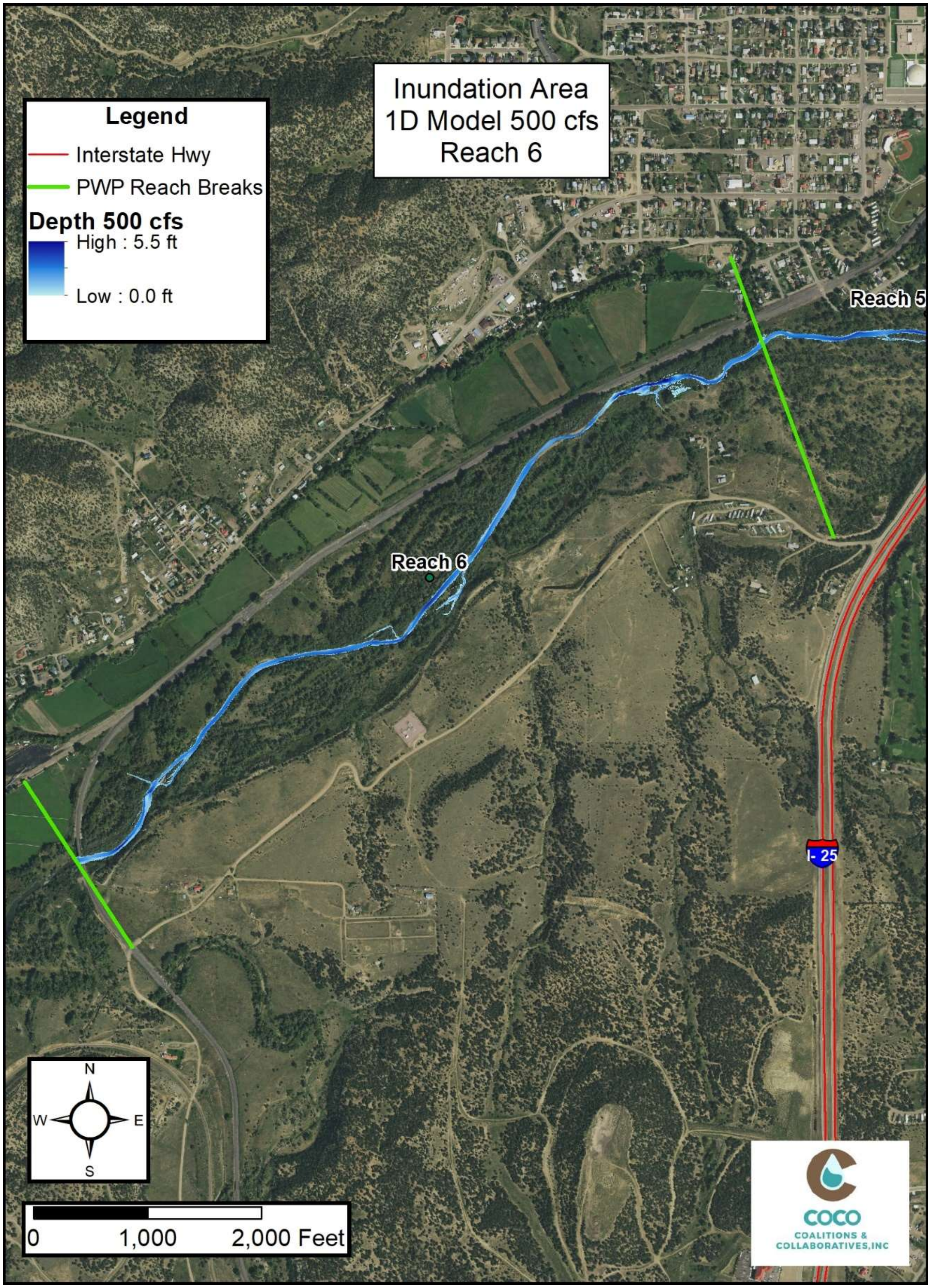

Figure 4-7 Map showing the depth of inundation for a 1D model run at 500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reach 6.

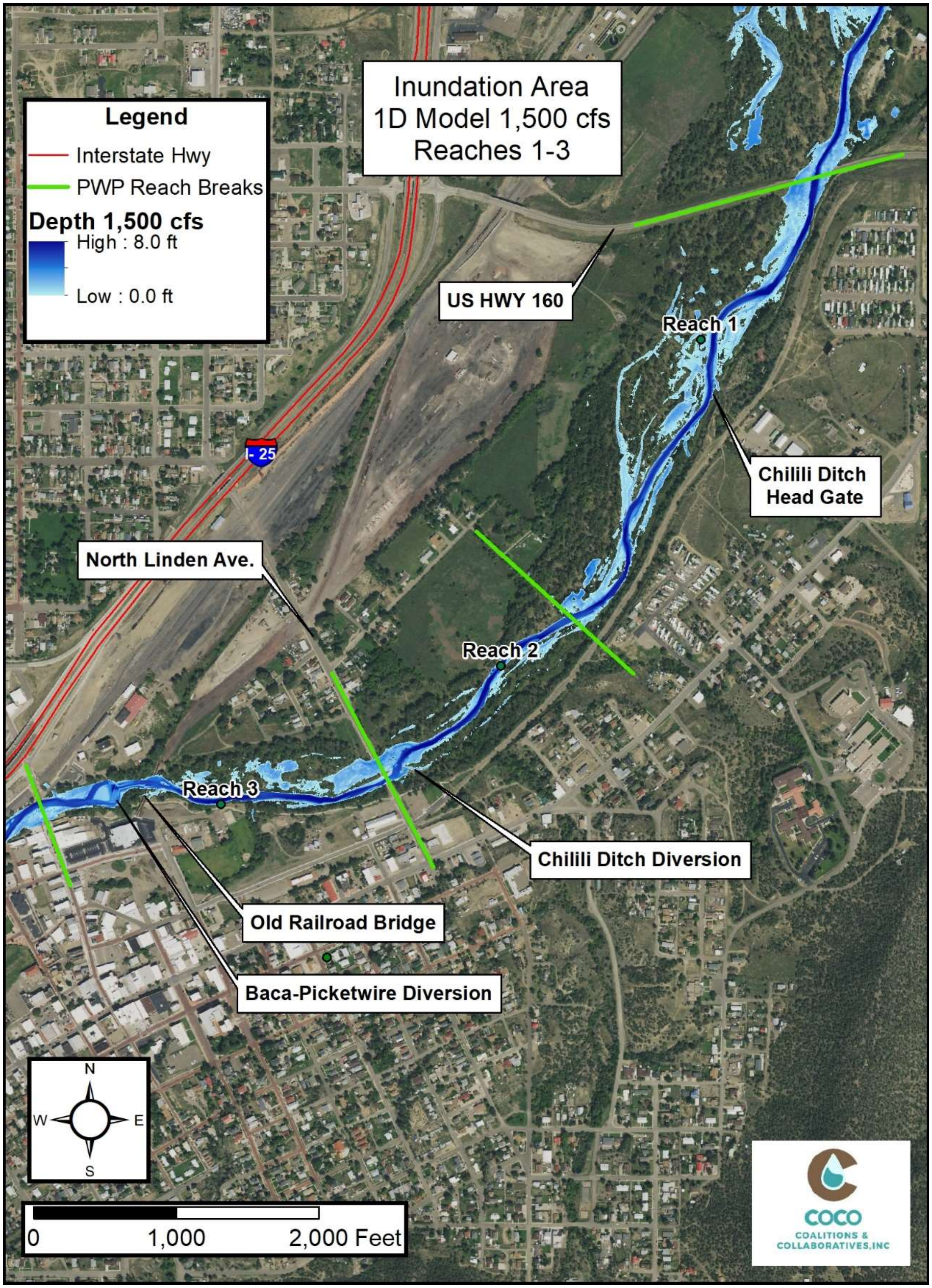

Figure 4-8. Map showing the depth of inundation for a 1D model run at 1,500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 1-3.

Figure 4-9. Map showing the depth of inundation for a 1D model run at 1,500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 4-5.

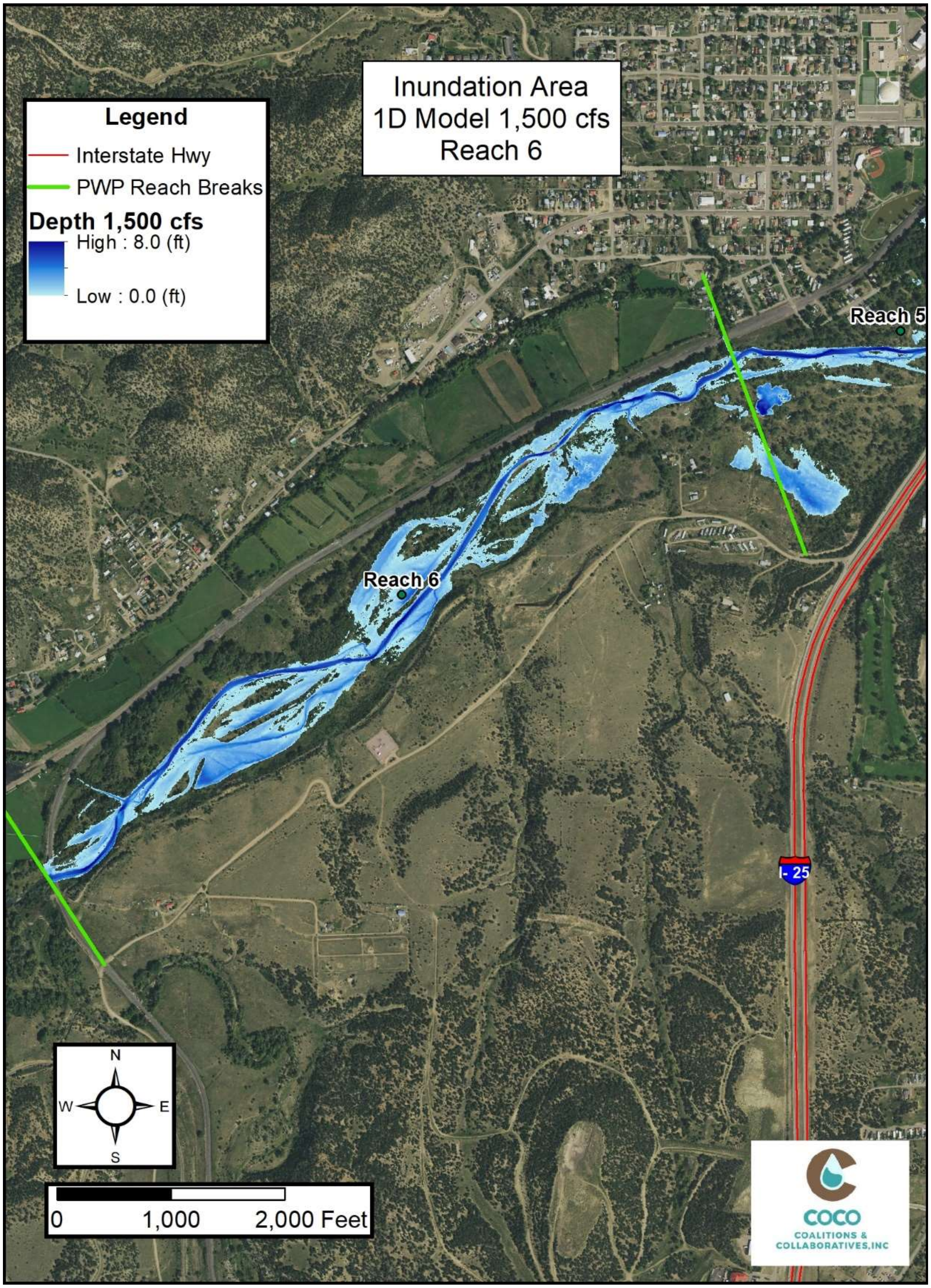

Figure 4-10. Map showing the depth of inundation for a 1D model run at 1,500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reach 6.

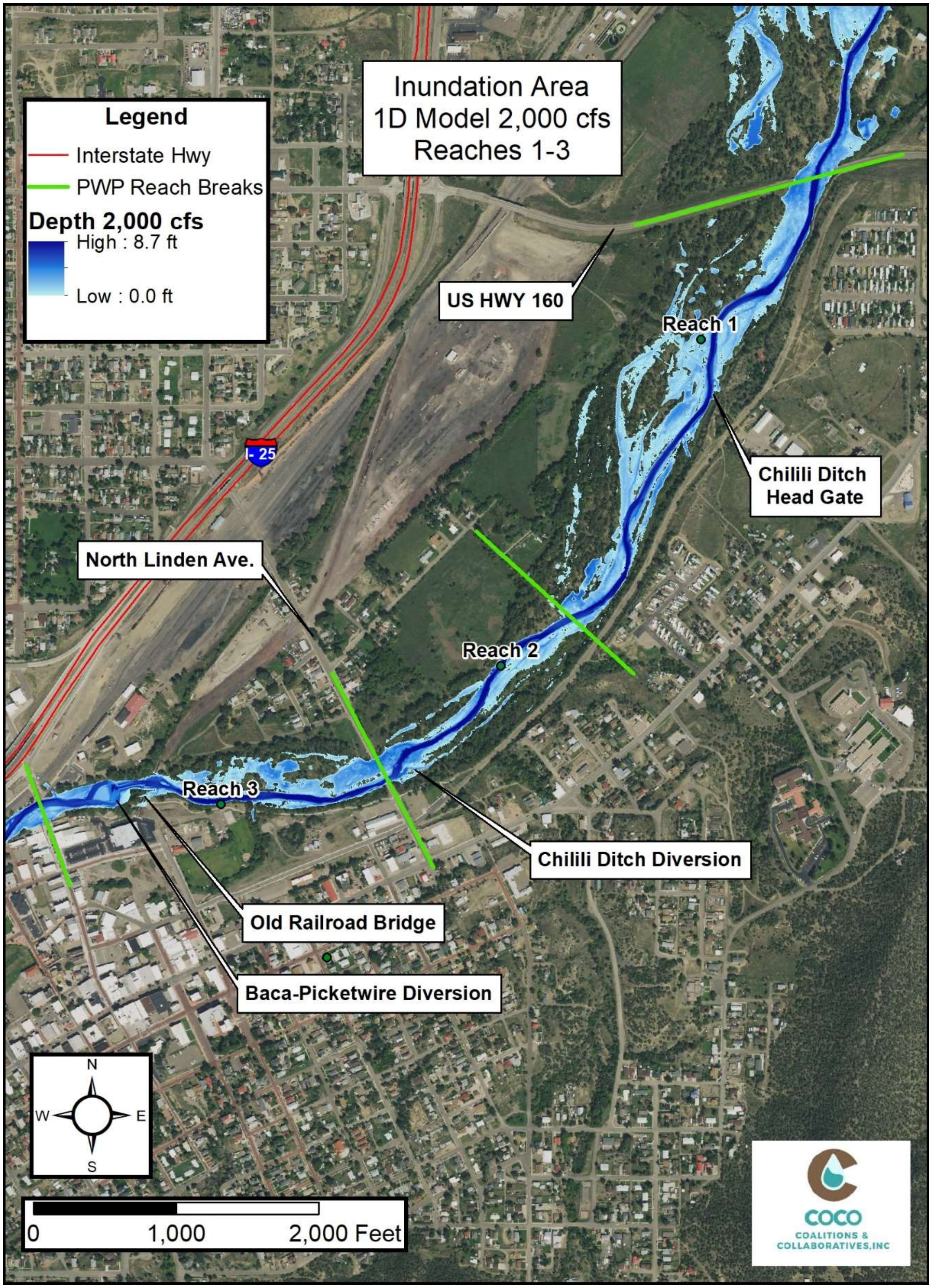

Figure 4-11. Map showing the depth of inundation for a 1D model run at 2,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 1-3.

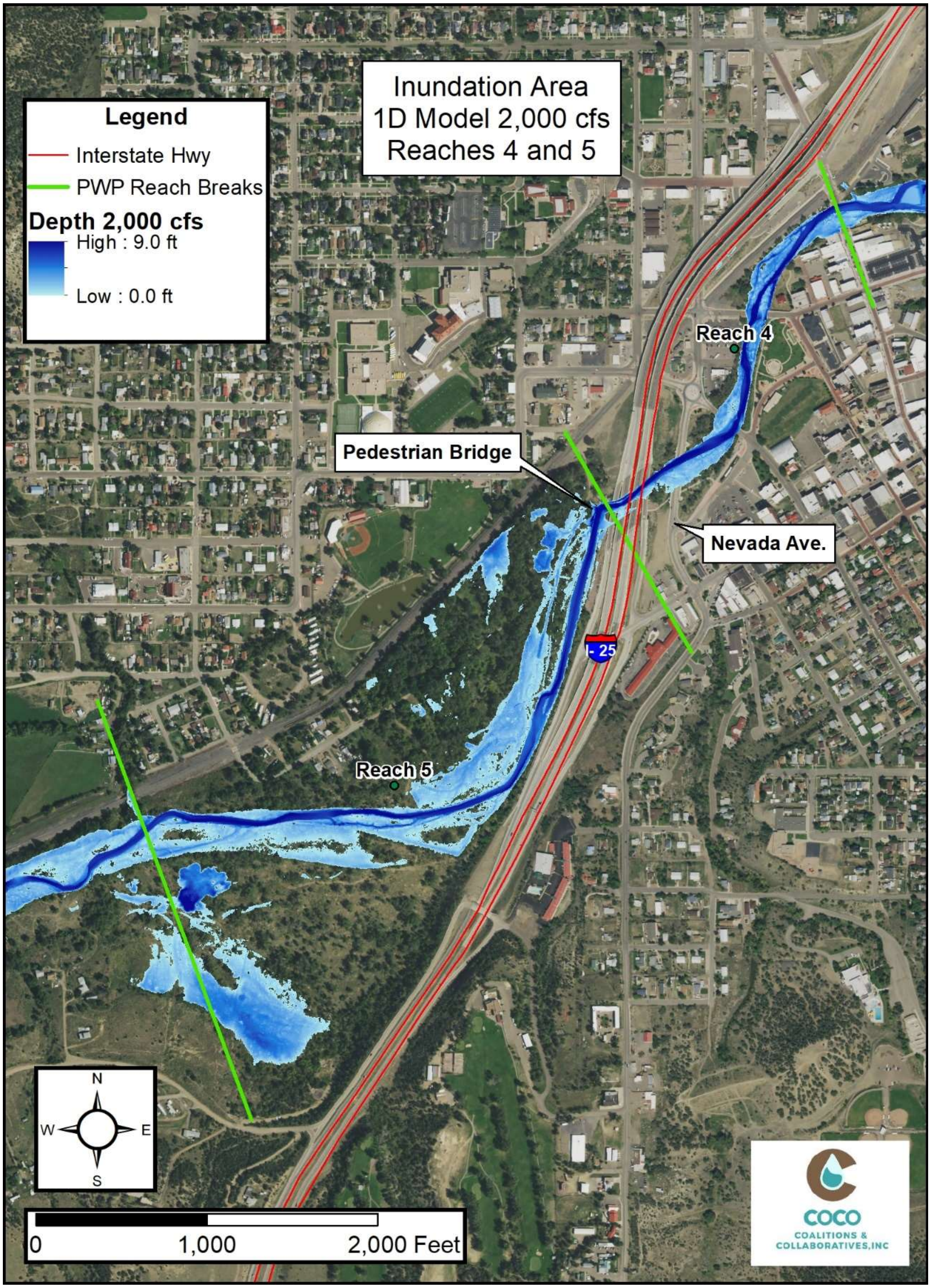

Figure 4-12. Map showing the depth of inundation for a 1D model run at 2,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 4-5.

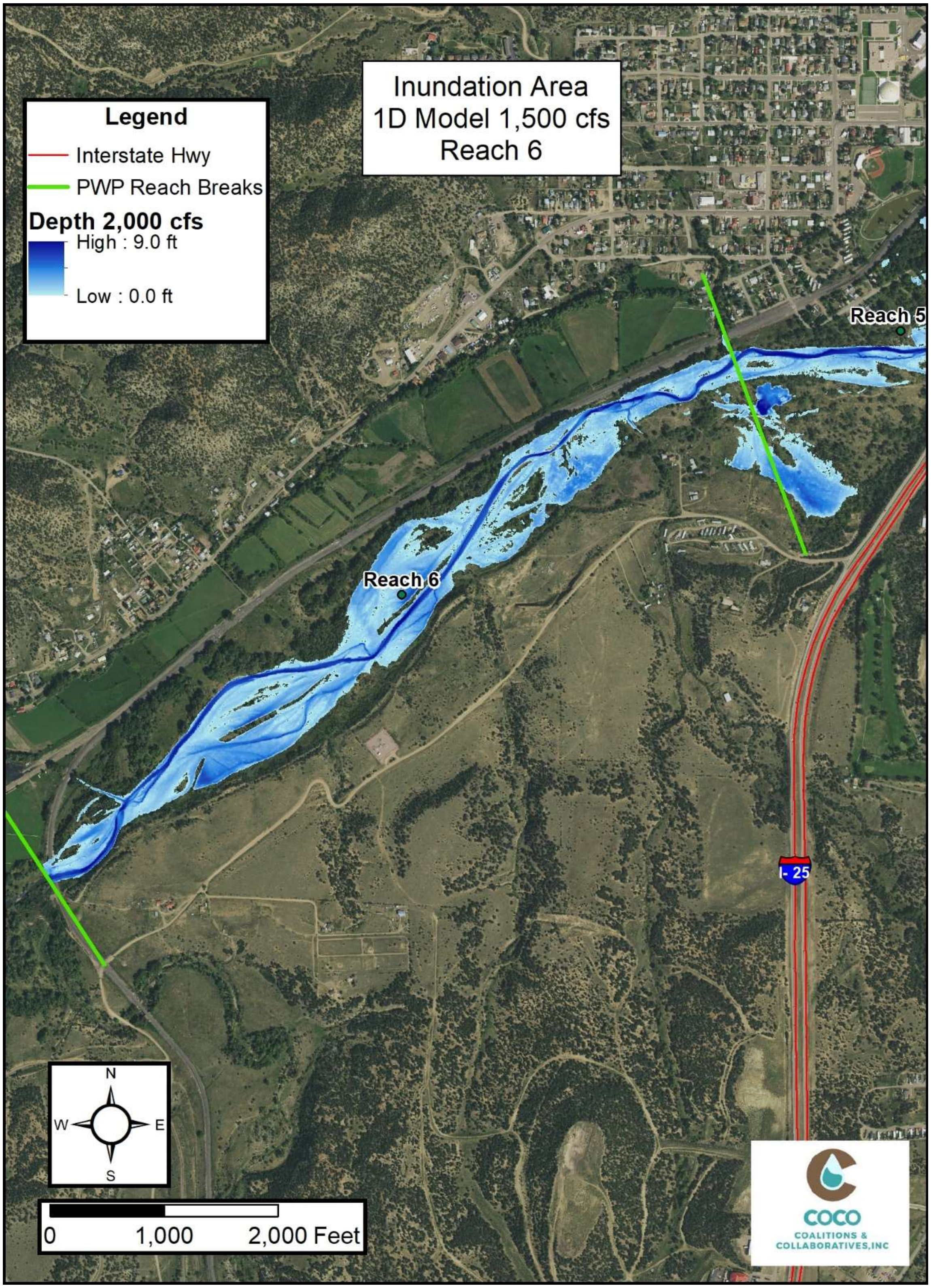

Figure 4-13. Map showing the depth of inundation for a 1D model run at 2,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reach 6.

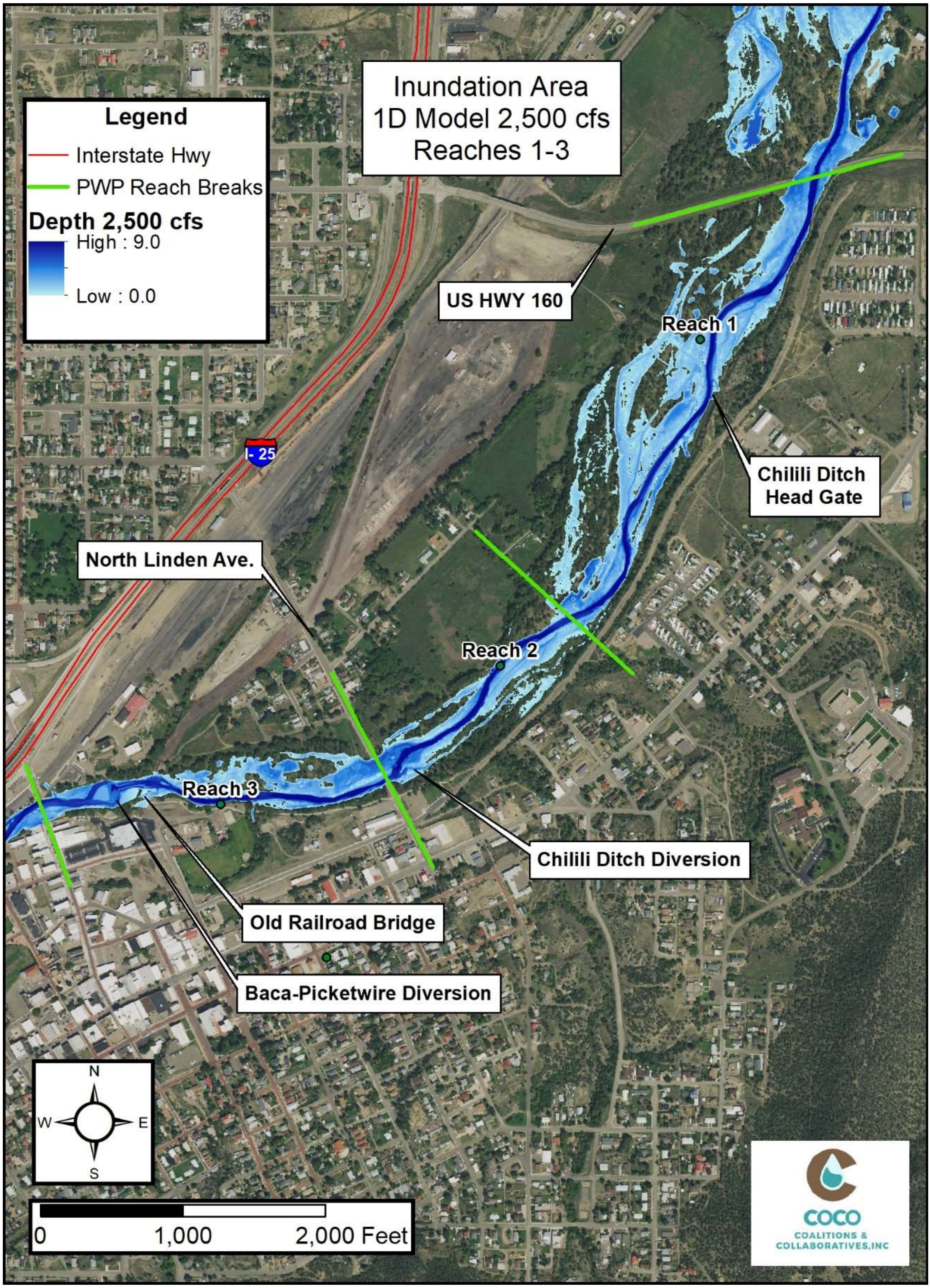

Figure 4-14. Map showing the depth of inundation for a 1D model run at 2,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 1-3.

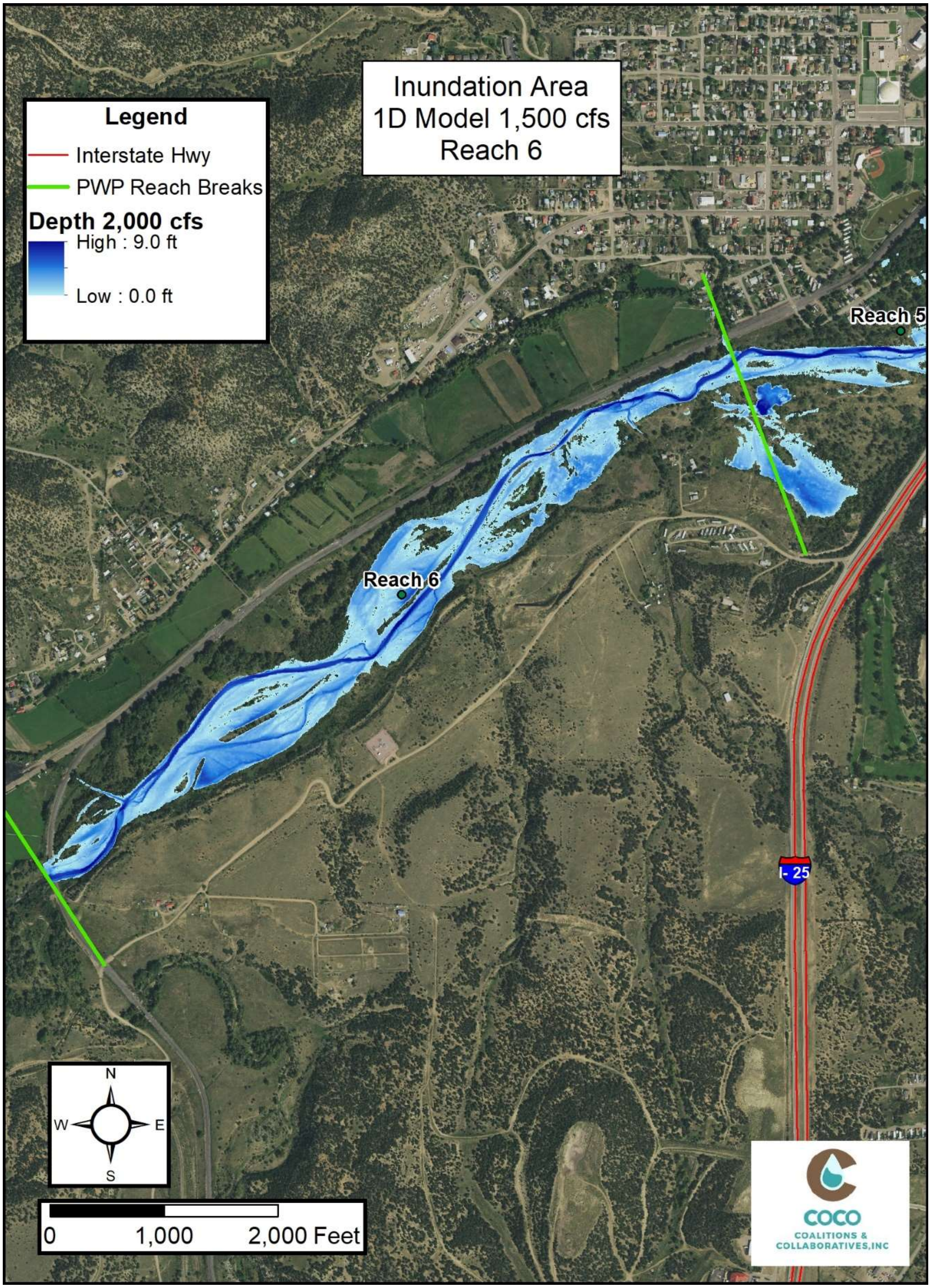

Figure 4-15. Map showing the depth of inundation for a 1D model run at 2,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reach 6.

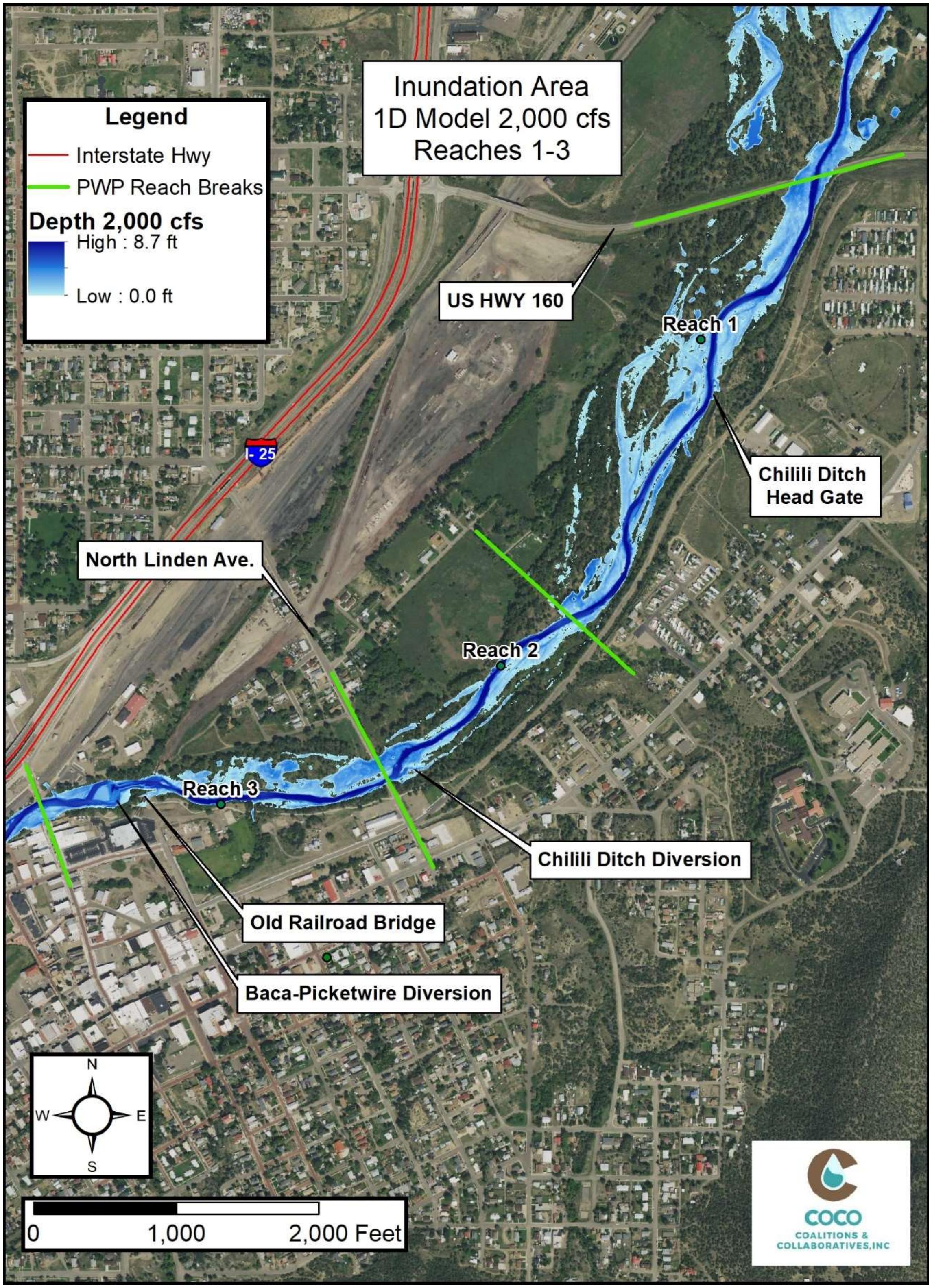

Figure 4-16. Map showing the depth of inundation for a 1D model run at 2,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 1-3.

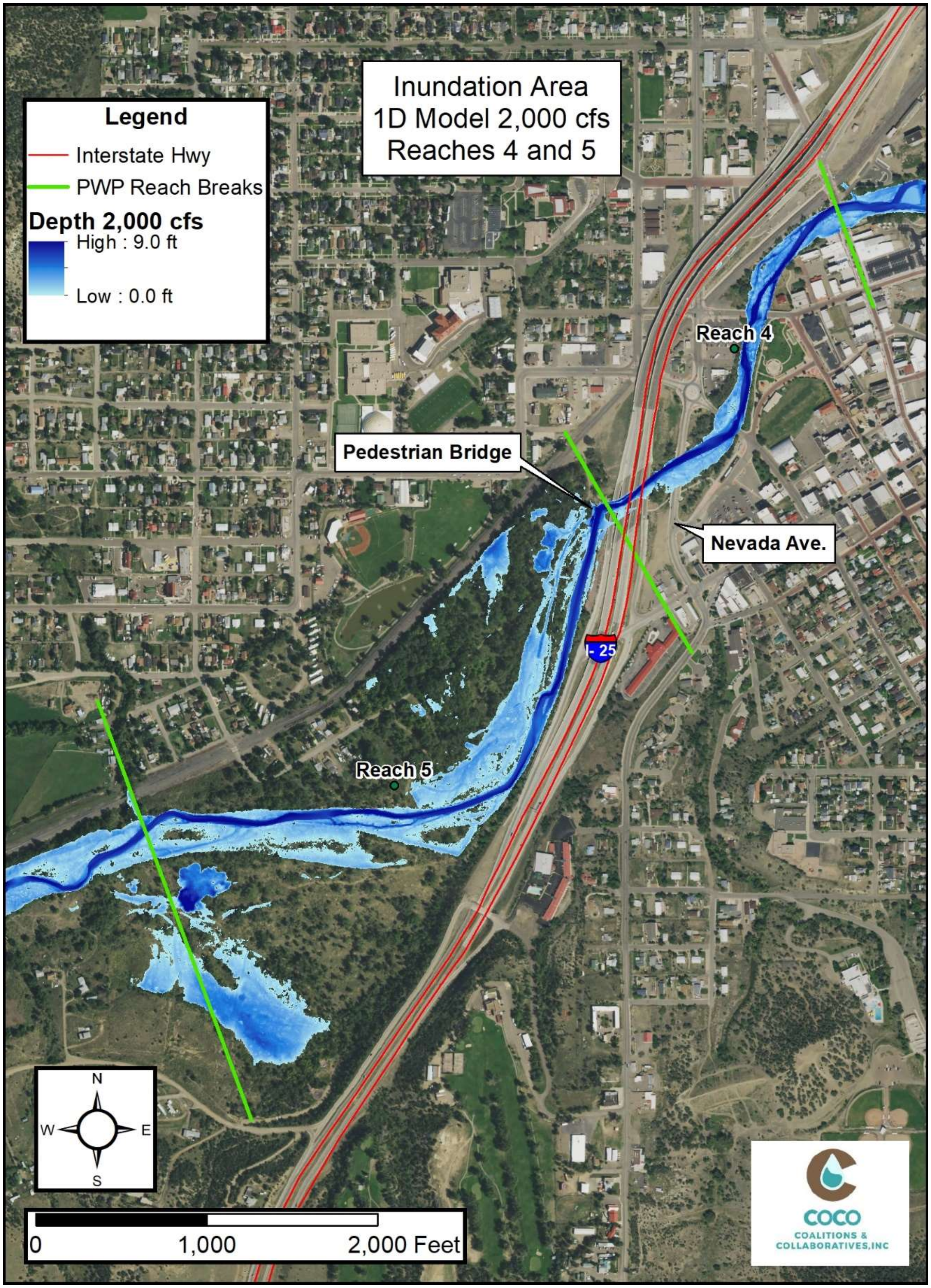

Figure 4-17. Map showing the depth of inundation for a 1D model run at 2,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 4-5.

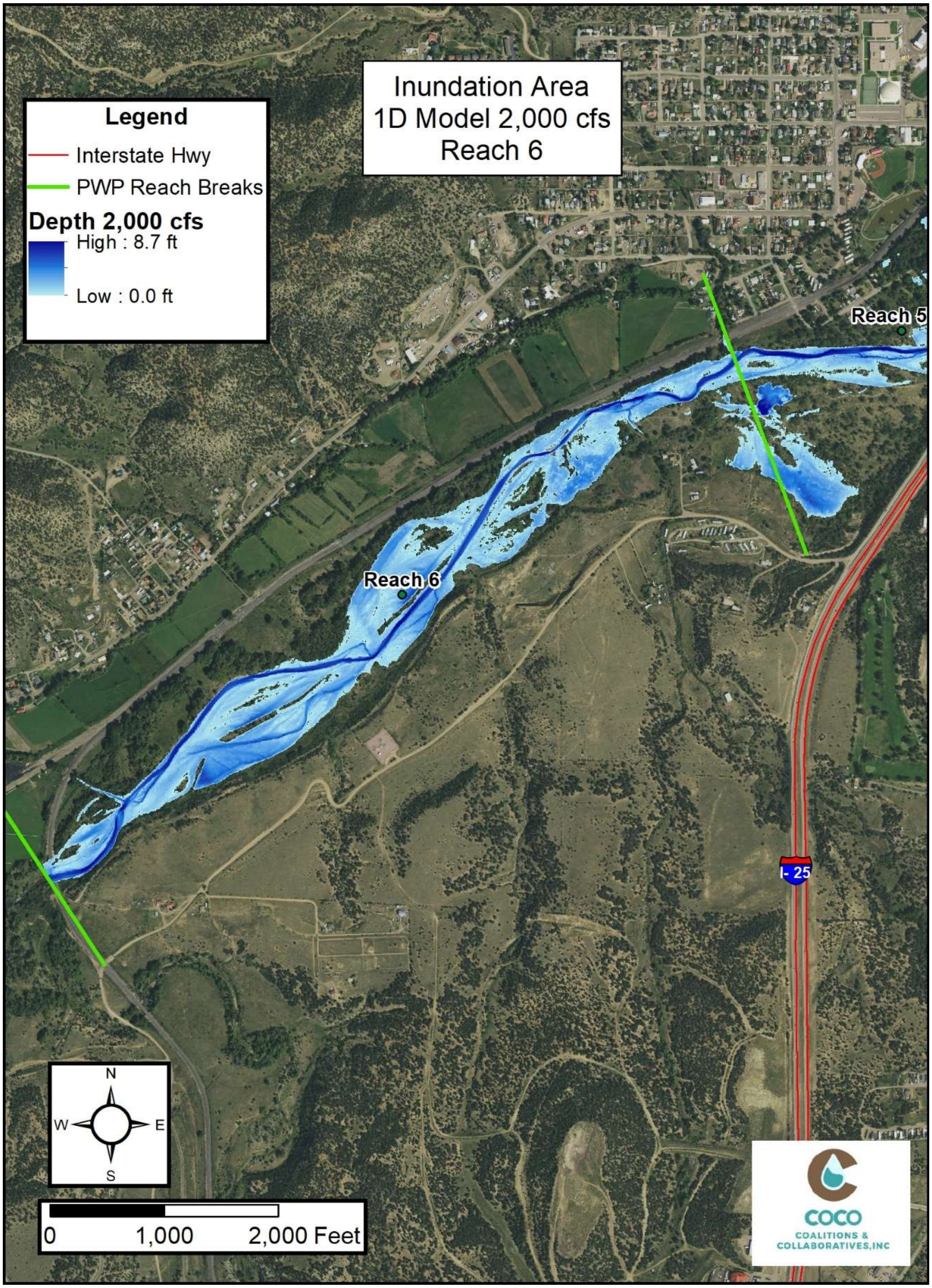

Figure 4-18. Map showing the depth of inundation for a 1D model run at 2,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reach 6.

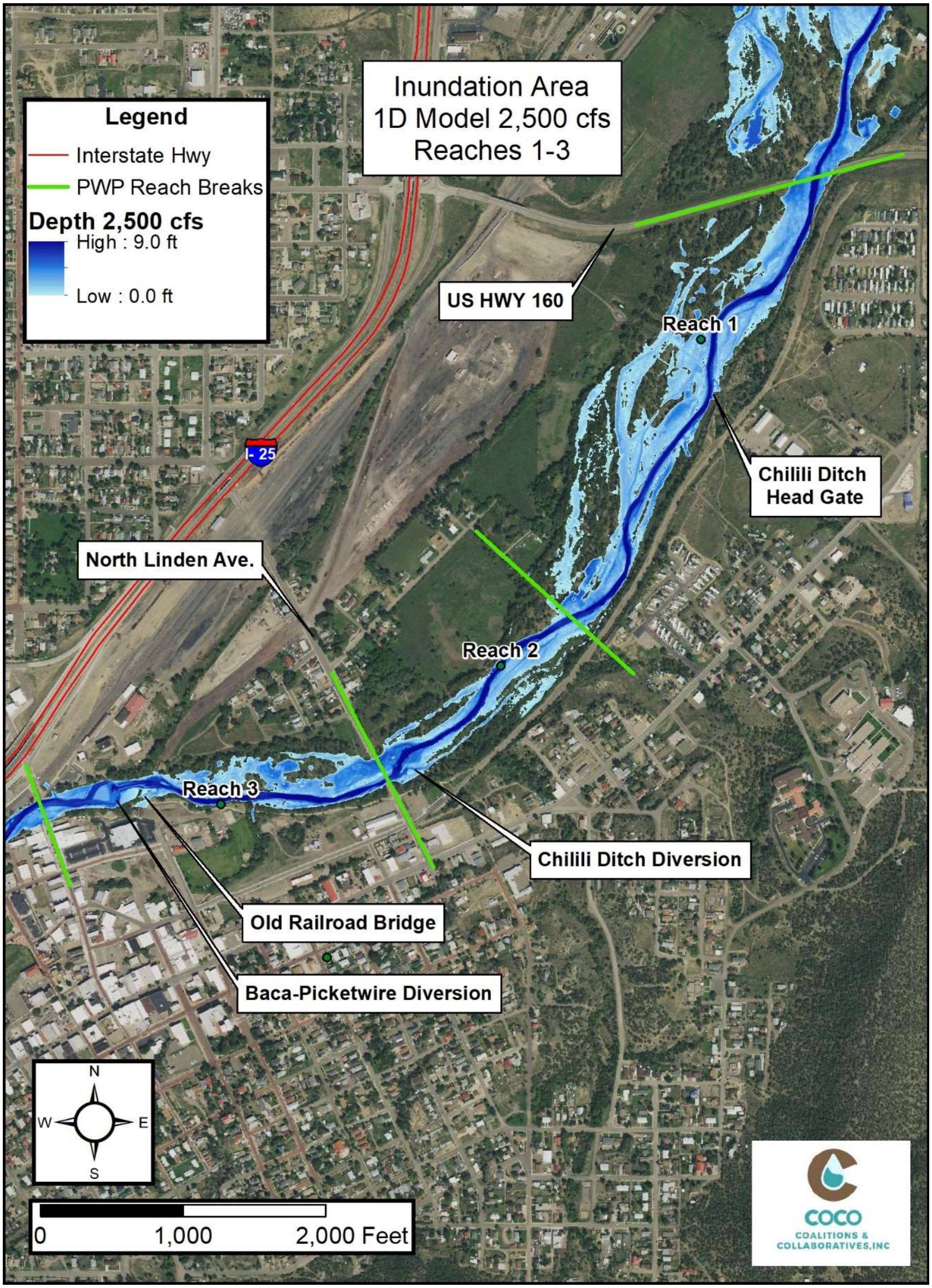

Figure 4-19. Map showing the depth of inundation for a 1D model run at 2,500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 1-3.

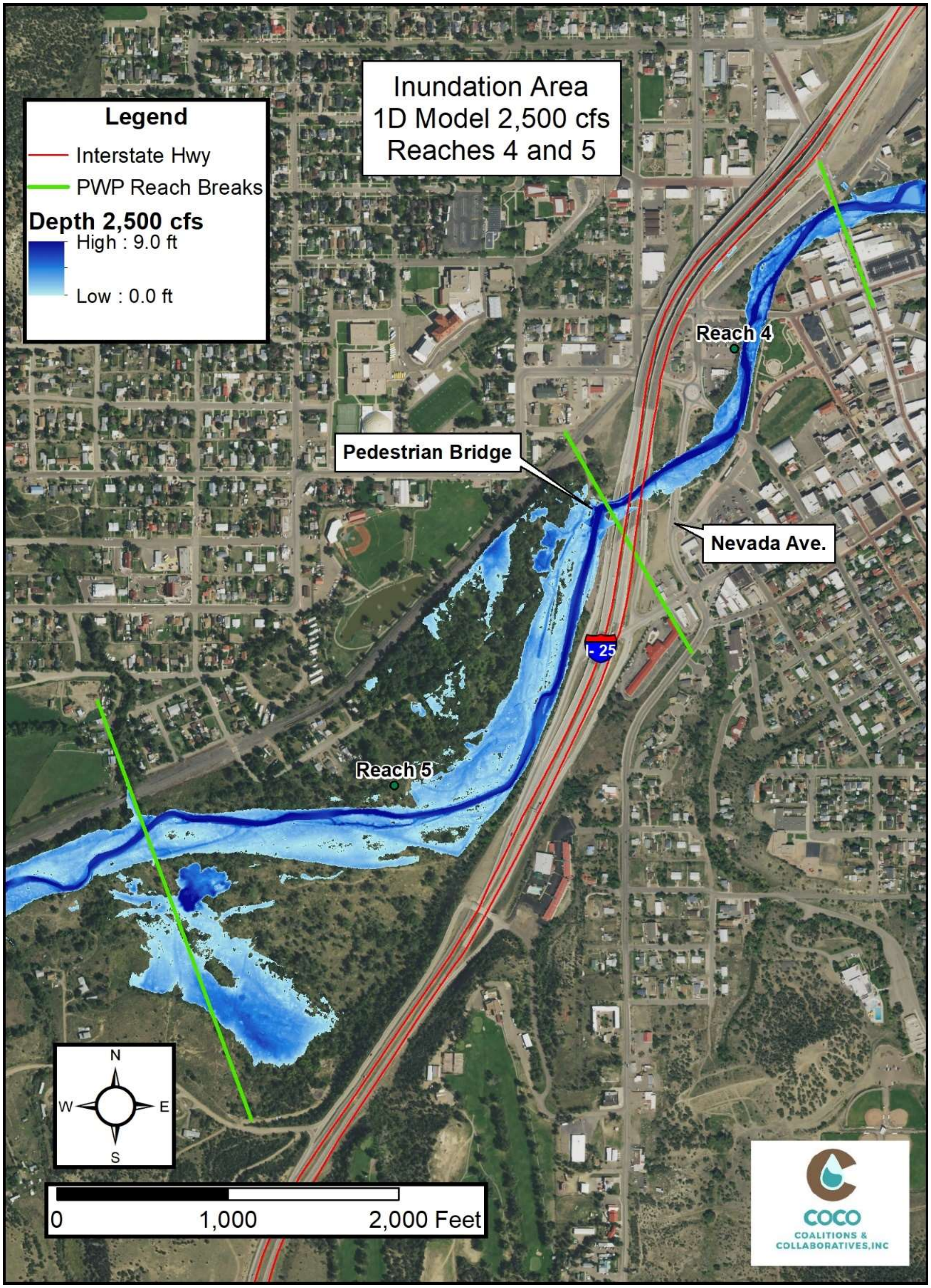

Figure 4-20. Map showing the depth of inundation for a 1D model run at 2,500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 4-5.

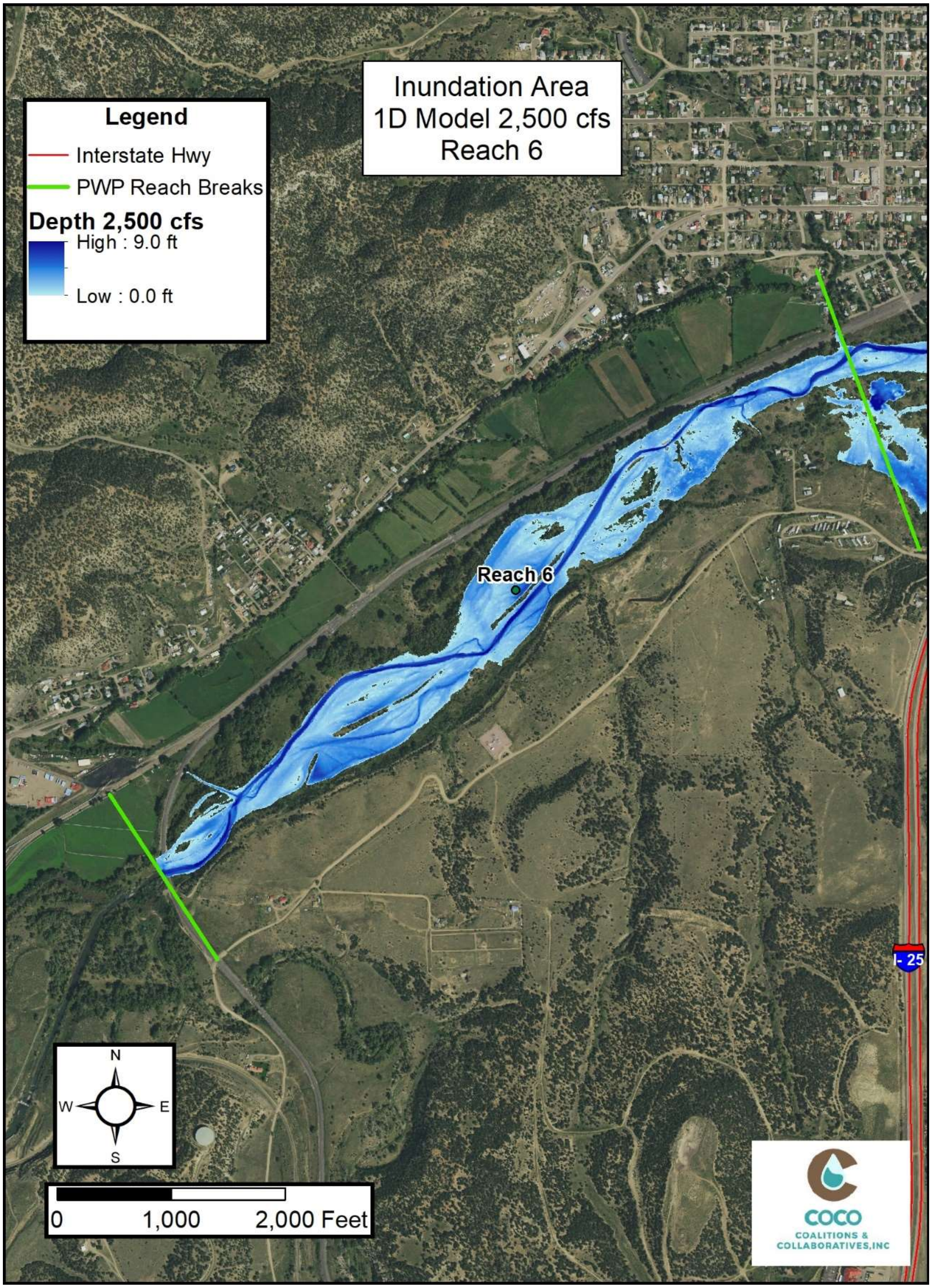

Figure 4-21. Map showing the depth of inundation for a 1D model run at 2,500 cfs using a channel n value of 0.033 and overbank value of 0.075 in reach 6.

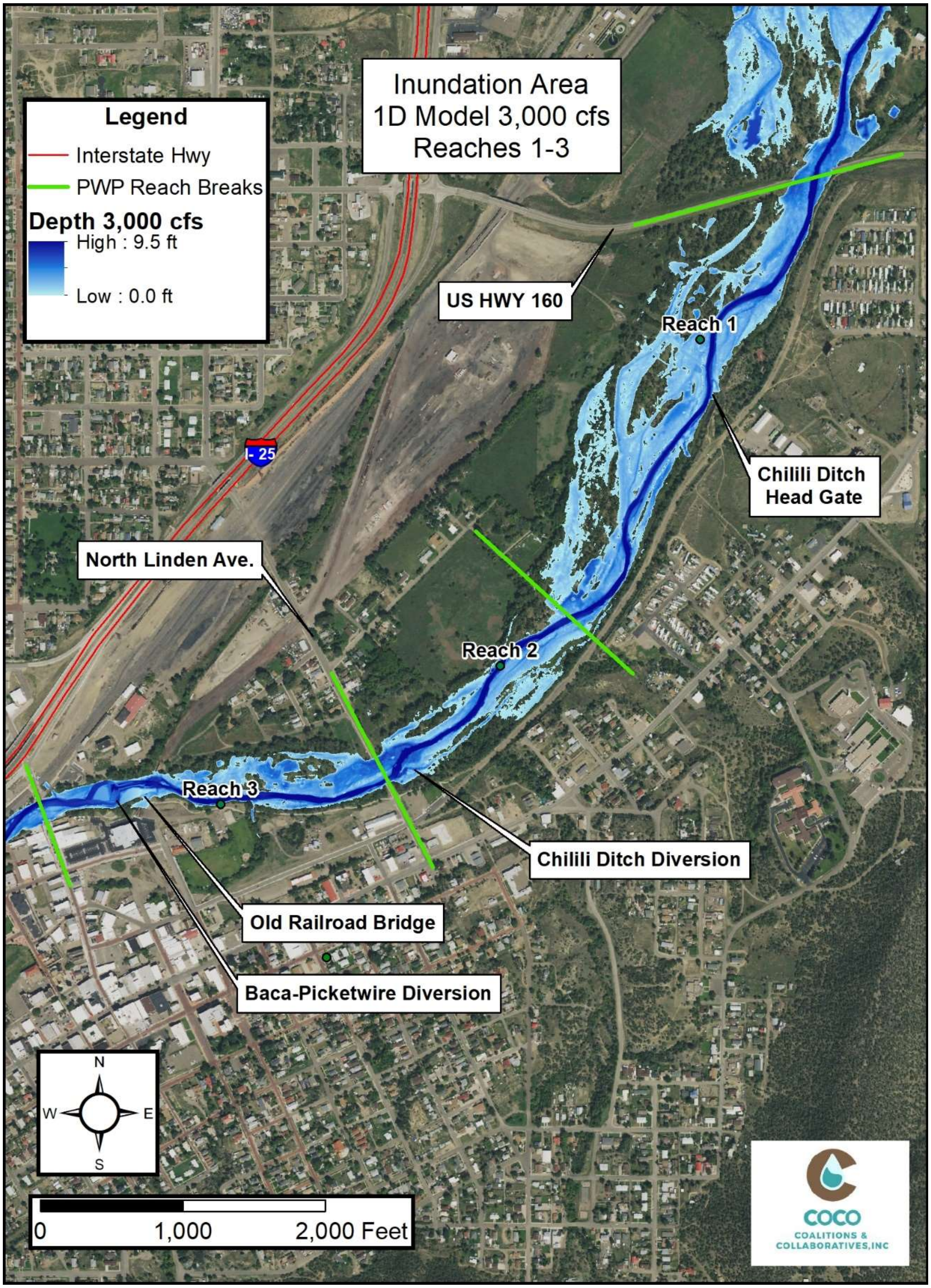

Figure 4-22. Map showing the depth of inundation for a 1D model run at 3,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 1-3.

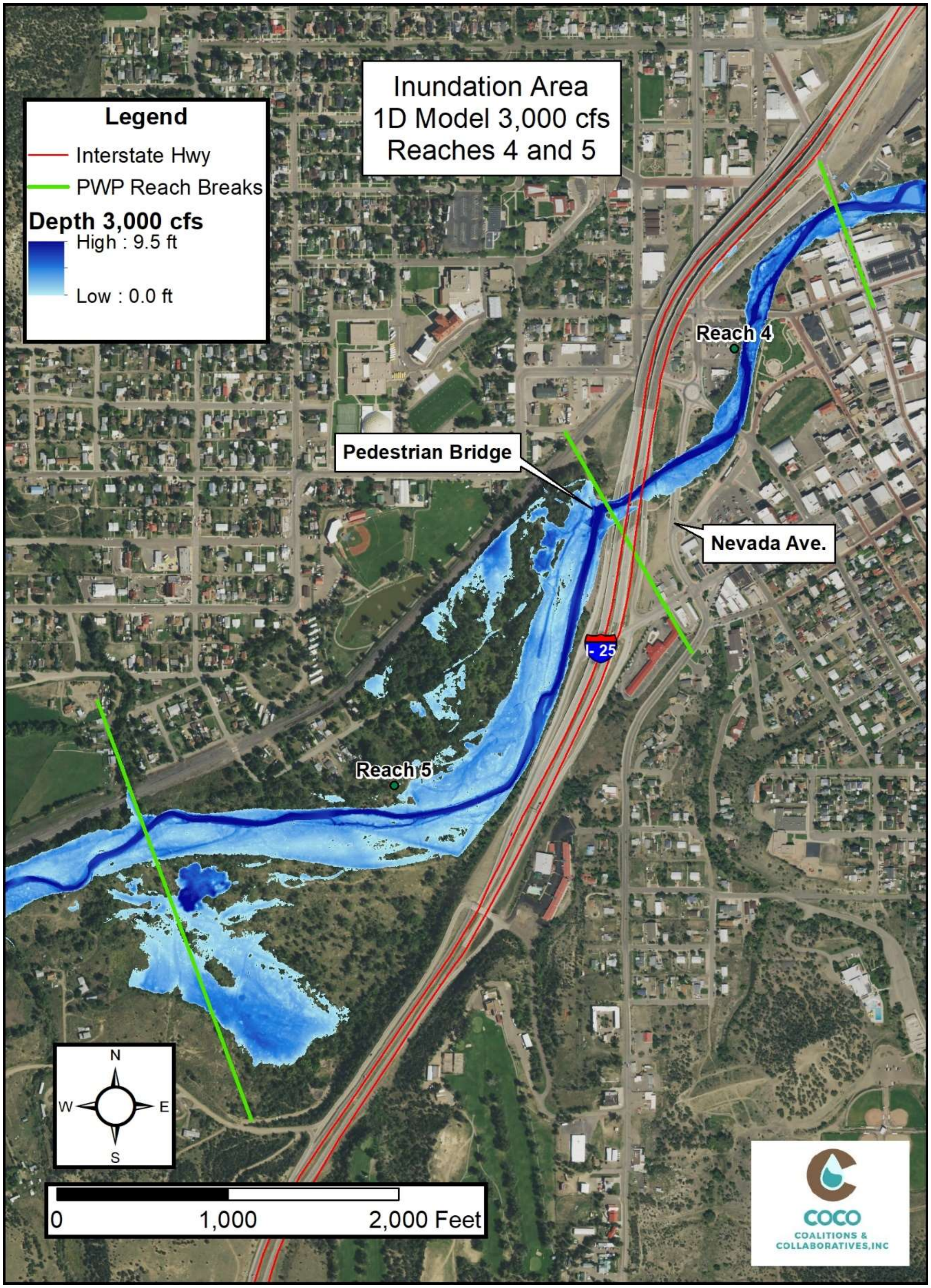

Figure 4-23. Map showing the depth of inundation for a 1D model run at 3,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 4-5.

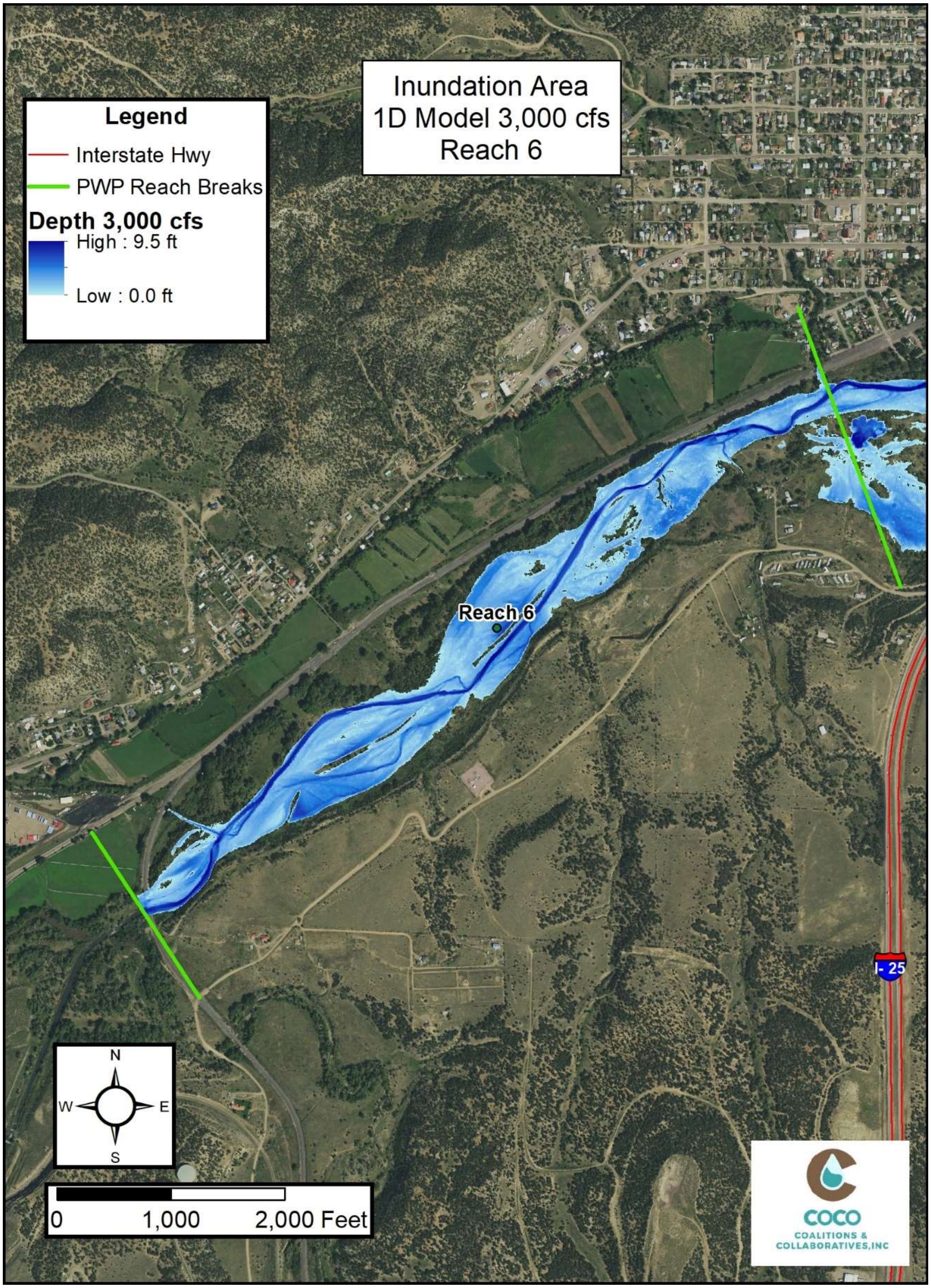

Figure 4-24. Map showing the depth of inundation for a 1D model run at 3,000 cfs using a channel n value of 0.033 and overbank value of 0.075 in reaches 6.

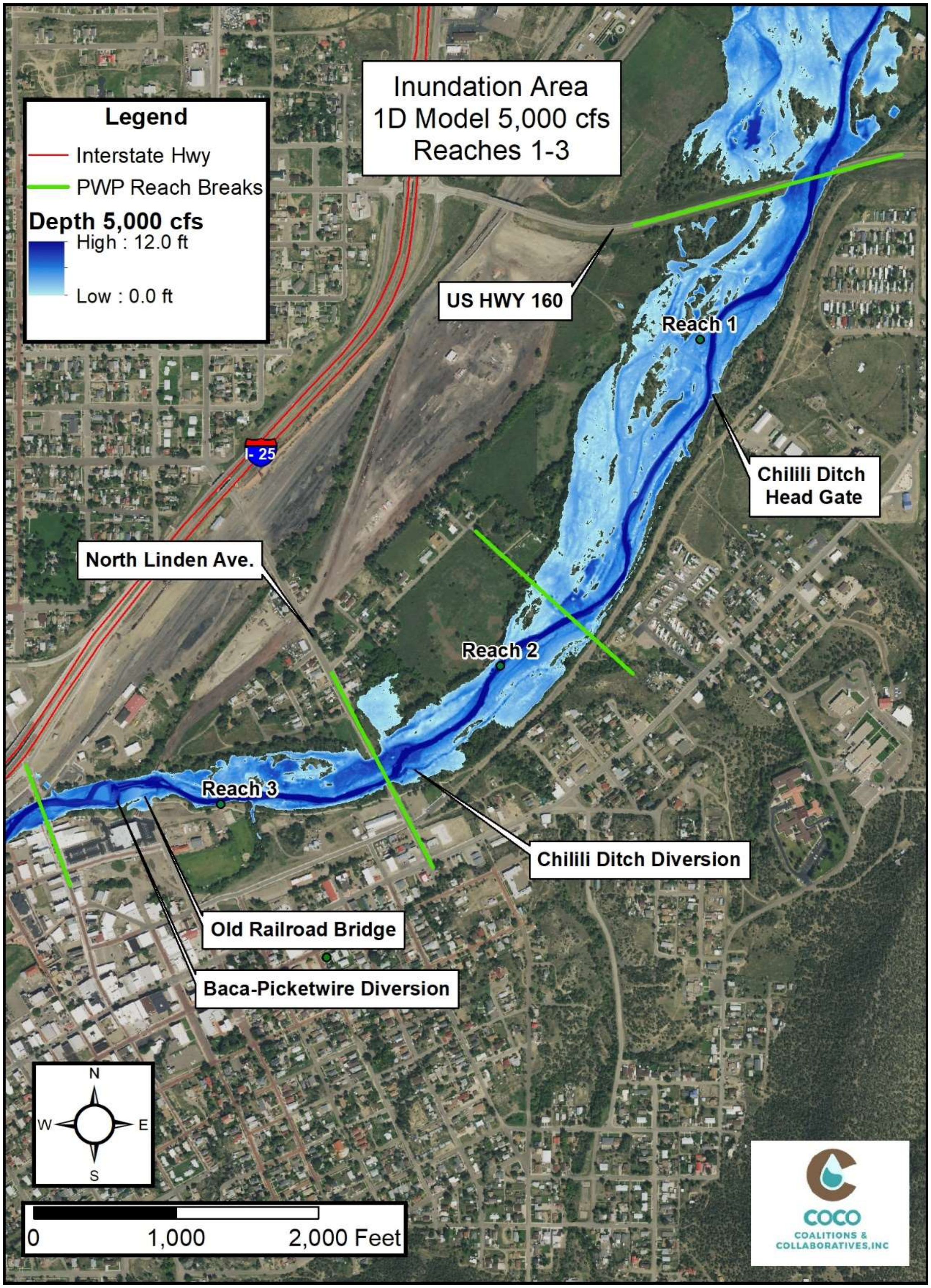

Figure 4-25. Map showing the depth of inundation for a 1D model run at 5,000 cfs using a channel n value of 0.033 and overbank value of 0.075.

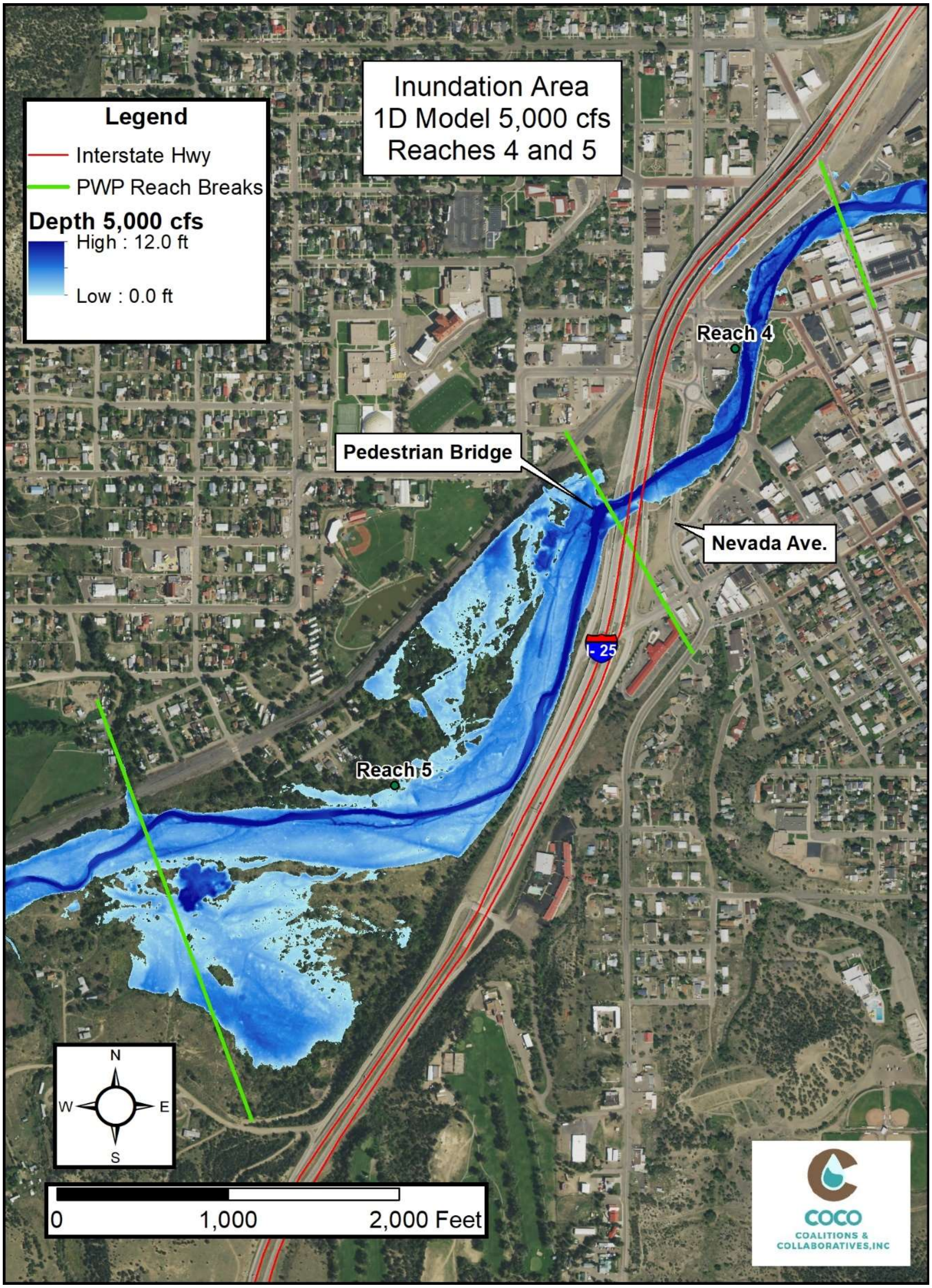

Figure 4-26. Map showing the depth of inundation in reaches 4 and 5 for a 1D model run at 5,000 cfs using a channel n value of 0.033 and overbank value of 0.075.

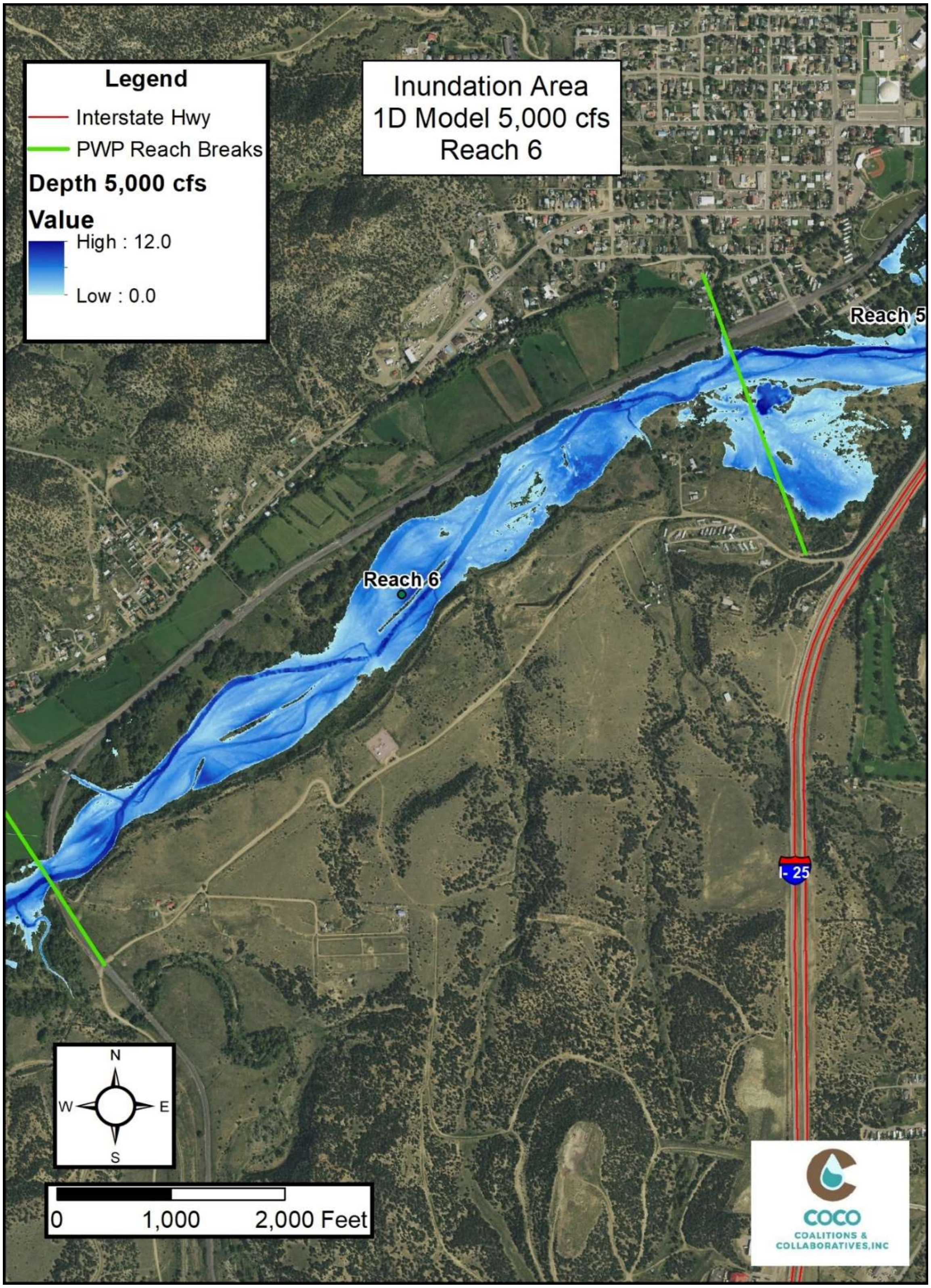

Figure 4-27. Map showing the depth of inundation in reach 6 for a 1D model run at 5,000 cfs using a channel n value of 0.033 and overbank value of 0.075.

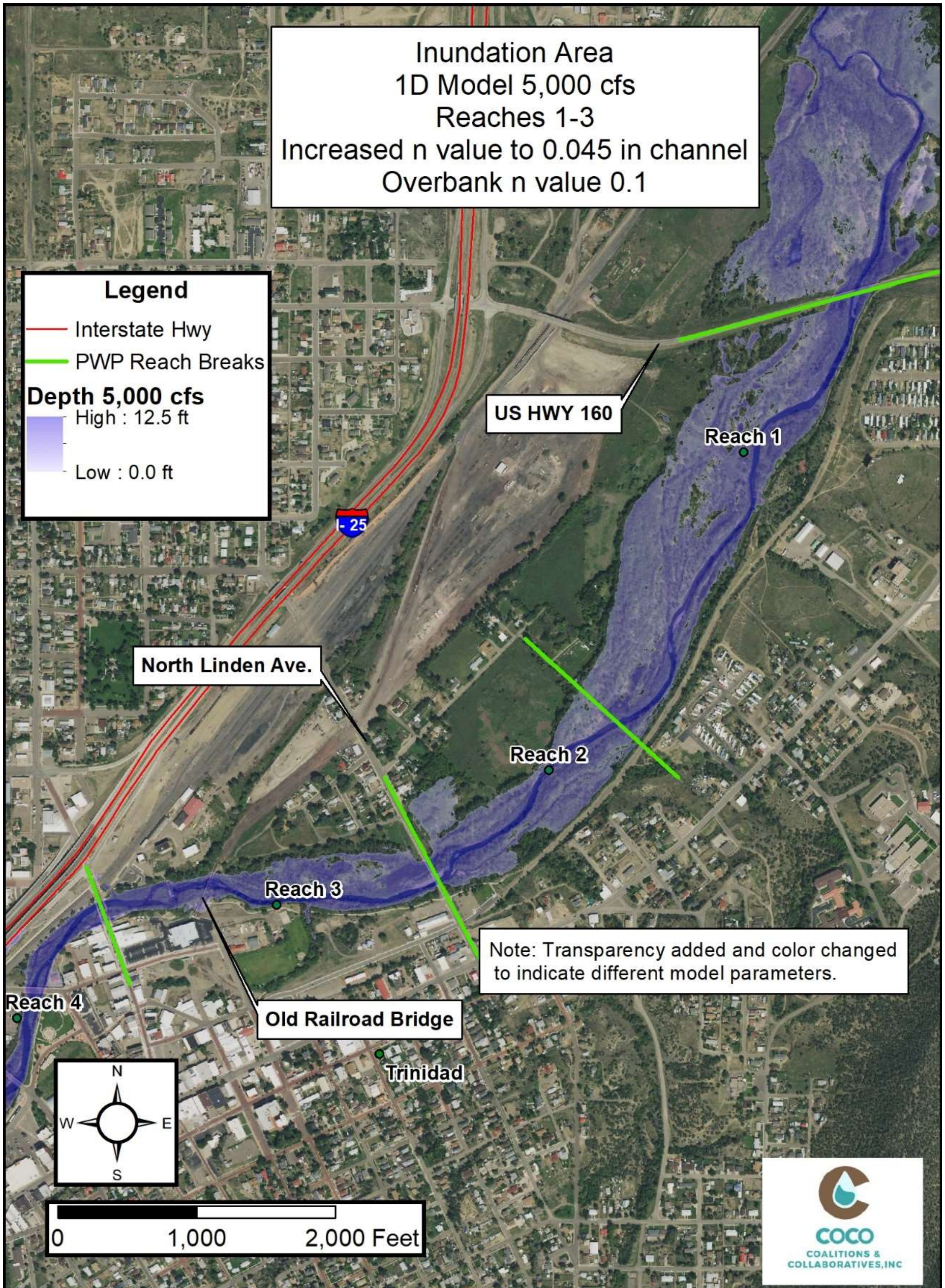

Figure 4-28. Map showing the depth of inundation in reaches 1, 2 and 3 for a 1D model run at 5,000 cfs using a channel n value of 0.045 and overbank value of 0.10.

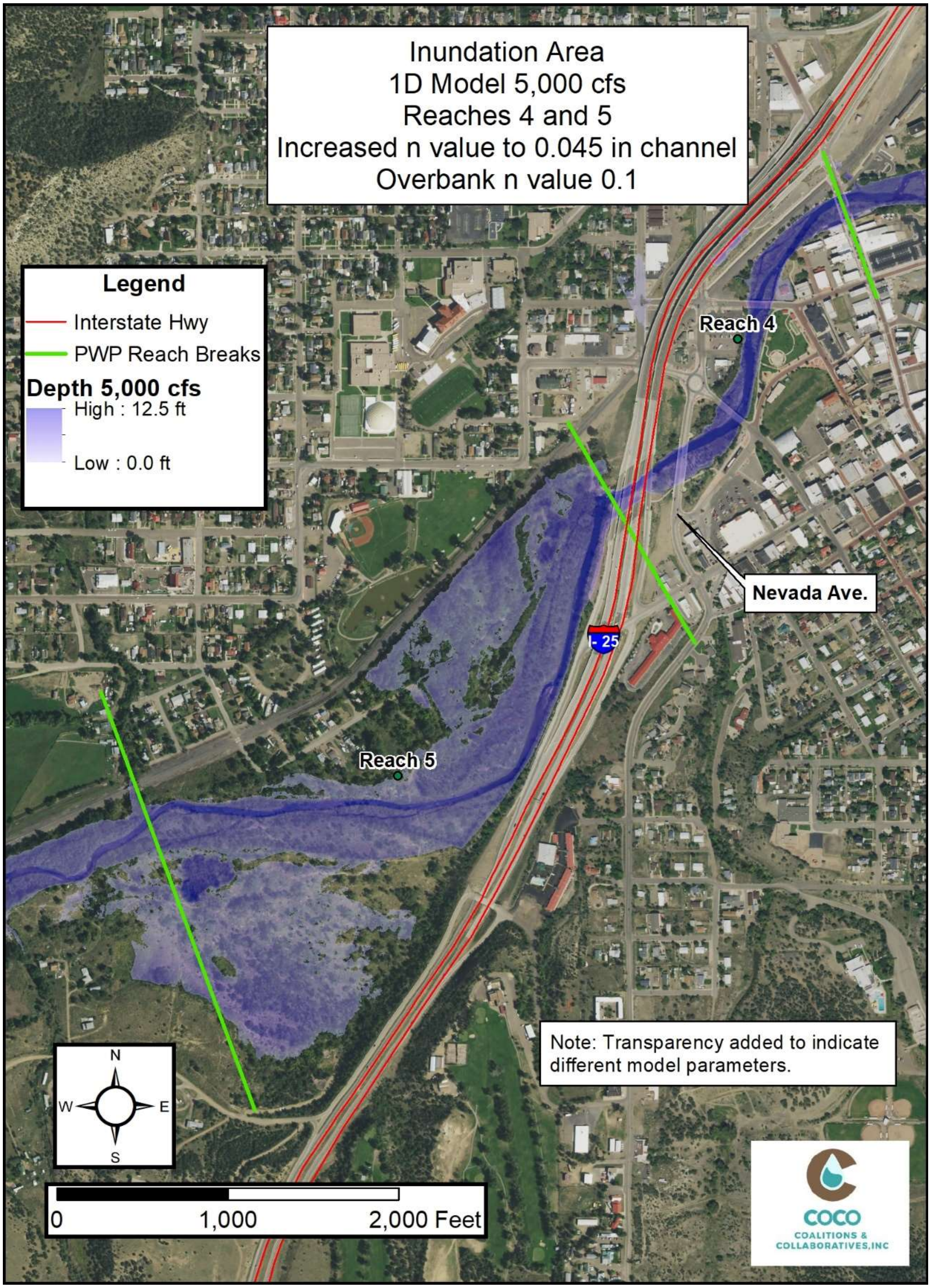

Figure 4-29. Map showing the depth of inundation in reaches 4 and 5 for a 1D model run at 5,000 cfs using a channel n value of 0.033 and overbank value of 0.075.

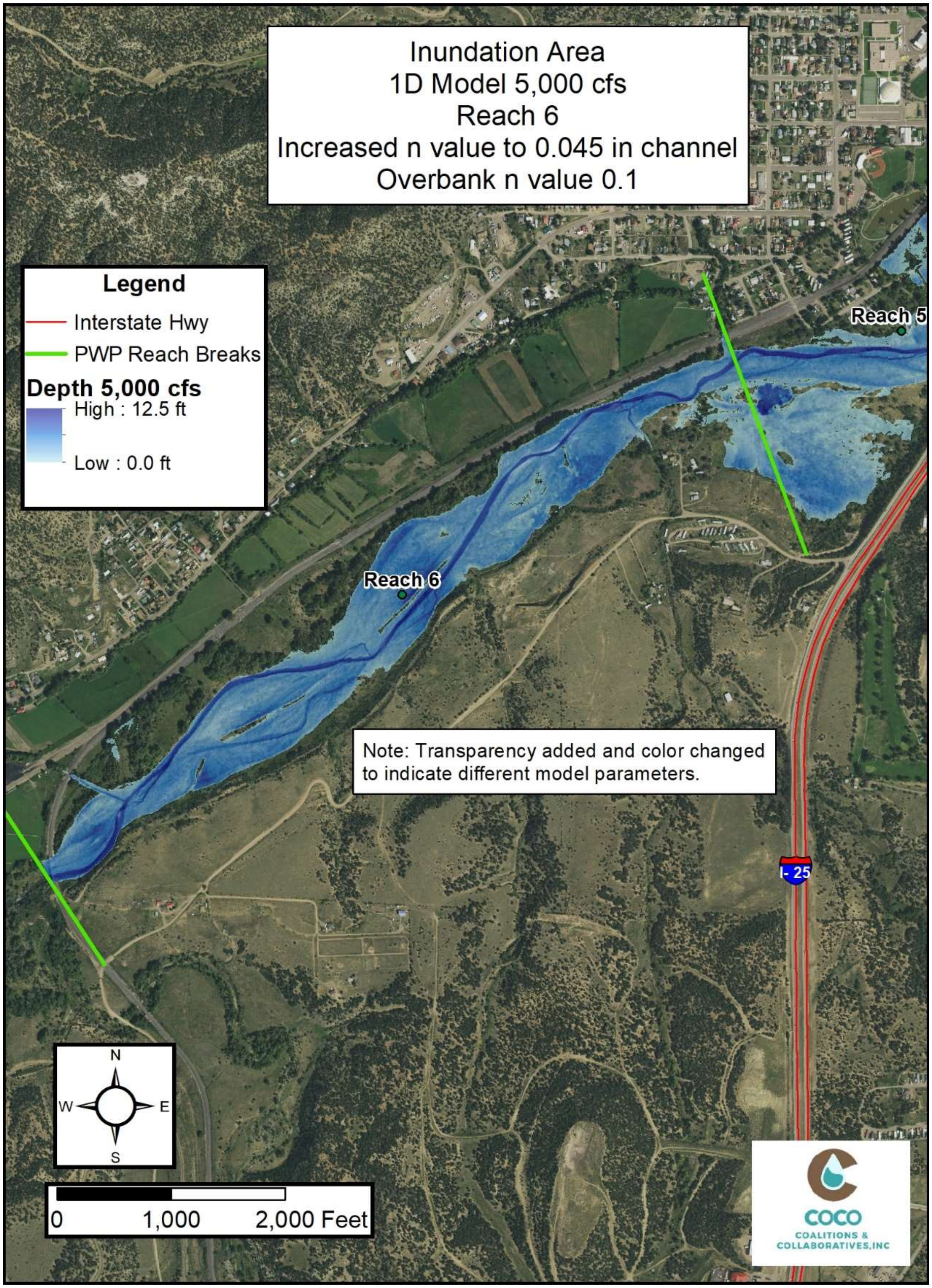

Figure 4-30. Map showing the depth of inundation in reach 6 for a 1D model run at 5,000 cfs using a channel n value of 0.045 and overbank value of 0.10.

4.6. 2D Area of Inundation Maps through the Study Area.

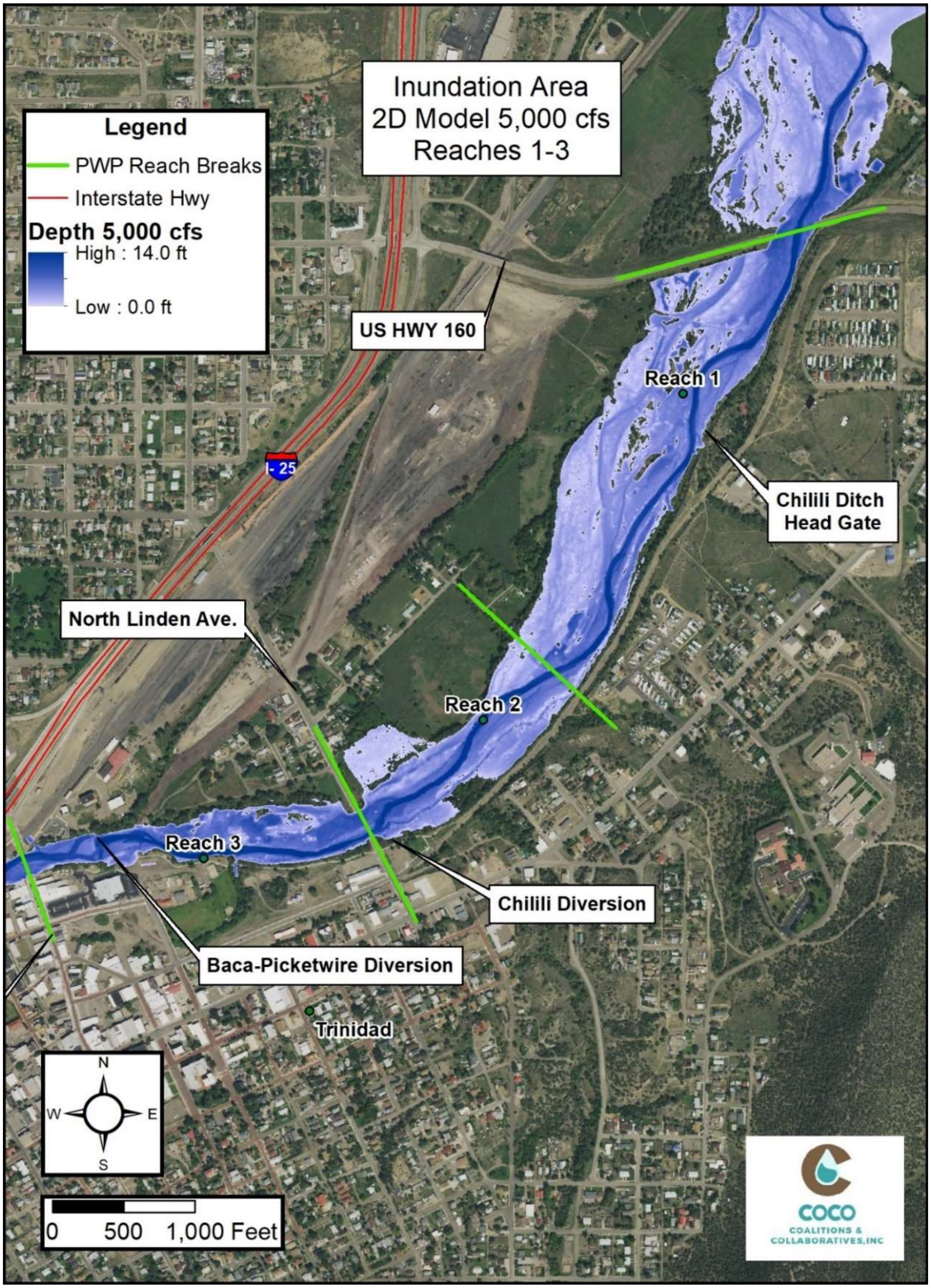

Figure 4-31. Map showing the depth of inundation in reaches 1-3 for the 2D model run at 5,000 cfs

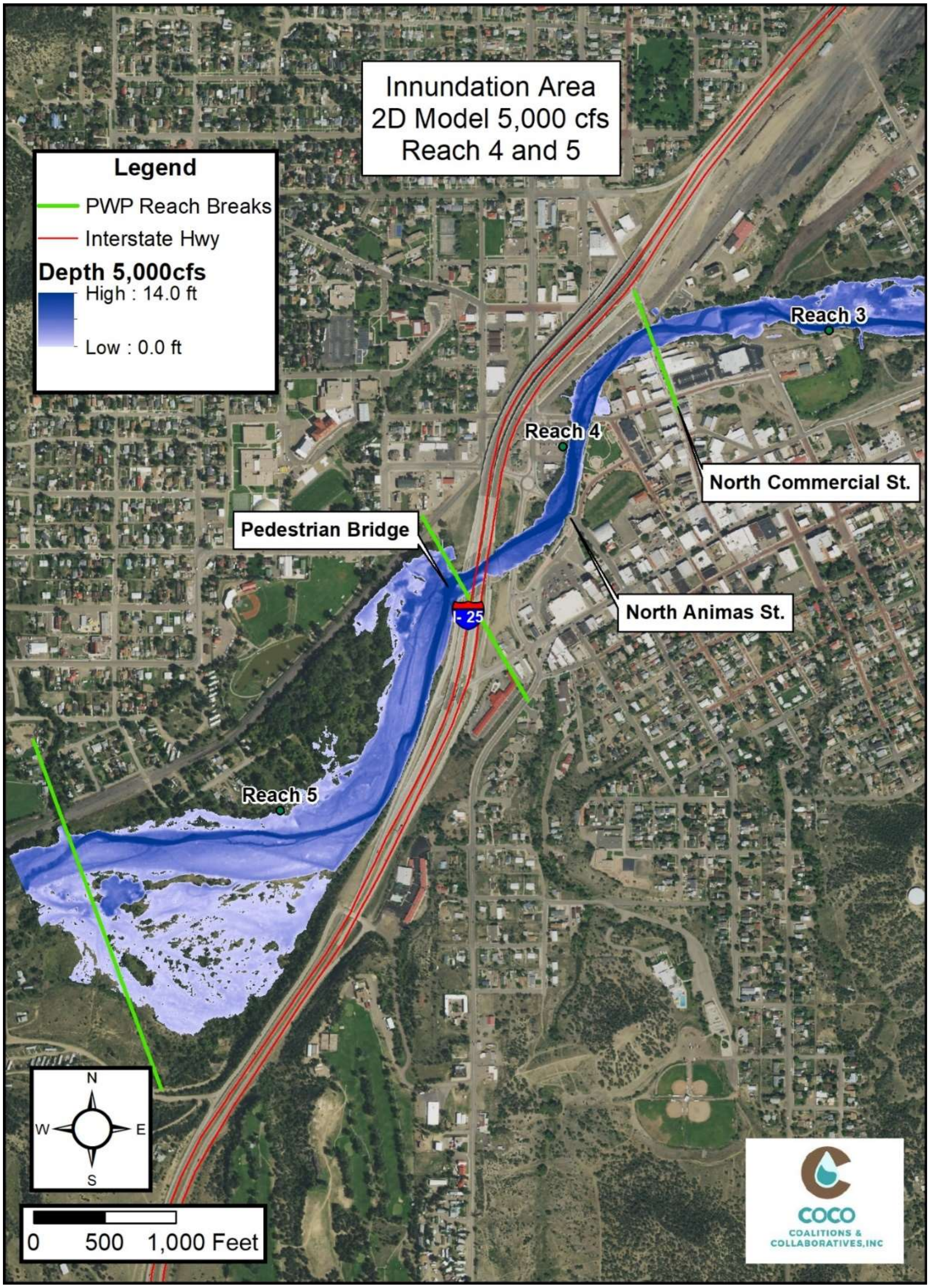

Figure 4-32. Map showing the depth of inundation in reaches 4-5 for the 2D model run at 5,000 cfs

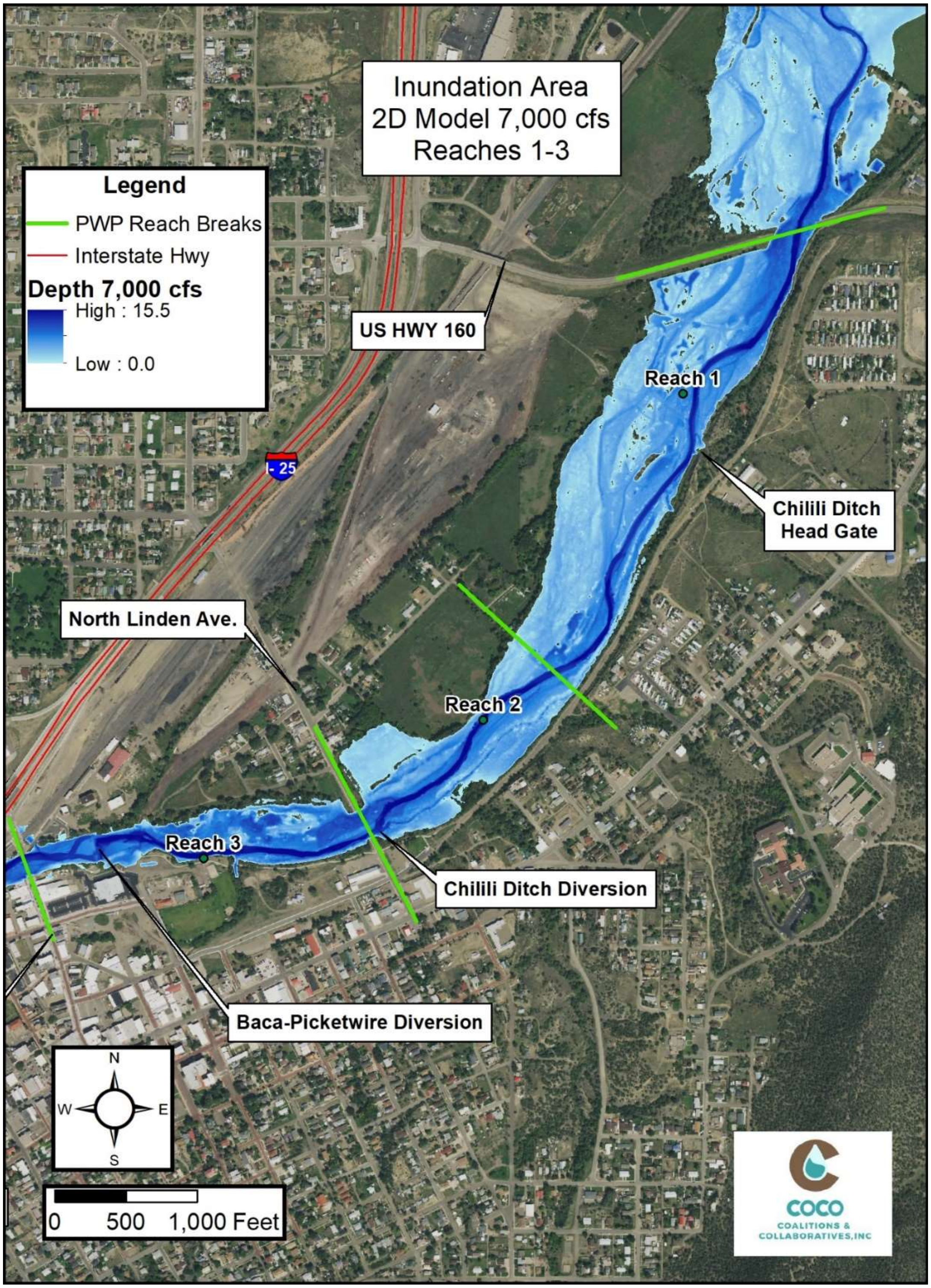

Figure 4-33. Map showing the depth of inundation in reaches 1-3 for the 2D model run at 7,000 cfs

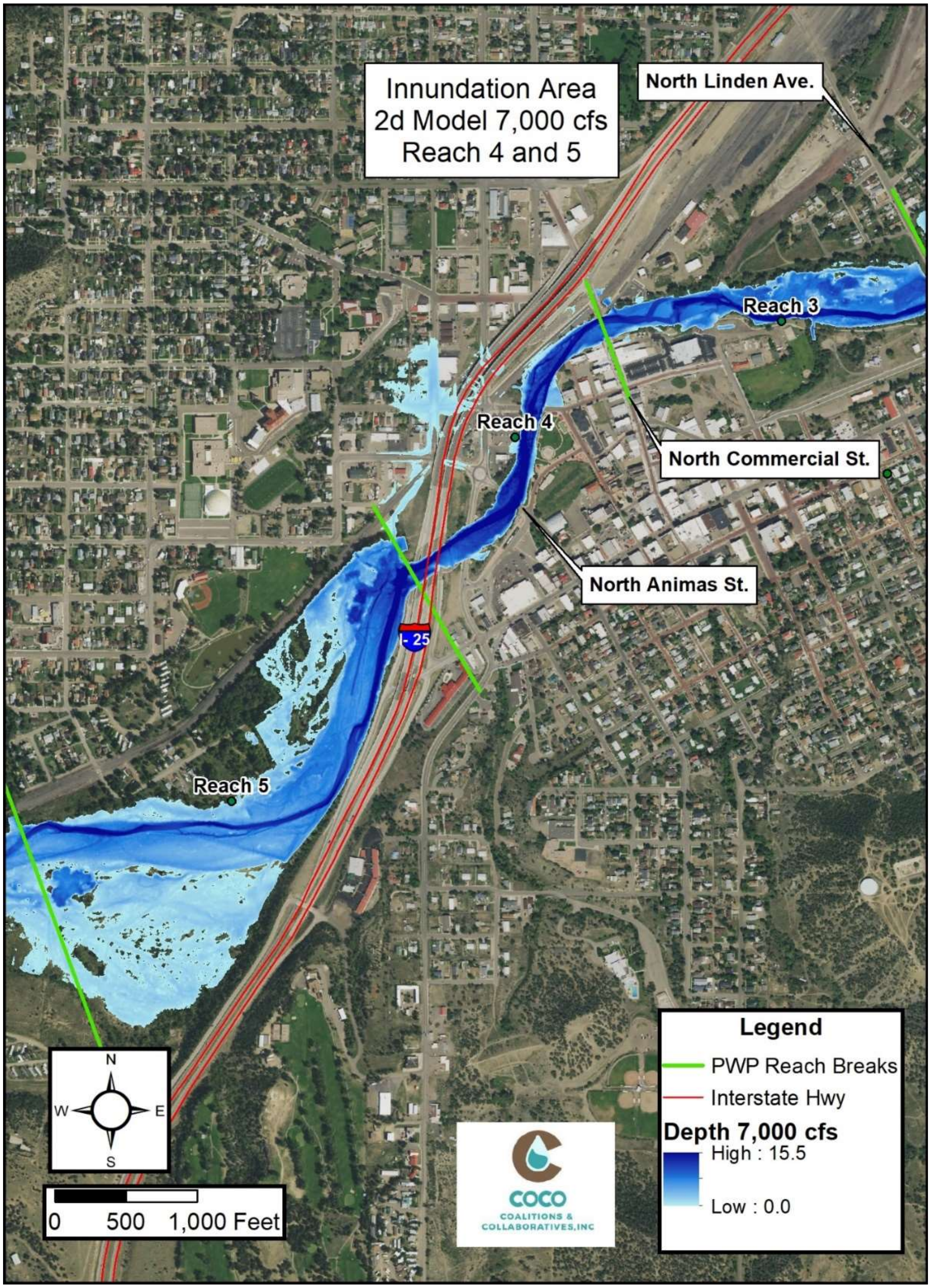

Figure 4-34. Map showing the depth of inundation in reaches 1-3 for the 2D model run at 7,000 cfs

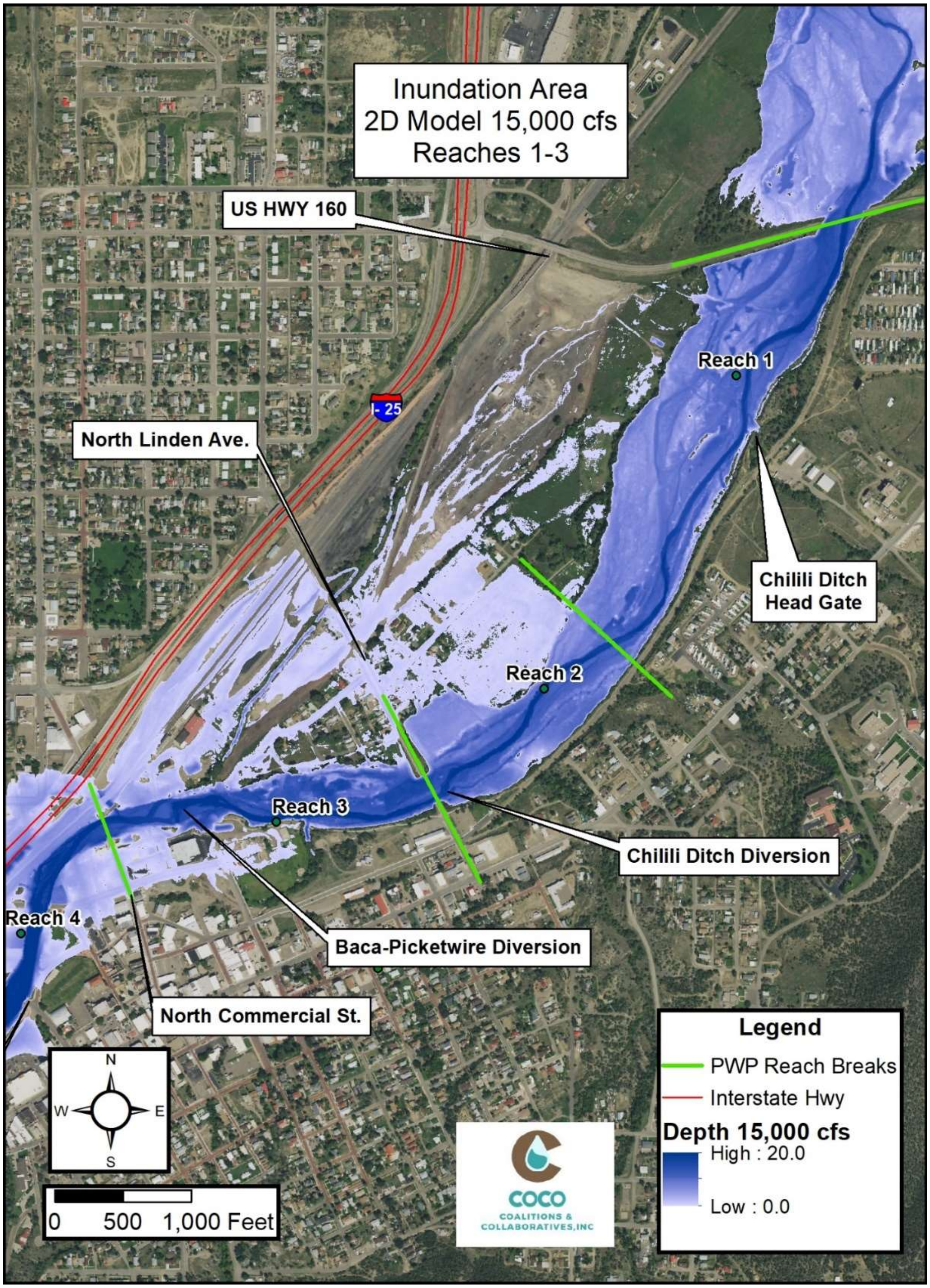

Figure 4-35. Map showing the depth of inundation in reaches 1-3 for the 2D model run at 15,000 cfs

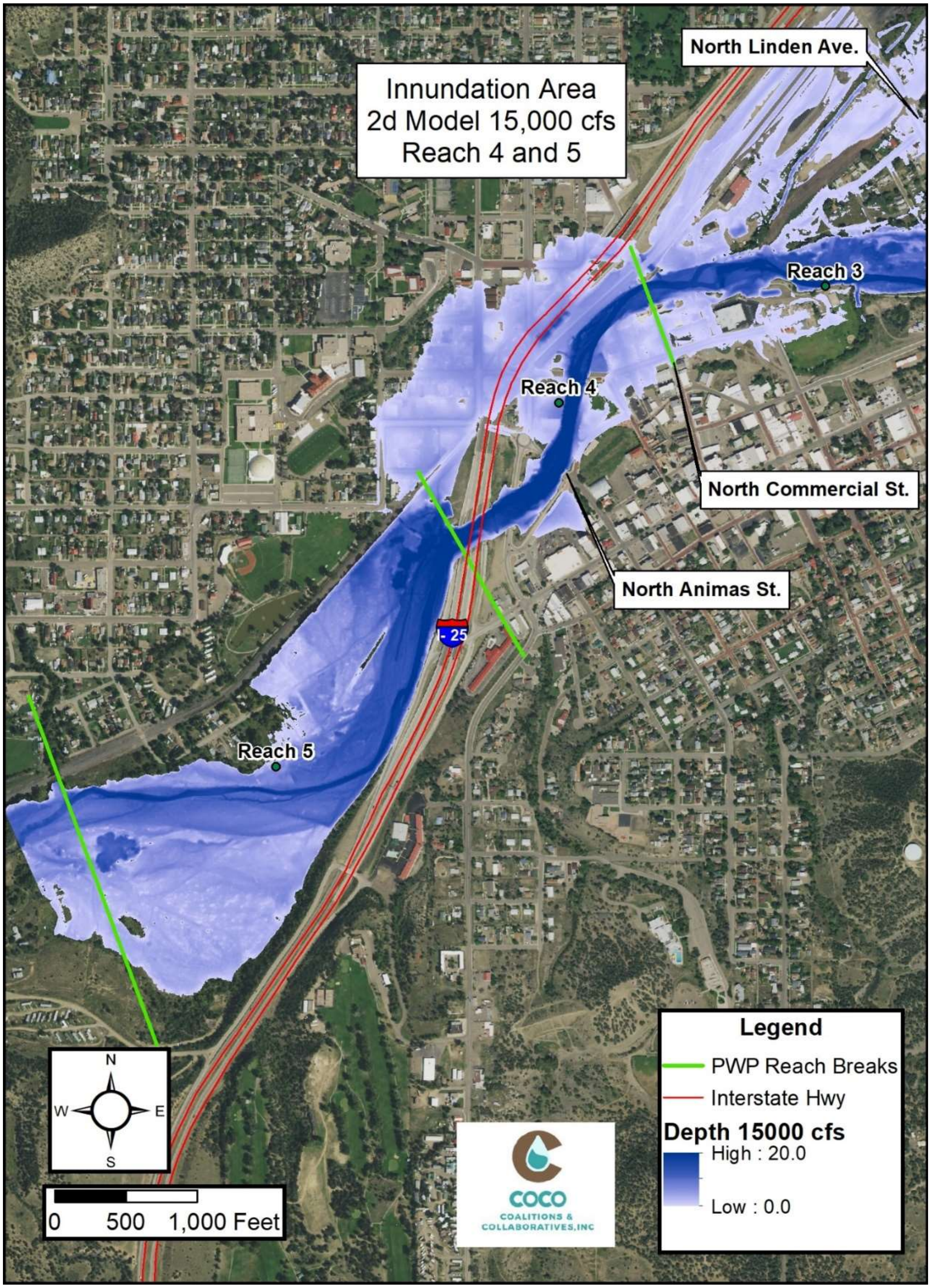

Figure 4-36. Map showing the depth of inundation in reaches 1-3 for the 2D model run at 15,000 cfs

5. Sediment Stability Analysis

5.1. Incipient Motion

Reach average hydraulics using the 1D HEC-RAS model were calculated and used for an incipient motion analysis. An incipient motion analysis evaluated the stability of the existing sediment in the reach. This compares the critical shear stress (moment when motion of bed particles is about to occur) to the calculated shear stress based on the following equations.

$$\tau_C = \tau_{*C} (\gamma S - \gamma) D_{50}$$

where

τ_c = critical shear stress for particle motion,
τ^*_c = dimensionless critical shear stress (often referred to as the Shields parameter),
γ_s = unit weight of sediment (~165 lb/ft^3),
γ = unit weight of water (62.4 lb/ft^3), and
D_{50} = median particle size of the bed material.

In performing the incipient-motion analysis, the bed shear stress due to grain resistance (τ') (Einstein, 1950), should be used rather than the total shear stress, because it is a better descriptor of the near bed hydraulic conditions that are responsible for sediment movement. The grain shear stress is computed from the following relation:

$$\tau' = \gamma R' S$$

where

R' = the portion of the total hydraulic stress associated with grain resistance and
S = the energy slope at the cross section.

Shear stress available to mobilize bed grains was calculated following Pitlick, et al. (2009), and replaces τ_0 in the above equation with τ', defined as follows:

$$\tau' = \rho g (0.013)^{1.5} (SD)^{0.25} U^{1.5}$$

where:

τ'= Bed grain shear stress (N/m2)
S = Friction slope
U = flow velocity (m/s)
D = grain size (mm)

Particle mobility was determined for each bed sediment size class, which varies by reach, by comparing τ^* against a critical dimensionless shear stress value (τ_c^*). When τ^* > τ_c^*, the particle is considered mobile at the representative critical dimensionless shear and associated discharge value. Values of τ_c^*=0.047 and 0.03 were evaluated for bed mobility. A range of flows were evaluated up to 5,000 cfs.

5.2. Sediment Data

Sediment data was provided by Fin-Up consultants. Pebble counts for each reach except reach 2 are shown in figure **5-1**.

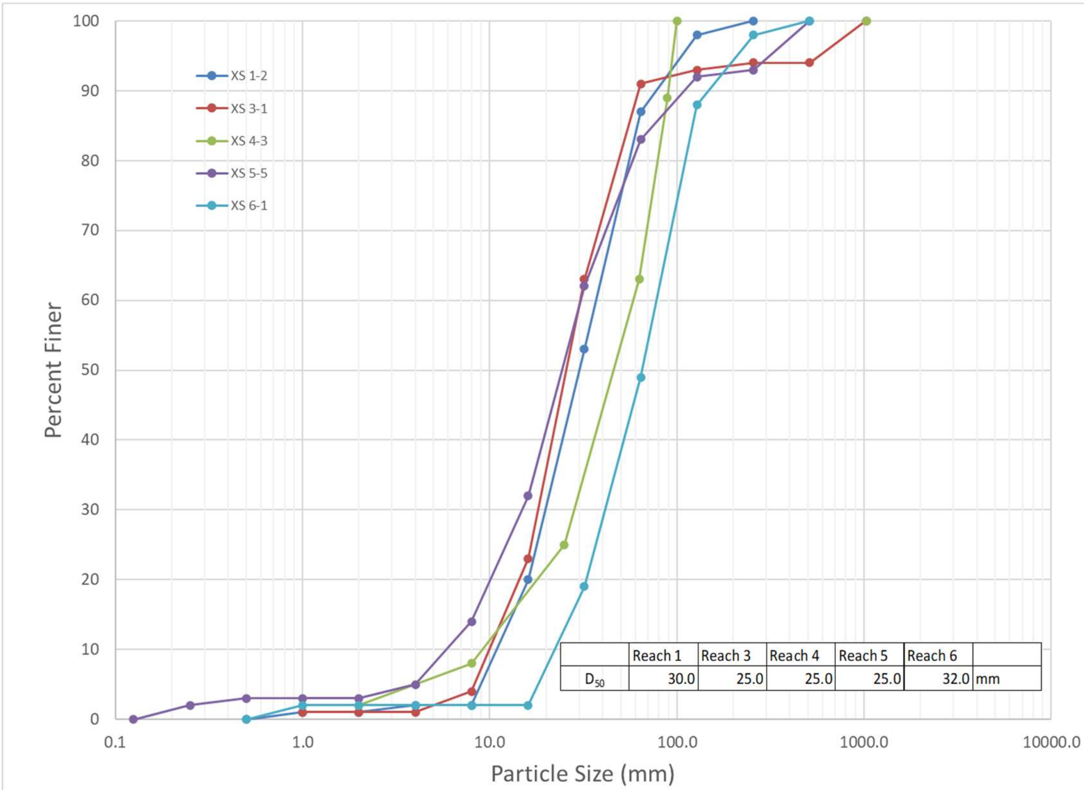

Figure 5-1. Sediment gradations collected by Fin-up.

5.3. Results of Incipient Motion Analysis

The critical shear stress was calculated and compared to the shear stress available for transport based on the shear stress partition. **Figure 5-2** shows the results of the incipient motion using a Shields parameter of 0.03 which is a more conservative estimate. The number in the chart corresponds to the flow shown in **table 5-1**. The chart is a ratio of the two equations. When the ratio exceeds 1 motion generally begins and when the ratio exceeds 1.5 it is assumed that the bed material is fully mobile. This analysis only applies to the bankfull or low flow channel.

Table 5-1. Stability calculation results. The green highlight indicates the critical shear stress is exceeded.

	Shear Stress Ratio															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	50 cfs	100 cfs	150 cfs	200 cfs	250 cfs	300 cfs	350 cfs	400 cfs	450 cfs	500 cfs	700 cfs	1000 cfs	2000 cfs	3000 cfs	4000 cfs	5000 cfs
Reach 6b	0.33	0.45	0.54	0.63	0.69	0.73	0.78	0.82	0.86	0.90	1.04	1.16	1.35	1.64	1.87	1.98
Reach 6a	0.32	0.44	0.54	0.61	0.67	0.73	0.78	0.83	0.87	0.91	1.05	1.20	1.57	1.58	1.60	1.71
Reach 5	0.35	0.47	0.56	0.64	0.71	0.78	0.84	0.90	0.95	1.00	1.18	1.37	1.73	1.84	2.02	2.14
Reach 4	0.25	0.32	0.40	0.43	0.47	0.50	0.54	0.58	0.61	0.64	0.75	0.89	1.24	1.48	1.67	1.83
Reach 3	0.27	0.37	0.44	0.48	0.51	0.55	0.59	0.62	0.66	0.70	0.83	1.00	1.36	1.54	1.78	1.98
Reach 2	0.28	0.38	0.46	0.52	0.57	0.61	0.66	0.70	0.75	0.79	0.93	1.10	1.52	1.71	1.87	1.97
Reach 1	0.24	0.33	0.39	0.45	0.50	0.55	0.59	0.63	0.66	0.69	0.78	0.91	1.23	1.43	1.51	1.66

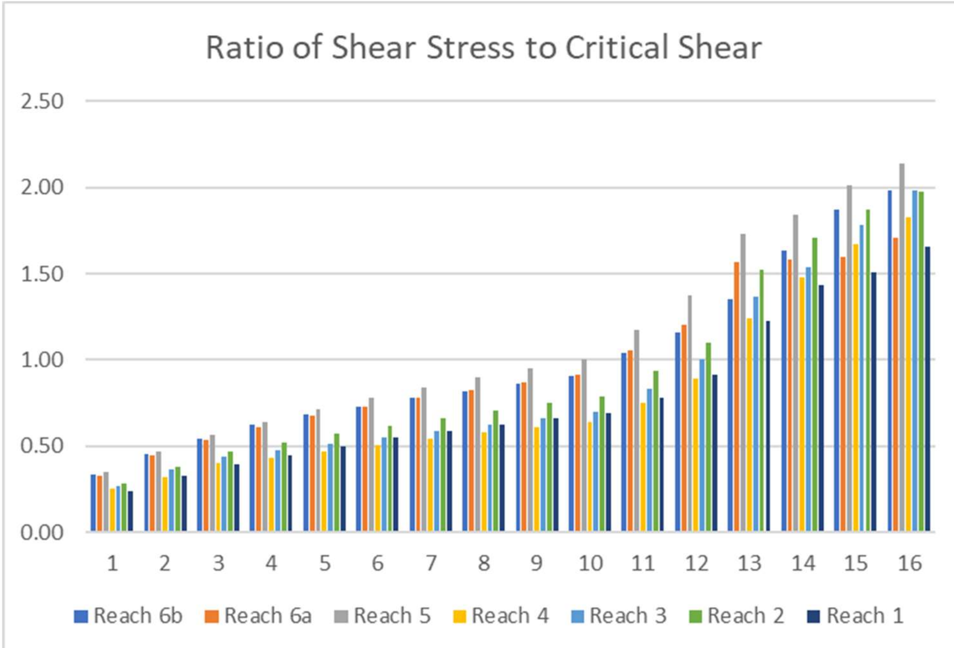

Figure 5-2. Incipient motion analysis ratio of critical shear to calculated shear for a range of flows. The number in the graph corresponds to the number in table 1.

The results of the incipient motion analysis support the assumption that, generally speaking, the bed of the bankfull channel is armored. The dam upstream has cut off sediment supply except for episodic inputs from the tributaries downstream. Sediment contributions are likely flushed through the system. Since the flows rarely exceed 200 cfs based on the flow duration analysis the bed is likely stable the majority of an average annual year. Episodic events from rainfall in tributary watersheds are the most likely contribution of sediment to the river. The finer grain material, smaller than small gravel, is likely flushed through the system.

6. Floodplain Mapping for Jetty Jack Locations

The Purgatoire Watershed Partnership and the stakeholders have an interest to remove jetty jacks within the study area. The Army Corps of Engineers requested the 5,000 cfs floodplain be mapped in relation to the jetty jacks. Acknowledging that the hydraulic model results has limitations due to the lack of high flow calibration data, the 5,000 cfs floodplain was mapped in relation to the approximate jetty jack locations, as that was of interest to USACE in evaluating the next steps towards jetty jack removal/mitigation. It is recommended that the jetty jack locations be surveyed for future planning and design purposes. Figures 5-3 and 5-4 show the approximate jetty jack locations in relation to the computed area of inundation

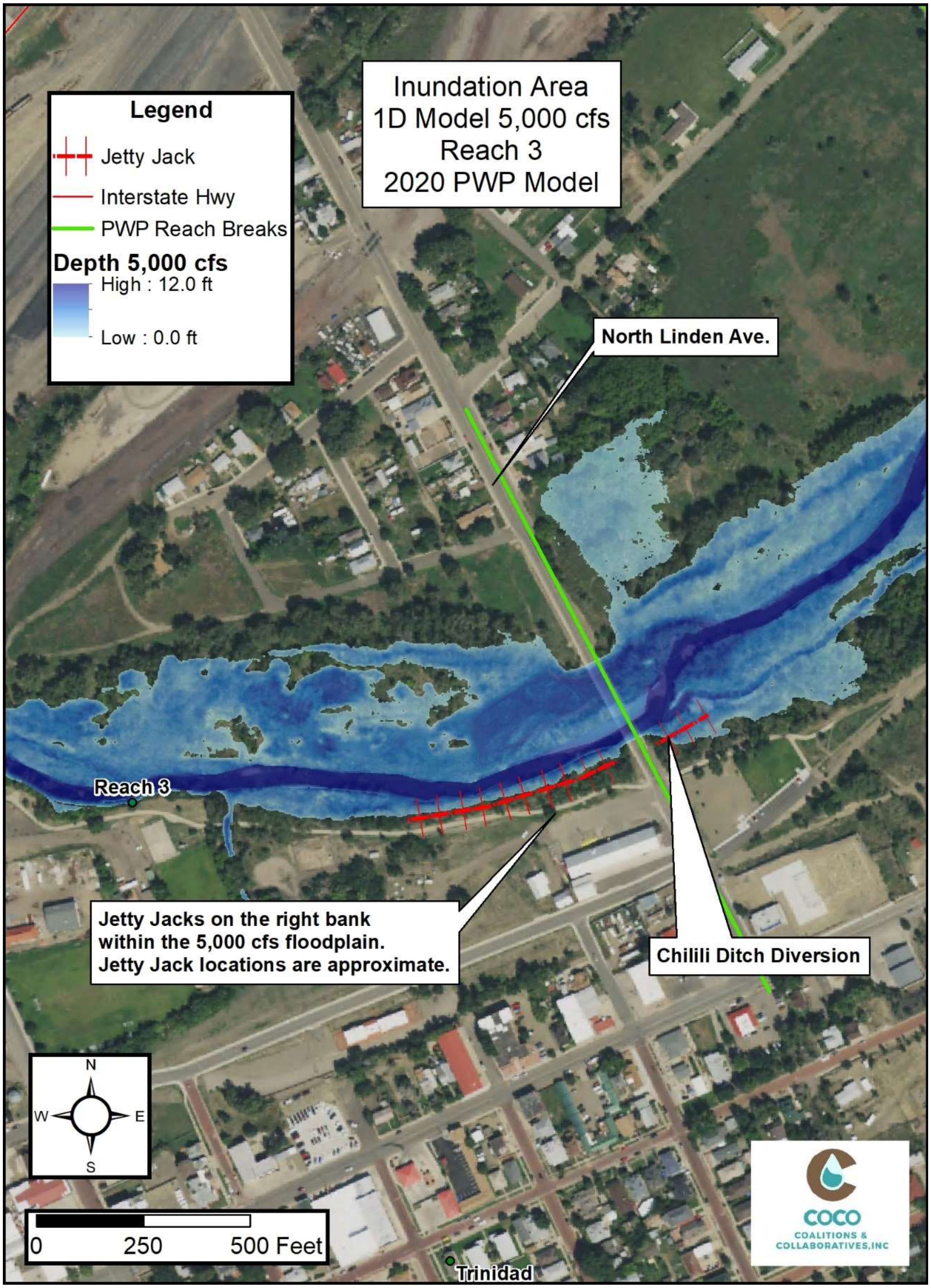

Figure 5-3. Approximate Jetty Jack locations in relation to the 1D 5,000 cfs inundation area.

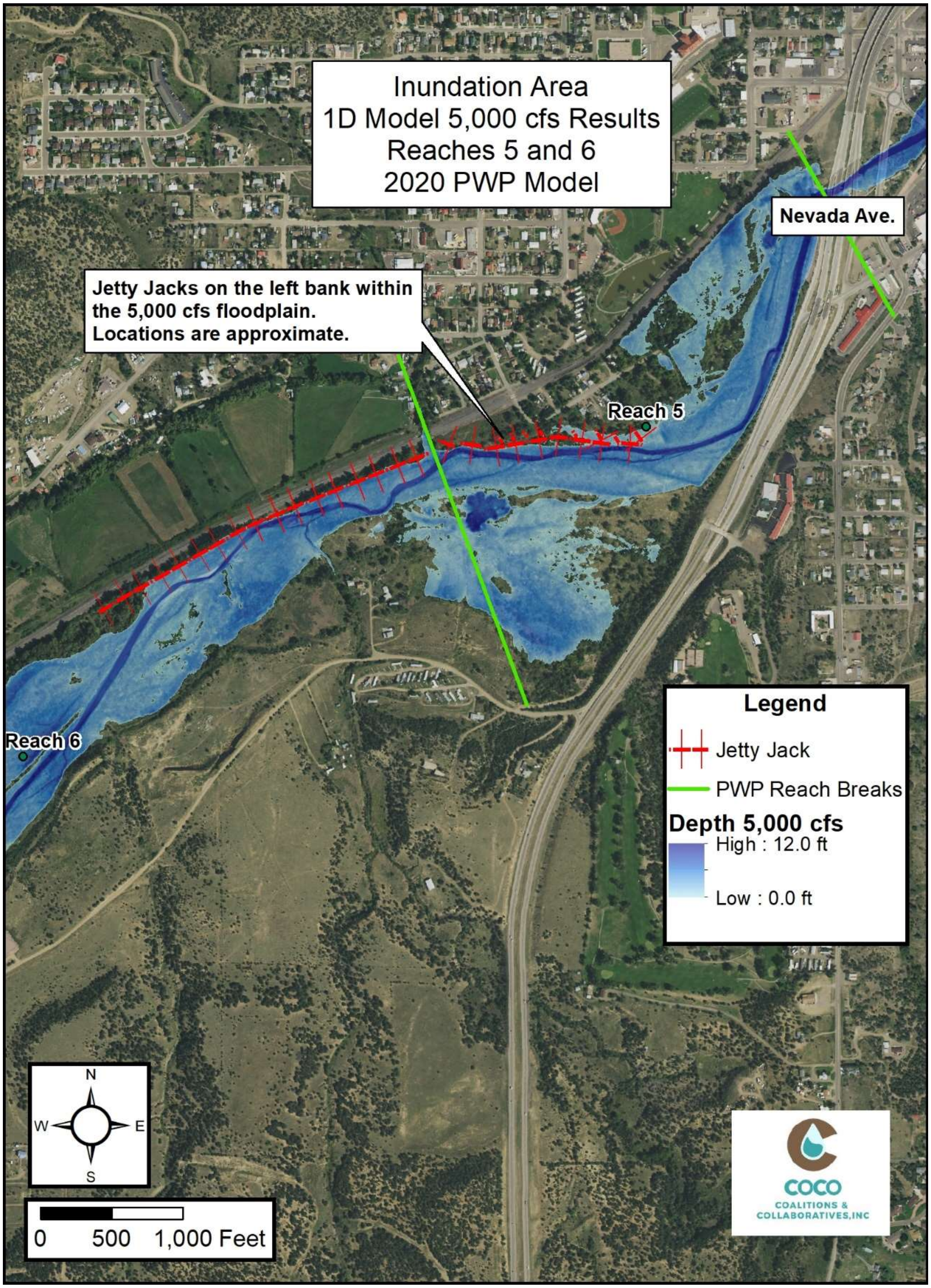

Figure 5-4. Approximate Jetty Jack locations in relation to the 1D 5,000 cfs inundation area in reach 5

7. Recommendations to Protect Infrastructure for Stormwater Release

Recommendations for future river engineering projects are the primary goal for this river assessment. As discussed in section 3 data collection is recommended to improve the accuracy of the hydraulic model if it is to be used for determining a non-damaging stormwater release. Recommendations for channel improvements are discussed here. The recommendations need to be considered along the river corridor as a whole. Thoughtful consideration is recommended if only one section of the river is examined for mitigation design for at a time.

The reach by reach assessment below is presented in reach order from downstream to upstream. This does not imply priorities for improvements. The priority improvement is the area under and immediately upstream of Interstate 25 (**Figure 7-5**). As noted in the hydraulics results section, the pedestrian bridge is predicted to be overtopped at 5,000 cfs. This overtopping increases pressure under the pedestrian bridge and, therefore, increases the scour potential. The floodplain area to the north of the bankfull channel is a critical location to protect the reach downstream through town. The combination of the pedestrian bridge and the overbank grading to accommodate the sidewalk chokes the flow under Interstate 25. The backwater could flow to the north of the Interstate-25 bridge.

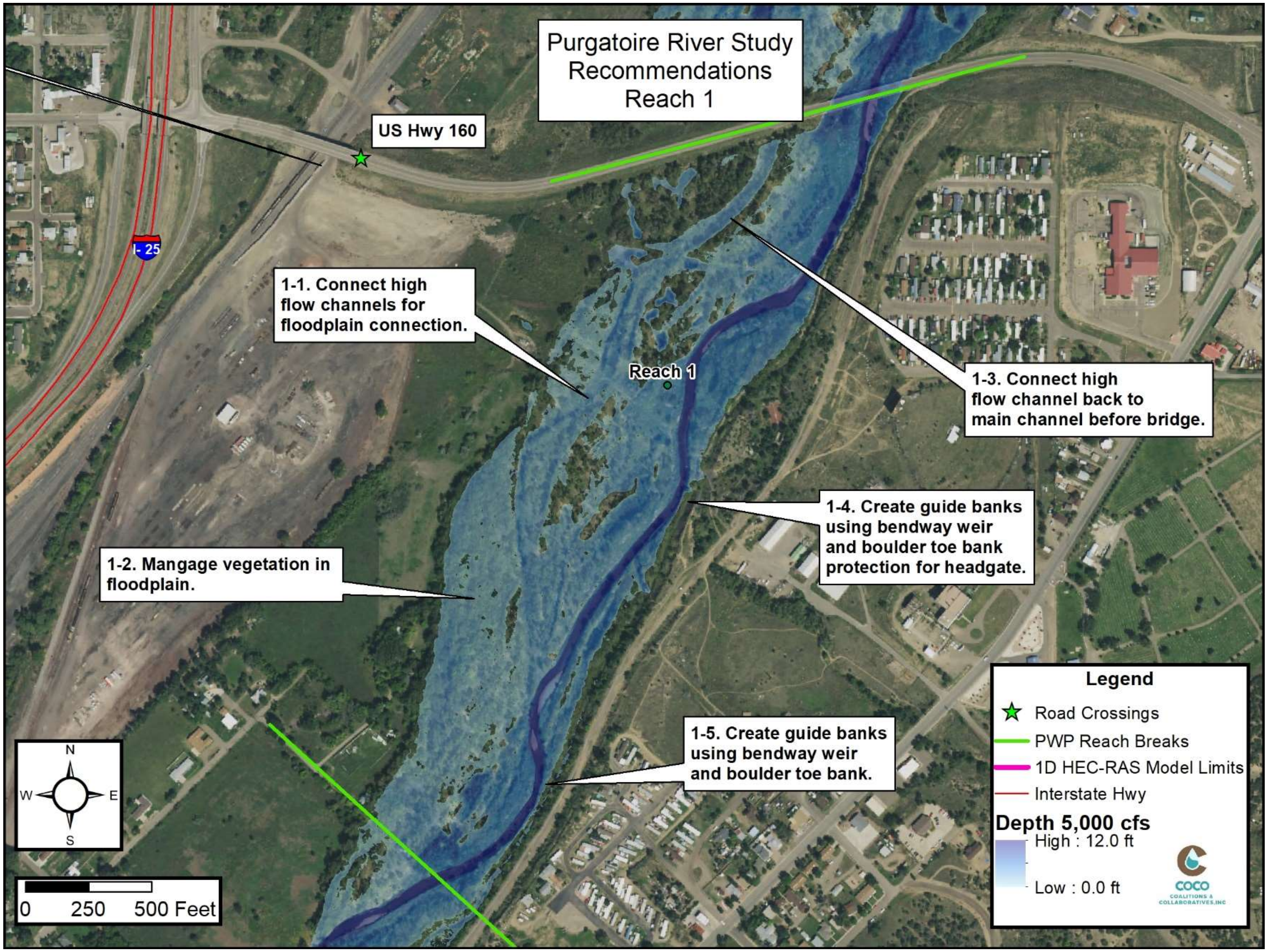

Figure 7-1. Recommendations for river improvements in reach 1.

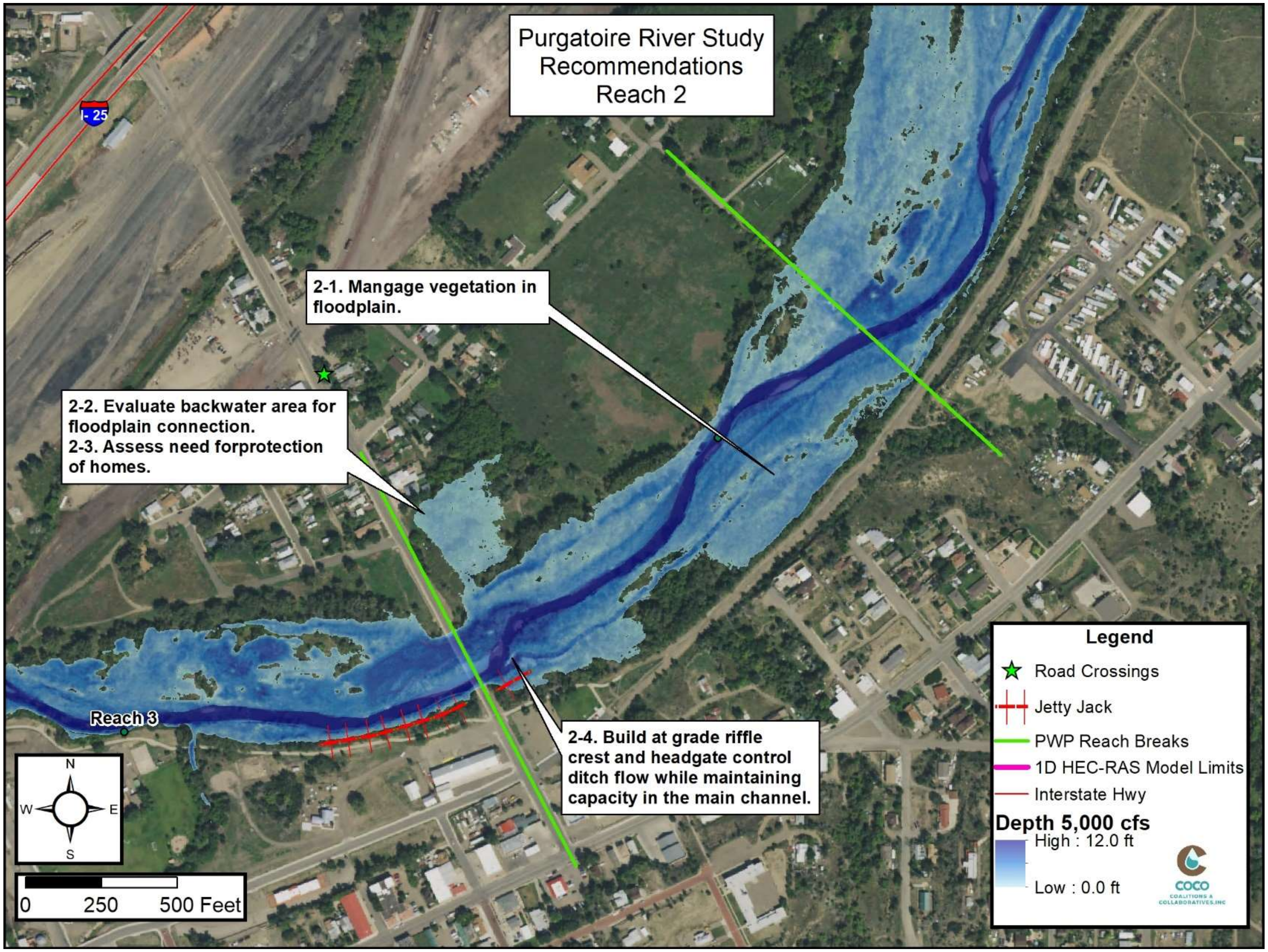

Figure 7-2. Recommendations for river improvements in reach 2.

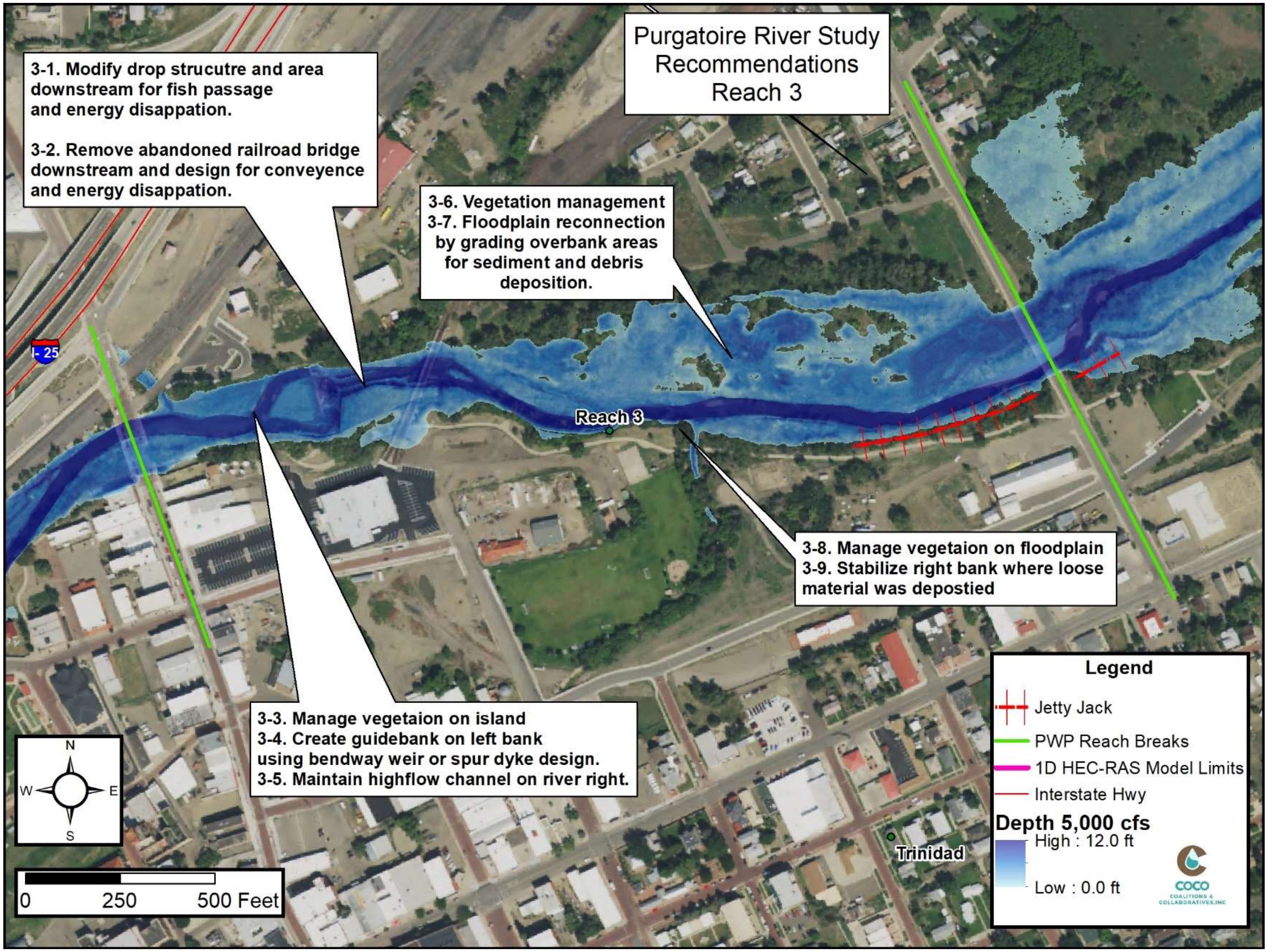

Figure 7-3. Recommendations for river improvements in reach 3.

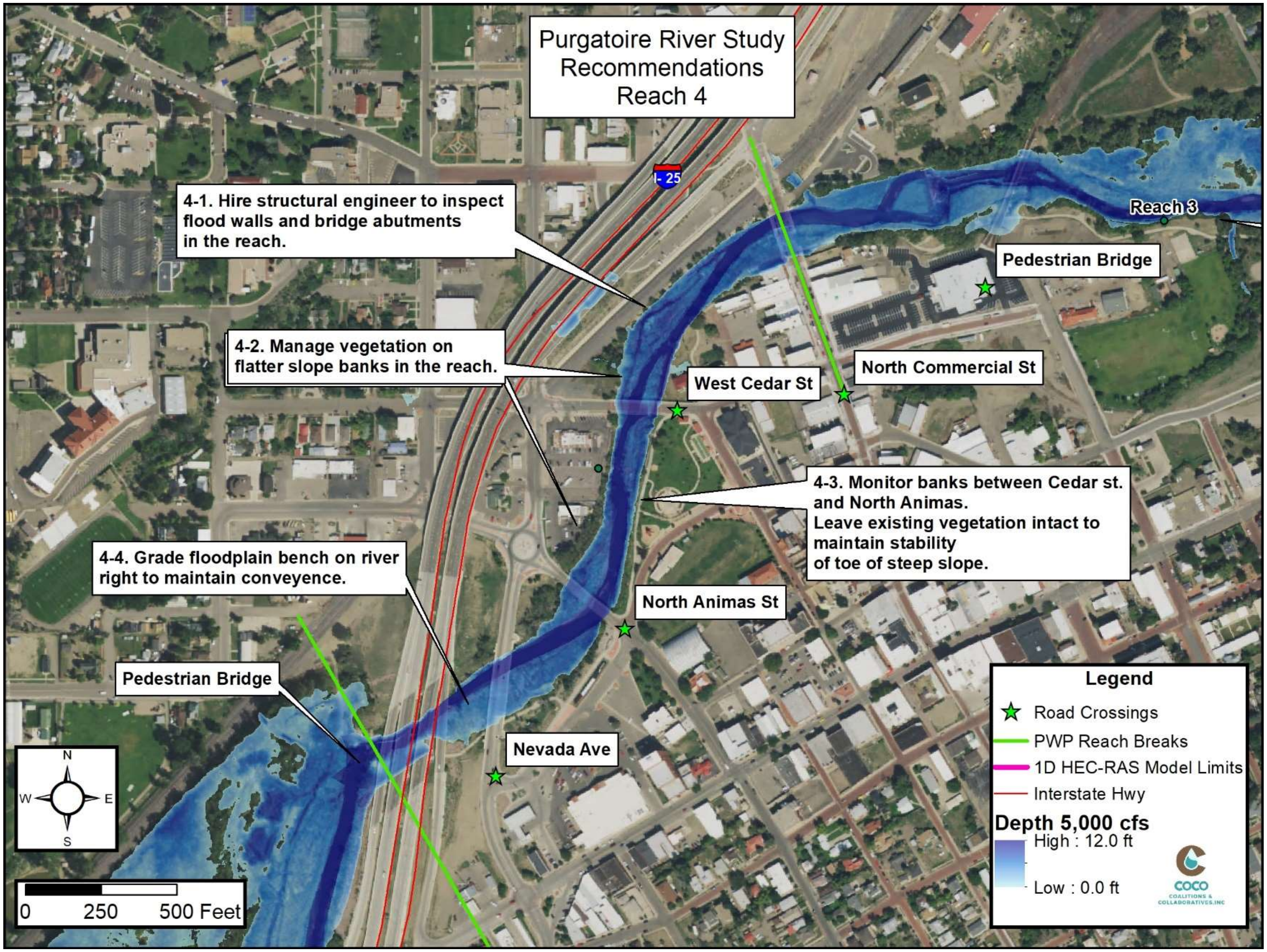

Figure 7-4. Recommendations for river improvements in reach 4.

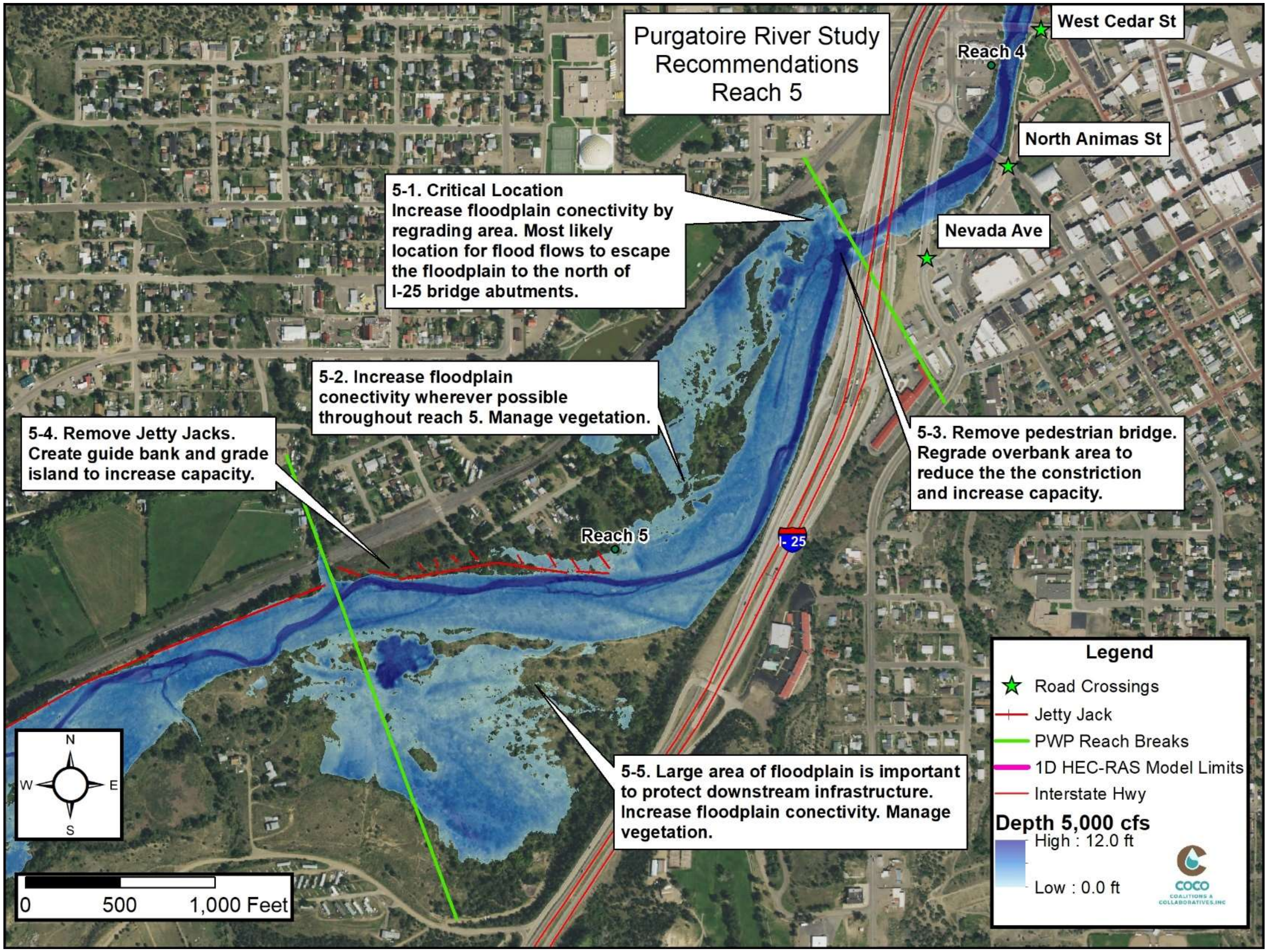

Figure 7-5. Recommendations for river improvements in reach 5.

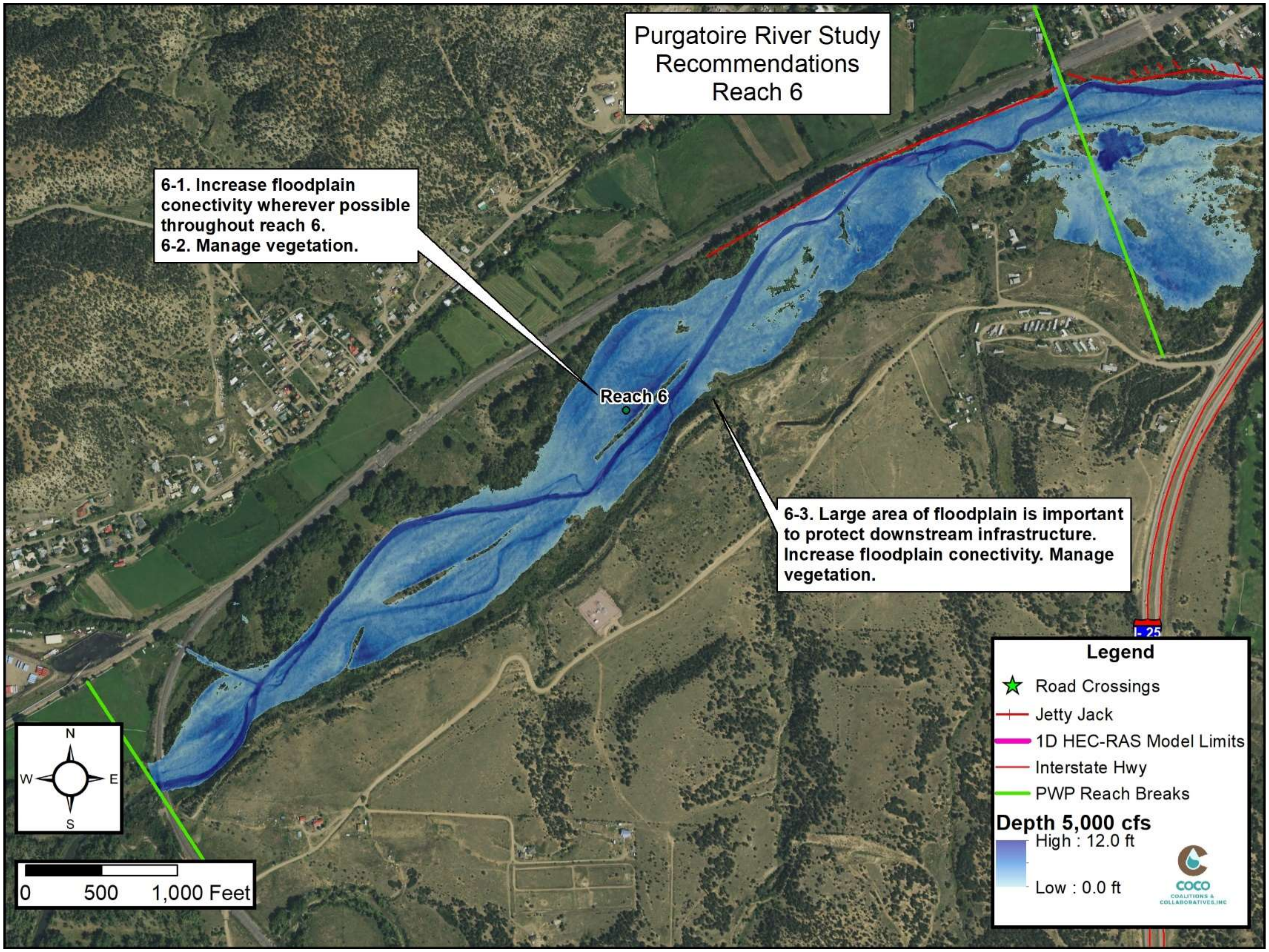

Figure 7-6. Recommendations for river improvements in reach 6.

8. References

Colorado Department of Water Resources gage data for the Purgatoire River at Trinidad downloaded from the State of Colorado Department of Water Resources website (http://www.dwr.state.co.us/)

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