Boxelder B-3: Dam Breach Analysis

Larimer County, Colorado
January 2011

Boxelder B-3 embankment

Welllington

USDA Natural Resources Conservation Service
Colorado State Office

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U.S. DEPARTMENT OF AGRICULTURE
NATURAL RESOURCES CONSERVATION SERVICE
COLORADO STATE OFFICE

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BOXELDER B-3: DAM BREACH ANALYSIS

Location: Larimer County, Colorado near Wellington on Coal Creek.

Summary: Predictions have been made of the likely extent and timing of flooding resulting from a catastrophic breach of the Boxelder B-3 flood retention structure. This report details the dam breach analysis performed on the reservoir for the purpose of evaluating the hazard classification and for use in an emergency action plan.

In the unlikely case of such a breach, farm and ranch land will be flooded, several highways and I-25 will be inundated, and bridges may be damaged. The extent of inundation with expected depth*velocity products greater than 7 indicate that many homes and businesses will be threatened with damage or destruction. Due to this loss of life potential, it is recommended that the hazard classification of this structure be increased from its current significant level to a high hazard classification.

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INTRODUCTION

This report details the methods and results of a dam breach analysis performed on the Boxelder B-3 Dam of Larimer County, Colorado. This analysis was performed primarily to evaluate the hazard classification of the structure, but is essential for other purposes such as the development of an emergency action plan. The analysis consists of breach hydrograph development and hydrograph routing through the stream valleys, agricultural lands, and communities below the structure. The primary communities impacted by a breach of this structure are Wellington and a small portion of the eastern suburbs of Fort Collins.

The Boxelder B-3 dam (NID ID: CO00512) is an earthen-embankment, typically-dry, flood retention structure. The structure is located on Coal Creek at approximately 5450 feet in elevation. This structure provides substantial flood-reduction benefits to the town of Wellington, dispersed homes and ranches downstream.

Average precipitation within the reservoir's 61 square mile watershed ranges from 15 to 17 inches, according to PRISM. The B-4 embankment has a maximum height of about 44.0 feet, with a crest elevation of 5489.0 feet, original ground elevation at the downstream toe of about 5445 feet and embankment length of 2700 feet. The maximum storage, with the water surface elevation at the crest of the embankment, is 6410 acre-feet. The auxilary spillways are two parallel, 200 foot wide, earthen structures on the left abutment. At the auxiliary spillway crest elevation of 5481.0 feet the associated reservoir storage is 3840 ac-ft. These volumes do not account for accumulated sediment since dam construction.

This dam breach analysis uses the available 10-meter DEM combined with supplemental surveying of a BNSF railroad embankment and multiple cross-sections in Wellington. Due most substantially to the use of the 10-meter DEM, the results of this analysis are approximate – they provide an approximation of the spatial extent of the flood inundation in the case of the catastrophic failure of the embankment. The results are least dependable where the relief is low and the floodwave will extend at shallow depths across a wide valley, such as in the last several miles of the Boxelder just above the Cache la Poudre River, in the eastern suburbs of Fort Collins. Despite these shortcomings, this analysis is appropriate for evaluating the hazard classification of the structure and does provide a reasonable approximation of the likely flood extent and timing in the case of a catastrophic breach, for the development of an emergency action plan.

This report details the methodology used to determine the likely effects of a catastrophic breach. The primary sections include an Introduction, Breach Hydrograph Development, Hydrograph Routing, Likely Inundation Extent and Timing, and Summary and Conclusions. **For results, see the INUNDATION EXTENT AND TIMING and SUMMARY AND CONCULSIONS sections. Inundation mapping is provided in APPENDIX A. Valley cross-sections in the vicinity of Wellington are provided in APPENDIX B.**

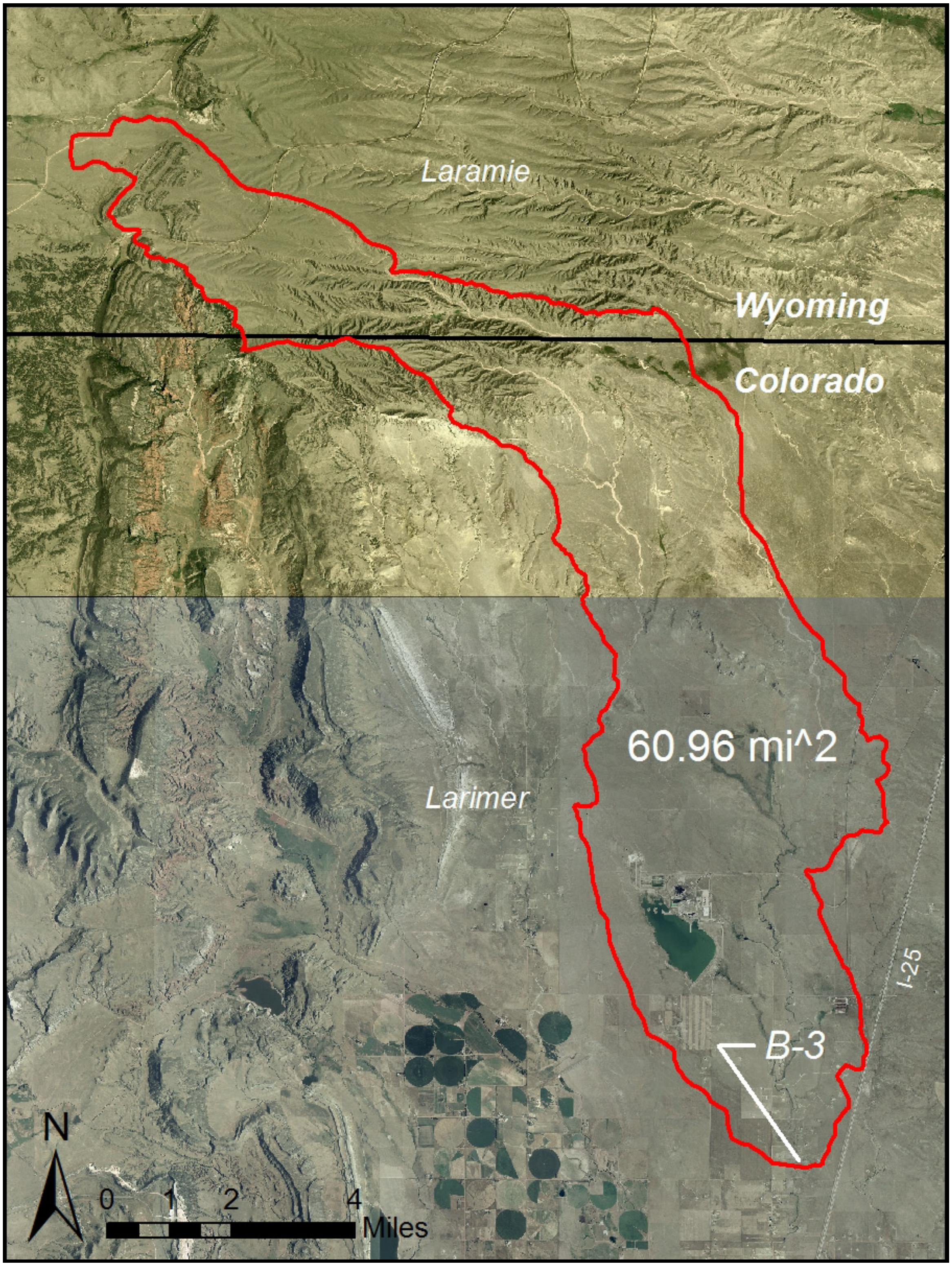

Figure 1: Boxelder B-3 reservoir watershed.

BREACH HYDROGRAPH DEVELOPMENT

As discussed in Froehlich (1995a), the International Commission on Large Dams reports that roughly a third of embankment dam failures are caused by overtopping due to inadequate spillway capacity; another third result from piping failure; and the last third result from embankment sliding, embankment settlement, and inadequate wave protection. An embankment piping failure with initial water surface assumed to be at the crest of the emergency spillway (5481 feet, breach volume = 3840 ac-ft) is modeled in this analysis.

There are various methods available for estimating a dam breach hydrograph and peak flow, including various regression equations for the peak flow (using embankment and reservoir characteristics), breach geometric characteristics, and time to full breach. Also, more physically-based methodologies are available. Wahl (2004) documented four equations for predicting breach width, five failure time equations, and 14 peak flow equations – there are many options available for predicting breach characteristics, using multiple approaches. Each approach has advantages and disadvantages, with no one method considered best.

Due to the various available approaches available for estimating the breach flow, several methods have been used to determine a range of potential breach hydrographs and professional judgment implemented to determine the most appropriate hydrograph for routing downstream.

Alternately, a stochastic process could be used to develop predicted peak flow rates, water surface elevations and inundation extents (Froehlich 2008). The stochastic approach acknowledges the inherit unpredictability of a breach failure and, instead of considering the process to be deterministic with readily predictable geometric or erosive properties, instead combines predictable outcomes with uncertainties to determine a statistically-defined range in outcomes. Froehlich (2008), when providing an example of such a methodology, performed a Monte Carlo simulation with 100,000 trials for three random variables (average breach width, breach formation time, critical overtopping depth). Such an approach has promise in dam failure studies. However, using a stochastic approach is currently considered too time-intensive for this structure, especially considering the limited geometric information available in the floodway and resulting uncertainties for the breach-wave routing.

Photos illustrating the general embankment characteristics of the Boxelder B-3 structure are provided in Figures 2 through 4. As illustrated in Figures 3, the downstream face is not armored by rock and is instead protected by vegetative cover dominated by Crested Wheatgrass (*Agropyron cristatum*). This is a clumpy grass cover. In the case of embankment overtopping during a rainfall event that approaches the probable maximum precipitation, this vegetation may actually be detrimental to the stability of the embankment, due to small-scale flow acceleration and enhanced erosion around the grass clumps.

Figure 2: Upstream face of B-3 embankment.

Figure 3: Downstream face of B-3 embankment.

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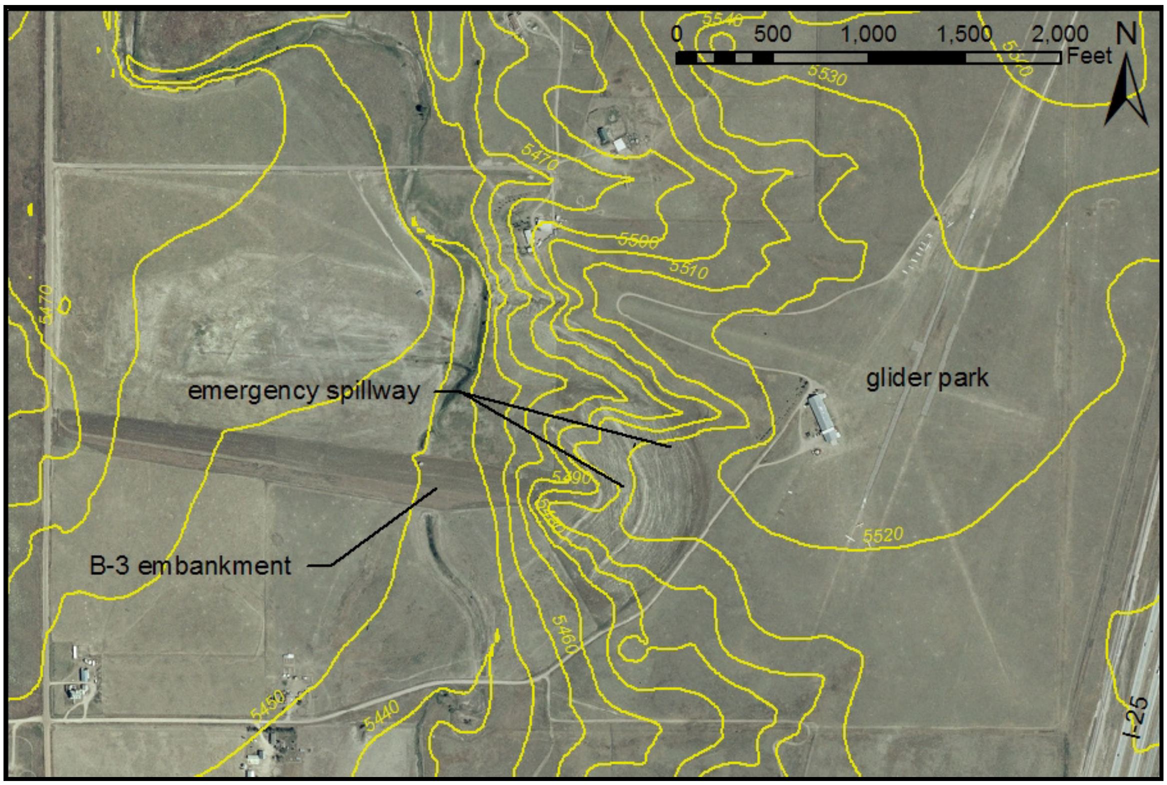


Figure 4: B-3 embankment. Aerial photography taken summer of 2005. Pre-construction 10-foot contours shown.

The methods used to develop possible hydrograph characteristics are peak flow equations developed by NRCS, Froehlich, Kirkpatrick, and the U.S. Bureau of Reclamation; and breach geometry prediction using Froehlich, U.S. Bureau of Reclamation, and Von Thun and Gillette; breach formation time using Froehlich, MacDonald and Langridge-Monoplolis, U.S. Bureau of Reclamation, and Von Thun and Gillette. A summary of the breach hydrograph characteristics predicted by each method is provided in Table 1.

Table 1: Breach hydrograph characteristics for the various methodologies. Initial water surface elevation at crest of emergency spillway (5481.0 feet).

Method	Peak Flow (cfs)	Volume (acre-feet)
NRCS peak flow	49,200	3840
Froehlich (1995) peak flow	39,000	3840
Kirkpatrick (1977) peak flow	19,200	3840
U.S. Bureau of Reclamation (1982) peak flow	56,900	3840
MacDonald and Langridge-Monoplolis (1984) peak flow	202,000	3840
Evan (1986) peak flow	87,700	3840
breach geometry prediction (in HEC-RAS)	54,500	3950
Average:	72,643	
Median:	53,050	

Peak Flow Prediction

As provided in TR-60 (NRCS 2005), peak flow can be estimated using the following empirical equations. The development of these equations is not well documented.

The criteria for peak flow prediction for an embankment height less than 103 ft is

$$Q_{max} = 1100 B_r^{1.35} \tag{1}$$

where

$$B_r = \frac{V_s H_w}{A} \tag{2}$$

But the peak flow is not to be less than

$$Q_{max} = 3.2 H_w^{2.5} \tag{3}$$

and need not exceed

$$Q_{max} = 65 H_w^{1.85} \tag{4}$$

where V_s is the reservoir storage at the time of failure (ac-ft), H_w is depth of water at dam at time of failure (ft), and A is the cross-sectional area of the embankment at the location of the breach (ft^2), normal to the longitudinal axis. With H_w = 36 feet, V_s = 3840 acre-feet and A = 5600 ft^2, the peak discharge is 83,400 cfs, should not be less than 24,900 cfs but not in excess of 49,200 cfs.

As documented in Froehlich (1995a), peak flow can be predicted from the following equation. (This well-documented peer reviewed equation, which was developed from 22 embankment dam failures, has a R^2 of 0.934.)

$$Q_p = 0.607 V_w^{0.295} H_b^{1.24} \tag{5}$$

where V_w is the reservoir volume at time of failure (1,567,000 m^3) and H_b is the height of water in the reservoir at the time of failure above the final bottom elevation of the breach (11.0 m). Using this equation, a peak discharge of 39,000 cfs (1110 cms) is estimated.

As presented in Wahl (2004), the Kirkpatrick (1977) equation is

$$Q_p = 1.268 (H_b + 0.3)^{2.5} \tag{6}$$

Using this equation, a peak discharge of 19,200 cfs (540 cms) is estimated.

The U.S. Bureau of Reclamation equation (1982) is

$$Q_p = 19.1(H_b)^{1.85} \tag{7}$$

Using this equation, a peak discharge of 56,900 cfs (1610 cms) is estimated.

The MacDonald and Langridge-Monoplolis equation (1984) is

$$Q_p = 3.85(V_w H_b)^{0.411} \tag{8}$$

Using this equation, a peak discharge of 202,000 cfs (5720 cms) is estimated.

The Evan equation (1986) is

$$Q_p = 0.72(V_w)^{0.53} \tag{9}$$

Using this equation, a peak discharge of 87,700 cfs (2490 cms) is estimated.

There was a substantial range in the breach peak flow time estimates, from 19,200 to 202,000 cfs. The average and median values were 72,600 and 53,100 cfs, respectively.

Breach Formation Time

A breach formation time estimate was developed using a number of methods, as documented in Wahl (2004). A summary of results is provided in Table 2. These equations are not independent since many of the same failures are likely used in each prediction equation.

Table 2: Breach formation time using various methodologies. Initial water surface elevation at crest of emergency spillway (5481.0 feet).

Method	Formation Time (hours)
Froehlich (1995b)	1.01
MacDonald and Langridge-Monoplolis (1984)	0.69
U.S. Bureau of Reclamation (1988)	0.36
Von Thun and Gillette (A) highly erodible (1990)	0.17
Von Thun and Gillette (A) erosion resistant (1990)	0.47
Von Thun and Gillette (B) highly erodible (1990)	0.44
Von Thun and Gillette (B) erosion resistant (1990)	1.04
Median:	**0.47**
Average:	**0.60**

The equation developed by Froehlich (1995b) is

$$t_f = 0.00254 V_w^{0.53} H_b^{-0.90} \tag{10}$$

where t_f is the breach formation time (hours), V_w is the reservoir volume at time of failure (m^3), and H_b is the height of breach (m). With V_w = 4,736,570 m^3 and H_b = 11.0 m, the breach formation time is estimated to be 1.01 hours.

MacDonald and Langridge-Monoplolis (1984) developed the following equation:

$$t_f = 0.0179 V_{er}^{0.364} \tag{11}$$

where

$$V_{er} = 0.0261 (V_w h_w)^{0.769} \tag{12}$$

is defined for earthfill dams, V_w is the reservoir volume (m^3) and h_w the depth of water (m) at the time of failure. With the B-3 embankment characteristics, V_{er} is 22,400 and the breach formation time is 0.69 hours.

The U.S. Bureau of Reclamation (1988) method predicts the formation time as

$$t_f = 0.011 (B_{avg}) \tag{13}$$

where B_{avg} is the breach width, which is predicted as

$$B_{avg} = 3h_w \tag{14}$$

This method predicts an average breach width of 33.0 meters (108 ft) and formation time of 0.36 hours.

Von Thun and Gillette (1990) developed two pairs of equations for predicting formation time with each pair providing predictions for highly-erodible and erosion-resistant conditions.

The first pair (A) predicts the formation time using only the depth of water:

$$t_f = 0.015 h_w \tag{15}$$

$$t_f = 0.020 h_w + 0.25 \tag{16}$$

where equation (15) is for highly-erodible materials and equation (16) is for erosion-resistant embankment materials. This method predicts the formation time as 0.17 and 0.47 hours.

The second pair of equations predicts the formation time using average breach width:

$$t_f = \frac{B_{avg}}{4h_w} \tag{17}$$

$$t_f = \frac{B_{avg}}{(4h_w + 61)} \tag{18}$$

where equation (17) is for highly-erodible materials and equation (18) is for erosion-resistant embankment materials. The average breach width (B_{avg}) is:

$$B_{avg} = 2.5 h_w + C_b \tag{19}$$

where C_b is a function of reservoir storage and equivalent to 18.3 in this circumstance (Wahl 1998).

This method predicts an average breach width of 45.8 meters (150 ft) and the formation time as 0.44 and 1.04 hours.

There was a substantial range in the breach formation time estimates, from 0.17 hours to 1.04 hours. The median value was 0.40 hours, while the average was 0.60 hours. The average breach formation time of 0.60 hours (36 minutes) is used in this analysis.

Breach Geometry Prediction

Breach geometry consists of an average breach width and side slope estimates. It is assumed that the side slopes are the average of what Froehlich (1996b) found to be the case in the piping failures he looked at: 0.9. The average breach width was computed using a number of prediction equations and the BREACH model. A summary is provided in Table 3.

Table 3: Average breach width using various methodologies. Initial water surface elevation at crest of emergency spillway (5401 feet).

Method	Average Breach Width (feet)
Froehlich (1995b)	130
U.S. Bureau of Reclamation (1988)	108
Von Thun and Gillette (1990)	150
Median:	**130**

The average breach width predicted using Froehlich (1995b) is:

$$\overline{B} = 15 k_0 V_{wm}^{0.32} h_w^{\ 0.19} \tag{20}$$

where V_{wm} is the reservoir volume at the time of failure (millions of m^3), h_w is the height of the final breach (meters), and k_o is equal to 1.4 for an overtopping failure mode or 1.0 for piping. With a reservoir volume of 4,736,600 m^3 and depth of water of 11.0 m, this method predicts an average breach width of 38.9 m (130 feet).

As developed from Equation (14), the U.S. Bureau of Reclamation (1988) predicts an average breach width of 33.0 meters (108 ft)

Von Thun and Gillette (1990) provides average breach width from equation 19. This method predicts an average breach width of 45.8 meters (150 ft)

There was a moderately-variable range in the average breach width estimates, from 108 to 150 feet. The median value was 130 feet. A breach hydrograph was developed for a scenario with a 130 feet wide average breach width, side slopes of 0.9 and formation time of 36 minutes.

The breach geometry and formation time were inputted into HEC-RAS unsteady and the resulting hydrograph was developed assuming a sine wave progression (Brunner 2010). Given this breach geometry and formation time, the model simulates a breach hydrograph with a peak at 54,500 cfs (at the embankment, station 200,000).

Selected Breach Hydrograph

Many potential breach hydrographs can be computed from the results of the numerous equations and methods summarized above. Reviewing the results of the various analyses, there is a very wide range of peak flow predicted using the various methodologies, from 19,200 cfs (Kirkpatrick) to 202,000 cfs (MacDonald and Langridge-Monoplolis peak flow).

Using professional judgment, it was decided to use the breach geometry prediction HEC-RAS model output, which is similar to the median peak breach flow value, with the median width of 130 feet, side slopes of 0.9:1, and a formation time of 36 minutes (0.60 hours). These parameters produce a peak breach flow of 54,500 cfs.

HYDROGRAPH ROUTING

The Hydrologic Engineering Center – River Analysis System (HEC-RAS) one-dimensional computer program, by the U.S. Army Corps of Engineers, was used to route the floodwave from the dam breach down the Boxelder to the Cache la Poudre River. HEC-RAS version 4.0 was used in this analysis.

Computation Methodology, HEC-RAS

To support the basis of the modeling used in this dam breach analysis and to discourage a "black box" mentality, the basic equations used in these computations are briefly presented.

The physical laws that govern unsteady flow modeling, as presented in the HEC-RAS Hydraulic Reference Manual (Brunner 2010), are conservation of mass (the continuity equation) and conservation of momentum. The general continuity equation (not separately written for both the channel and floodplain) is:

$$\frac{\partial A}{\partial t} + \frac{\partial S}{\partial t} + \frac{\partial Q}{\partial x} - q_1 = 0$$

Where: ∂ = partial differential.
A = cross-sectional area.
t = time.
S = storage from non conveying portions of cross section.
Q = flow.
x = distance along the channel.
q_l = lateral inflow per unit distance.

The momentum equation can be stated as "the net rate of momentum entering the volume (momentum flux) plus the sum of all external forces acting on the volume be equal to the rate of accumulation of momentum" (Brunner 2010). In differential form, it is:

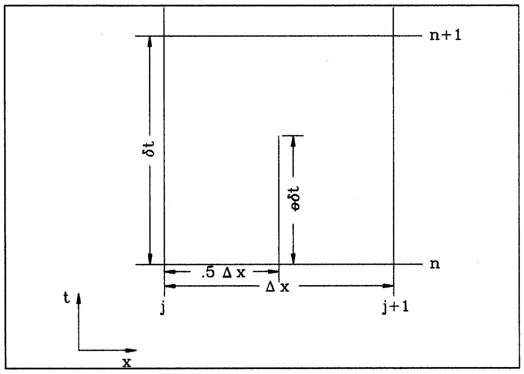
$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0$$

$$S_f = \frac{Q|Q|n^2}{2.208R^{4/3}A^2}$$

Where: V = velocity
g = acceleration due to gravity.
$\frac{\partial z}{\partial x}$ = water surface slope.
S_f = friction slope.
n = Manning's roughness estimate.
R = hydraulic radius = area/wetted perimeter.

The most successful and accepted procedure for approximating solutions to the non-linear unsteady flow equations is with a four-point implicit solution scheme, also known as a box scheme (Brunner 2010). The HEC-RAS Hydraulic Reference Manual describes this as follows:

Under this scheme, space derivatives and function values are evaluated at an interior point, $(n+\theta)\Delta t$. Thus values at $(n+1)\Delta t$ enter into all terms in the equations. For a reach of a river, a system of simultaneous equations results. The simultaneous solution is an important aspect of this scheme because it allows information from the entire reach to influence the solution at any one point Consequently, the time step can be significantly larger than with explicit numerical schemes.



[Typical finite difference cell used in HEC-RAS computations (Brunner and Goodwell, 2002).]

The general implicit finite difference forms are as follows:

The time derivative is approximated as: $\dfrac{\partial f}{\partial t} \approx \dfrac{\Delta f}{\Delta t} = \dfrac{0.5(\Delta f_{j+1} + \Delta f_j)}{\Delta t}$

The space derivative is approximated as: $\dfrac{\partial f}{\partial x} \approx \dfrac{\Delta f}{\Delta x} = \dfrac{(f_{j+1} - f_j) + \theta(\Delta f_{j+1} - \Delta f_j)}{\Delta x}$

The function value is: $f \approx \overline{f} = 0.5(f_j + f_{j+1}) + 0.5\theta(\Delta f_j + \Delta f_{j+1})$

Where: Δ = difference or change in.

Using this methodology, the finite difference form of the continuity equation used by HEC-RAS (which separates channel and floodplain flow) is:

$$\Delta Q + \frac{\Delta A_c}{\Delta t}\Delta x_c + \frac{\Delta A_f}{\Delta t}\Delta x_f + \frac{\Delta S}{\Delta t}\Delta x_f - \overline{Q_l} = 0$$

Where: c = channel.
f = floodplain.
$\overline{Q_l}$ = average lateral inflow.

Assuming a horizontal water surface across the cross section and perpendicular flow to the plane of the cross section, the finite difference form of the momentum equation is:

$$\frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t \Delta x_e} + \frac{\Delta(\beta V Q)}{\Delta x_e} + g\overline{A}\left(\frac{\Delta z}{\Delta x_e} + \overline{S_f} + \overline{S_h}\right) = \xi \frac{Q_l V_l}{\Delta x_e}$$

Where: Δx_e = equivalent flow path

$\Delta(\beta VQ) = \Delta(V_c Q_c) + \Delta(V_f Q_f)$

S_f = frictional slope for the entire cross section.

S_h = local frictional slope, from bridge piers, navigation dams, cofferdams, ect.

Q_l = lateral inflow.

V_l = average velocity of lateral inflow.

ξ = fraction of momentum entering a receiving stream.

If the implicit finite difference solution scheme is applied directly to these non-linear equations, a series of non-linear algebraic equations result. To avoid the resulting slow and unstable iteration solution schemes, these equations are linearized for their use in HEC-RAS (Brunner 2010).

For a more comprehensive presentation of the solution equations and techniques used in HEC-RAS, please see the HEC-RAS Hydraulic Reference Manual (Brunner 2010).

Upper Model

Using sections developed in HEC-GeoRAS, an ArcGIS extension, and geometry developed from both a 10-meter DEM (based on 7.5-minutes USGS quadrangles) and supplemental surveyed cross sections in the vicinity of the BNSF railroad embankment, an unsteady flow model was developed from the B-3 embankment to just below the railroad crossing of Coal Creek, adjacent to I-25 exit 281. This hydraulic model was inherently stable and provides reasonable estimates of peak discharge and water surface elevations given the limited geometric data available.

A normal depth boundary condition assumption was made at the downstream limit of the model (slope = 0.0091) and an initial flow of 500 cfs was assumed at all sections.

Split Flow

In the vicinity of I-25 exit 281, high flow through the Coal Creek valley will split between parallel valleys, with the western flow path continuing down Coal Creek through Wellington and the eastern flow path proceeding through Clark Reservoir before joining again with Coal Creek downstream of Wellington. An aerial image with 2-foot contours of the splitting flow is provided in Figure 5. These 2-foot contours are an extrapolation from the 10-foot contours but are helpful for judging what proportion of the total flow will pass through each of the valleys. A low dyke was constructed to divert flows from Coal Creek into Clark Reservoir but during a dam breach this earthen feature would be quickly overtopped and breached; it will likely have minimal impact.

During a B-3 breach flood, momentum and a partial barrier formed by I-25 will encourage the flow to continue strait through the Coal Creek valley instead of towards Clark Reseroir. Due to this momentum, it is reasonable to make the general assumption that more than half of the flow will continue down Coal Creek to Wellington. With this assumption it is very unlikely that no risk for loss of life exists in Wellington in the case of a breach. However, some minority of the discharge is expected to flow towards Clark Reservoir. Using flow path widths at the split as guidance, with the Coal Creek flow path having roughly twice the width as the path towards Clark reservoir, it is assumed in the modeling that 2/3 of the flow will pass through the western valley and 1/3 through the eastern valley.

Figure 5: Flow split.

West Model: Wellington

Using sections developed in HEC-GeoRAS and geometry developed from both a 10-meter DEM and supplemental surveyed cross sections in Wellington, an unsteady flow model was developed for this western portion of the flow split, from the point of the split to the confluence with the eastern portion of the flow split downstream of Wellington. Stability in the unsteady flow solution was initially a problem, indicating unreasonable results or causing non-convergence in the vicinity of the Jefferson Avenue bridge (station 154700). The unreasonable results consisted of excessive modeled depths at the bridge, and flat gradients, ponding and unreasonable attenuation just upstream of the bridge. This instability was corrected by not modeling the Jefferson Avenue bridge and its associated channel, which provided minimal flow conveyance compared to the overall flow at this section. The model still has a few relatively-insignificant points of instability but these points are isolated and do not produce outliers -- they do not significantly impact the peak flow and attenuation estimates of the model. However, in channel velocity estimates in for the sections immediately upstream and downstream of this point will be inaccurate.

The road crossing at Washington Avenue (dual culverts) and Cleveland Avenue (bridge) were modeled. Other crossings, such as the railroad, the two I-25 crossings, and a number of lesser crossings were not modeled, due to limited geometric data availability (I-25 and railroad) and expected insignificant impacts (lesser crossings).

A normal depth boundary condition assumption was made at the downstream limit of the model (slope = 0.0055) and an initial flow of 500 cfs was assumed at all sections.

East Model: Clark Reservoir

Using sections developed in HEC-GeoRAS and geometry developed from a 10-meter DEM, as well as construction drawings of Clark Reservoir, an unsteady flow model was developed for this eastern portion of the flow split, from the point of the split to the confluence with the western portion of the flow split downstream of Wellington. The reservoir embankment and auxiliary spillway were included in the model; however, the principal spillway was not modeled. Figures 6 and 7 illustrate the upstream and downstream embankment faces of Clark Reservoir.

A normal depth boundary condition assumption was made at the downstream limit of the model (slope = 0.0047) and an initial flow of 500 cfs was assumed at all sections.

Figure 6: Upstream face of Clark Reservoir embankment.

Figure 7: Downstream face of Clark Reservoir embankment.

Lower Model

Using sections developed in HEC-GeoRAS and geometry developed from a 10-meter DEM, an unsteady flow model was developed for the lower portion of the model, to just below the confluence with the Cache la Poudre River. A normal depth boundary condition assumption was made at the downstream limit of the model (slope = 0.0029) and an initial flow of 1000 cfs was assumed at all sections.

INUNDATION EXTENT AND TIMING

This analysis provided a prediction of the extent and timing of flooding from a catastrophic breach of the Boxelder B-3 dam embankment. The extent of the expected inundation is shown in Figure 8. These results are sufficient for an evaluation of the hazard classification and for developing an emergency action plan. However, due to limited available geometric data, the model only provides an approximate extent of inundation in the case of a breach. The nature and limitations of these predictions must be kept in mind when using these results.

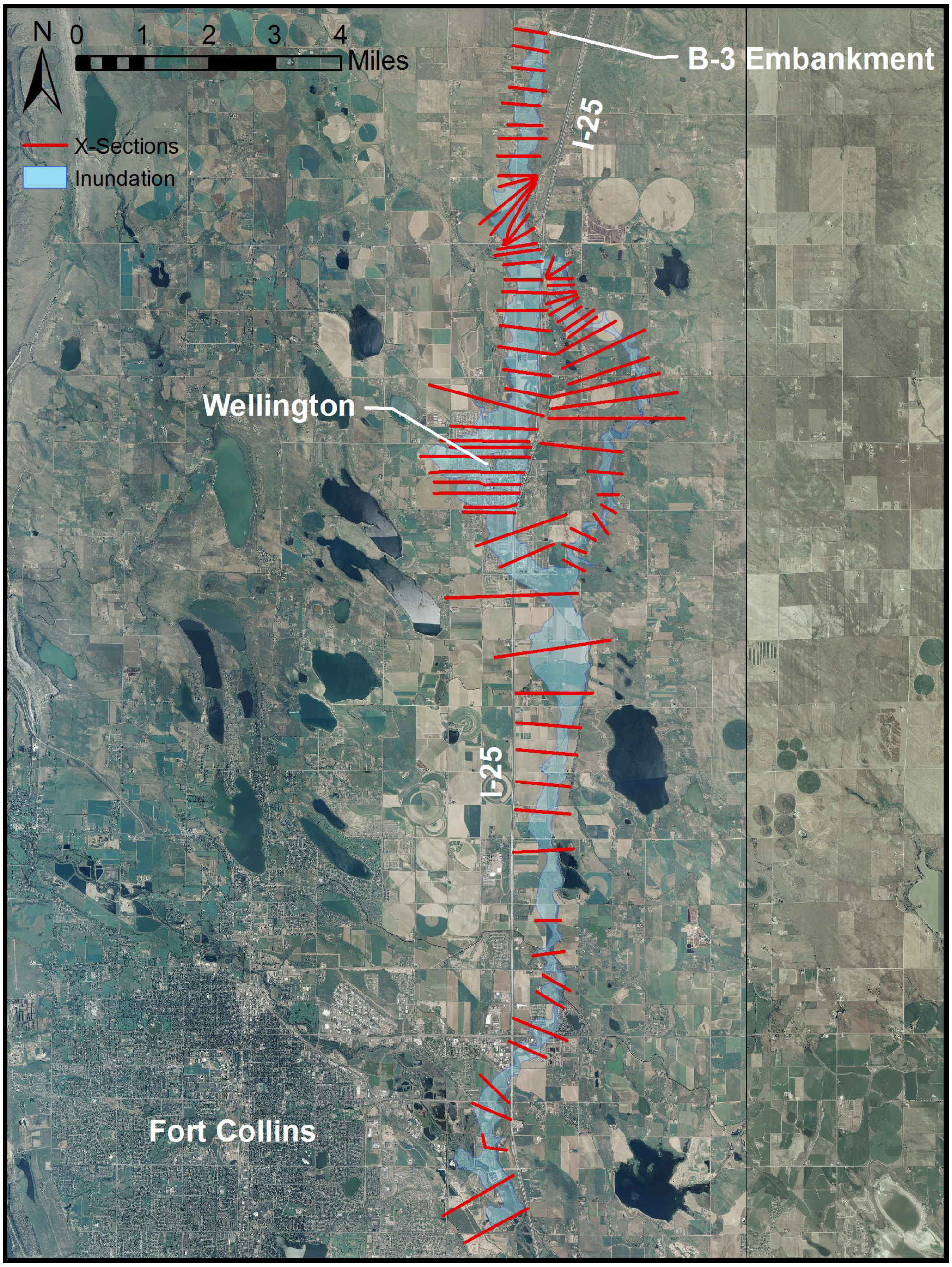


Figure 8: Inundation extent, Boxelder B-3 breach analysis.

Starting with a peak flow of 54,500 cfs at B-3, the flow attenuates to 45,000 cfs at the BNSF railroad embankment crossing then splits in the vicinity of I-25, exit 281. It is assumed that 1/3 of the breach volume flows towards Clark Reservoir while the remaining flows towards Wellington. With this assumption, Clark Reservoir is modeled to contain its portion of the breach flow without overtopping, with 7700 cfs exiting the reservoir, while Wellington will have a peak flow of 24,000 cfs flowing through town. The split flow paths rejoin below Wellington, with a peak flow of 31,000 cfs that attenuates to 28,000 cfs at Fort Collin's Mulberry Avenue and 24,000 cfs at the Cache la Poudre River. This discharge corresponds to about 2.1-times the 100-year flood event of 11,200 cfs (Appendix C).

The maximum inundation extent and timing are provided (Appendix A). Tables imbedded within these figures indicate peak discharge at each section, maximum depth and velocities, and breach wave timing and steepness. Points with computed depth*velocity values are included, with a product of 7 being assumed as a threshold for endangering life. Additionally, a few selected cross sections in Wellington are provided (Appendix B). These sections include the water surface elevations, structures and relevant hydraulic characteristics of the peak flow.

Table 4 provides the analysis results at the modeled cross sections. Figure 9 illustrates the routed breach hydrographs at 6 points within the analysis extent. The extent of inundation with expected depth*velocity products greater than 7 indicates that, in the unlikely case of such a breach, hundreds of homes and businesses will be threatened with damage or destruction, farm and ranch land will be flooded, a railroad, several highways and I-25 will be inundated, bridges may be damaged, and many lives could be lost. Due to this potential, **it is recommended that the hazard classification of this structure be increased from its current significant level to a high hazard classification**.

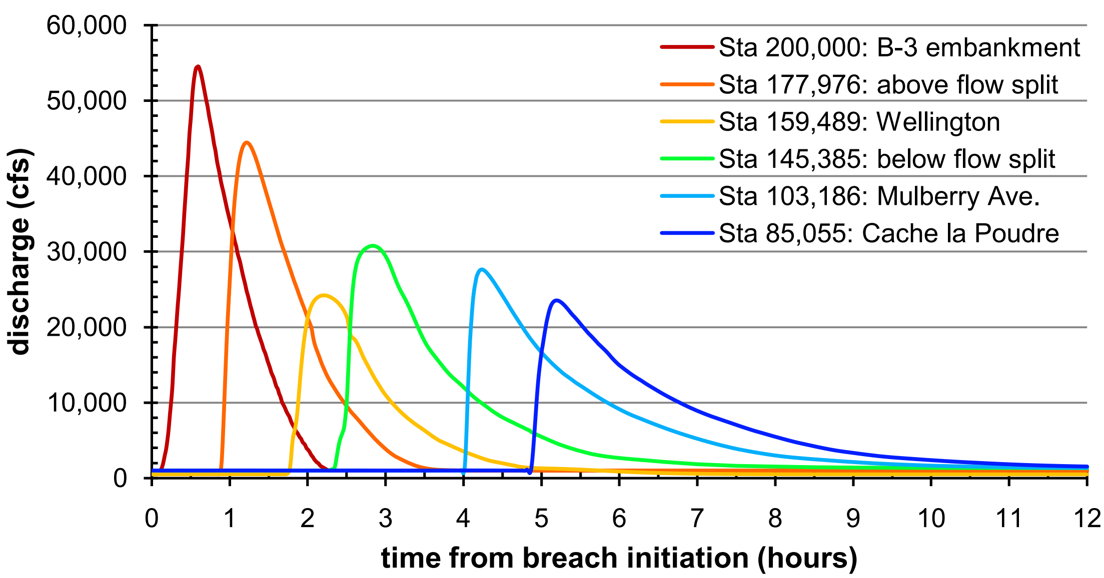


Figure 9: Breach hydrographs.

Table 4: Breach analysis results at maximum water surface elevation, Boxelder B-3 breach analysis.

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (feet)	Peak Velocity			Froude Number	
			Channel (ft/s)	Left (ft/s)	Right (ft/s)	Channel	X-Section
200,000	54,500	5455.87	14.1	8.4	4.9	0.77	0.90
198,211	54,100	5445.39	13.7	6.5	9.1	0.72	0.78
196,326	53,300	5433.37	9.7	7.5	7.1	0.49	0.61
194,657	52,700	5424.25	10.9	5.4	7.3	0.56	0.71
192,084	52,400	5413.71	12.7	6.6	7.3	0.64	0.73
190,166	52,200	5404.00	9.4	6.2	4.3	0.54	0.61
188,665	52,100	5395.73	10.1	5.5	6.0	0.60	0.62
186,172	51,900	5383.59	15.6	5.0	9.4	0.86	1.04
184,496	51,700	5372.67	8.9	6.4	4.5	0.56	0.53
183,195	49,800	5364.02	17.4	6.8	7.0	1.12	1.03
181,975	44,800	5360.46	13.2	8.2	6.7	0.67	0.60
178,738	44,500	5339.13	7.0	5.2	4.5	0.49	0.45
177,976	44,500	5335.50	8.5	7.7	6.9	0.82	0.78
177,483	44,400	5331.09	10.5	8.1	6.8	0.91	0.87
176,563	29,600	5324.02	7.9	6.2	5.1	0.68	0.65
175,163	29,600	5313.46	8.1	5.3	6.2	0.75	0.71
174,094	29,500	5306.68	6.5	5.2	4.4	0.50	0.51
172,612	29,300	5298.53	7.3	4.7	4.3	0.57	0.56
171,072	29,200	5289.28	4.2	6.0	4.3	0.41	0.58
169,137	29,100	5276.63	7.4	5.3	4.7	0.70	0.63
167,114	28,900	5264.38	8.8	4.7	5.2	0.77	0.63
165,478	28,800	5253.01	5.0	6.8	4.2	0.51	0.75
164,063	28,700	5239.69	7.4	4.1	6.7	0.74	0.66
162,173	24,400	5226.24	6.0	3.2	10.0	0.48	0.84
161,002	24,300	5224.38	2.0	1.0	1.7	0.10	0.13
160,350	24,200	5219.44	7.2	3.3	1.6	0.52	0.55
159,489	24,200	5213.80	5.4	3.2	1.3	0.44	0.46
158,007	23,600	5206.64	5.1	2.1	2.3	0.36	0.34
156,984	23,400	5198.46	6.2	2.5	1.4	0.44	0.34
156,227	23,400	5194.97	5.5	2.8	1.5	0.45	0.38
154,947	18,500	5190.02	1.9	2.1	0.9	0.19	0.20
154,468	23,300	5184.29	7.8	3.4	5.1	0.64	0.66
152,161	23,300	5171.17	5.0	3.7	4.1	0.38	0.45
149,765	23,200	5161.24	2.6	3.4	1.5	0.28	0.43
176,110	14,800	5307.64	6.4	5.4	2.0	0.48	0.42
169,070	7,670	5295.99	11.5	4.4	4.0	1.33	1.41
167,623	7,660	5276.78	5.8	3.4		0.71	0.72
165,912	7,650	5257.21	6.2	3.1	2.6	0.60	0.65
164,979	7,650	5251.57	8.9	1.7	4.4	0.81	0.88
163,633	7,650	5237.93	9.4	7.2	4.7	0.78	0.93
162,223	7,640	5224.38	5.6		3.5	0.59	0.60
159,375	7,630	5207.78	5.8	1.4	2.9	0.64	0.68
157,470	7,630	5190.06	9.1	5.2	5.4	0.69	0.70
155,503	7,630	5179.19	9.1	6.8	5.6	0.84	0.80
154,263	7,620	5170.25	5.5	3.6	5.5	0.38	0.46
152,862	7,610	5166.13	5.2	3.6	3.5	0.43	0.43
151,145	7,600	5160.40	7.0	4.8	3.8	0.71	0.71
149,643	7,590	5151.56	6.7	4.0	3.7	0.60	0.61
148,140	7,590	5144.83	7.1	3.7	4.6	0.65	0.67
145,385	30,800	5136.33	7.5	2.8	4.8	0.66	0.63
140,740	30,700	5113.58	6.4	4.4	3.9	0.62	0.65
136,272	30,500	5091.79	6.7	2.3	4.3	0.56	0.59
133,245	30,400	5074.09	9.6	5.4	6.9	0.65	0.70
130,873	30,300	5060.56	7.2	6.4	4.4	0.48	0.53

Table 4 (cont.): Breach analysis results at maximum water surface elevation, Boxelder B-3 breach analysis.

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (feet)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Froude Number Channel	Froude Number X-Section
140,740	30,700	5113.58	6.4	4.4	3.9	0.62	0.65
136,272	30,500	5091.79	6.7	2.3	4.3	0.56	0.59
133,245	30,400	5074.09	9.6	5.4	6.9	0.65	0.70
130,873	30,300	5060.56	7.2	6.4	4.4	0.48	0.53
128,301	30,000	5049.37	9.2	2.5	5.2	0.67	0.69
125,974	29,700	5038.20	8.8	4.4	4.7	0.61	0.62
122,750	29,500	5018.49	7.0	2.8	4.1	0.54	0.59
116,596	28,500	4998.54	7.7	4.8	7.7	0.57	0.68
113,577	28,400	4980.97	9.9	6.0	5.4	0.65	0.64
110,761	28,200	4970.44	7.7	4.5	6.9	0.54	0.62
106,370	26,600	4955.36	8.1	2.0	4.4	0.50	0.59
103,186	27,600	4938.35	6.0	2.6	2.4	0.47	0.45
100,821	26,800	4929.45	6.2	4.0	3.7	0.44	0.41
95,705	26,000	4908.72	7.0	5.7	5.6	0.54	0.62
93,642	25,800	4898.38	5.0	4.5	3.9	0.40	0.48
90,634	25,300	4886.08	7.4	2.7	5.7	0.59	0.70
87,918	24,700	4869.42	4.1	2.1	2.1	0.27	0.29
85,055	23,500	4862.78	2.2	1.1	3.4	0.18	0.39

SUMMARY AND CONCLUSIONS

A comprehensive approach was implemented to develop a most likely breach hydrograph of the Boxelder B-3 embankment, in the unlikely case of a breach. The methods implemented included peak flow equations developed by NRCS, Froehlich, Kirkpatrick, and the U.S. Bureau of Reclamation; breach geometry prediction using Froehlich, U.S. Bureau of Reclamation, and Von Thun and Gillette; and breach formation time using Froehlich, MacDonald and Langridge-Monoplolis, U.S. Bureau of Reclamation, and Von Thun and Gillette. After reviewing the various results, which provided a wide range of peak flow values, and recognizing the similarity in the median predicted peak flow and the HEC-RAS breach geometry prediction model, it was decided to use the breach hydrograph as developed by the HEC-RAS breach simulation, with a peak of 54,500 cfs, formation time of 0.6 hours, and volume of 3950 acre-feet.

The breach hydrograph was routed using HEC-RAS 4.1 from the embankment to the confluence with the Cache la Poudre River, 19 miles downstream. The flow attenuates to 45,000 cfs at the BNSF railroad crossing, the flow splits with 24,000 cfs in Wellington and 7700 cfs exiting Clark Reservoir, and the split flows recombines, with 28,000 cfs in the eastern suburbs of Fort Collins and 24,000 cfs at the Cache la Poudre River.

In and in the vicinity of the most-populated portion of the floodway, Wellington, the extent of inundation with expected depth*velocity products greater than 7 indicate that hundreds of homes and businesses will be threatened with damage or destruction, farm and ranch land will be flooded, a railroad, several highways and I-25 will be inundated, bridges may be damaged, and many lives could be lost. Due to this potential, it is recommended that the hazard classification of the Boxelder B-3 structure be increased from its current significant level to a high hazard.

REFERENCES

Brunner, Gary W. 2010. *HEC-RAS River Analysis System, Hydraulic Reference Manual* US Army Corps of Engineers, Hydraulic Engineering Center (HEC), CPD-69.

Brunner, Gary W. 2010 *HEC-RAS River Analysis System, User's Manual* US Army Corps of Engineers, Hydraulic Engineering Center (HEC), CPD-68.

Froehlich, David C. 2008. Embankment Dam Breach Parameters and Their Uncertainties. *ASCE Journal of Hydraulic Engineering* volume 134, issue 12, pp. 1708-1721.

Froehlich, David C. 1995a. Peak Outflow from Breached Embankment Dam. *Journal of Water Resources Planning and Management*, vol. 121, no.1, 90-97.

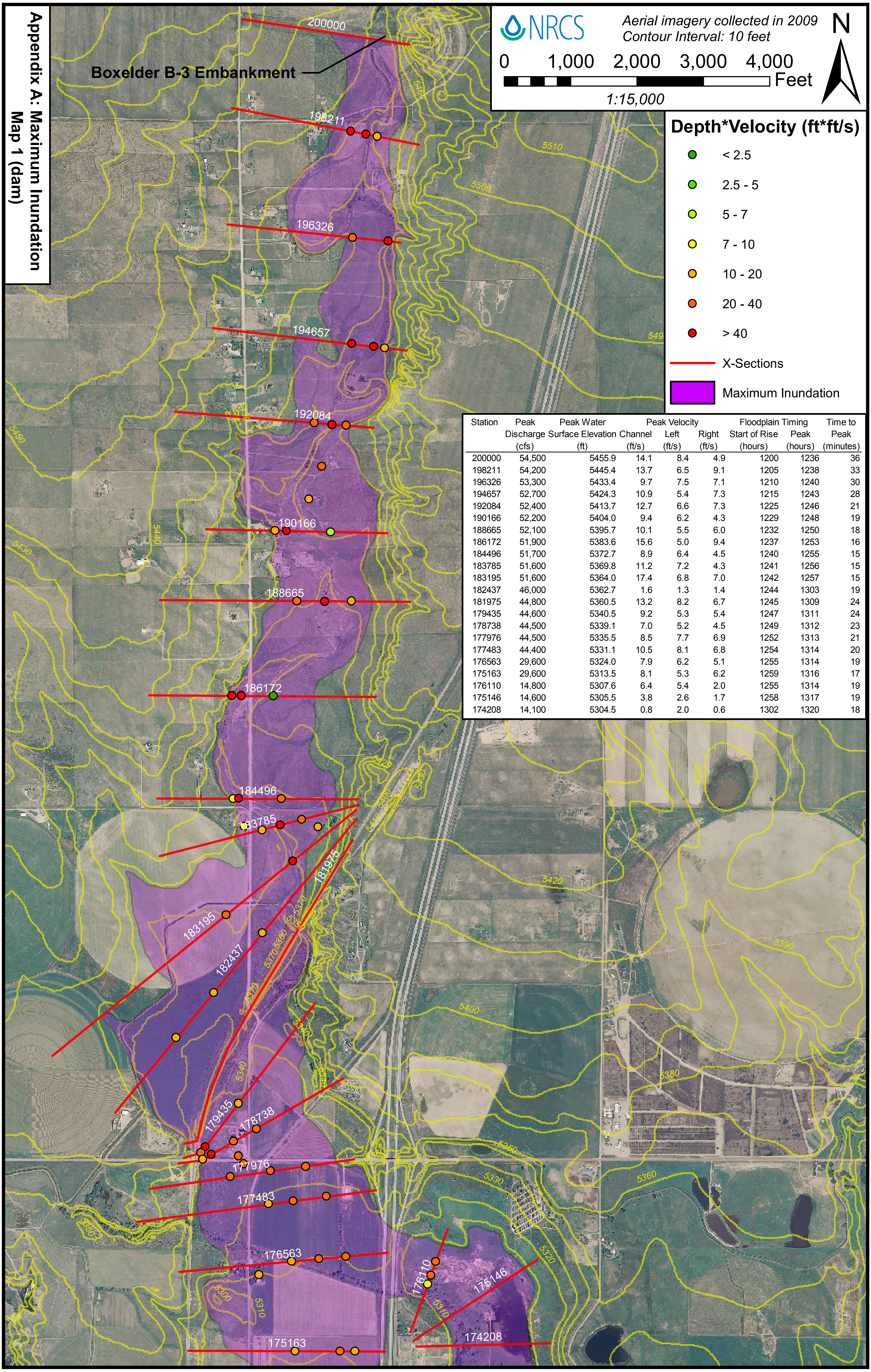
Froehlich, D.C. 1995b. Embankment Dam Breach Parameters Revisited. *Proc., First International Conference on Water Resources Engineering*, ASCE, Reston, VA, 887-891.

Natural Resources Conservation Service (NRCS) 2005. *Earth Dams and Reservoirs, TR-60* U.S. Department of Agriculture, Natural Resources Conservation Service, Conservation Engineering Division, July 2005.

Wahl, T.L. 2004. Uncertainty of Predictions of Embankment Breach Parameters. *Journal of Hydraulic Engineering* vol. 130, no. 5, 389-397.

Wahl, Tony L. 1998. *Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment* U.S. Department of Interior, Bureau of Reclamation, Dam Safety Office, DSO-98-004.

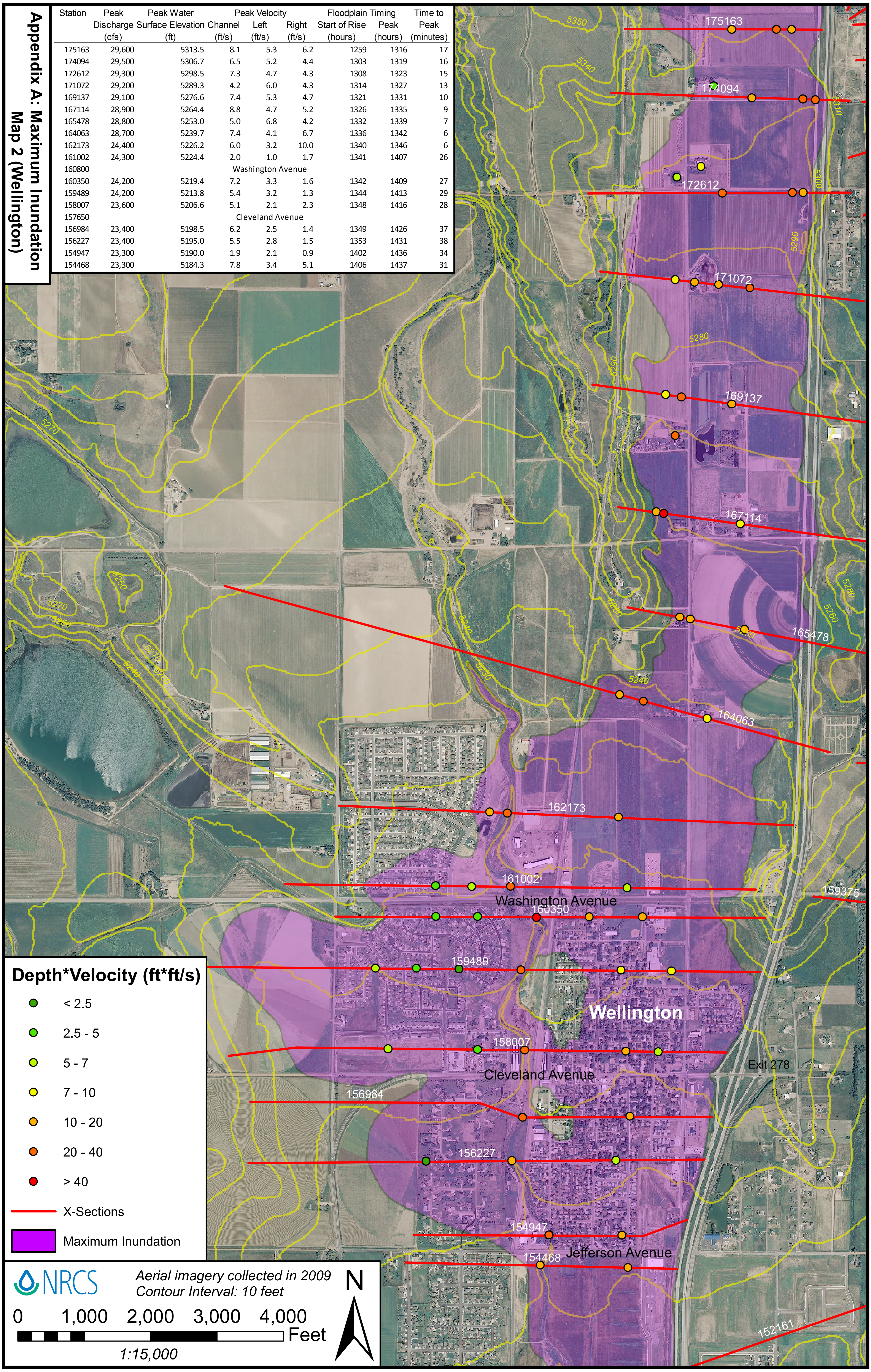
APPENDIX A: Maximum Likely Inundation



Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodplain Timing Start of Rise (hours)	Floodplain Timing Peak (hours)	Time to Peak (minutes)
200000	54,500	5455.9	14.1	8.4	4.9	1200	1236	36
198211	54,200	5445.4	13.7	6.5	9.1	1205	1238	33
196326	53,300	5433.4	9.7	7.5	7.1	1210	1240	30
194657	52,700	5424.3	10.9	5.4	7.3	1215	1243	28
192084	52,400	5413.7	12.7	6.6	7.3	1225	1246	21
190166	52,200	5404.0	9.4	6.2	4.3	1229	1248	19
188665	52,100	5395.7	10.1	5.5	6.0	1232	1250	18
186172	51,900	5383.6	15.6	5.0	9.4	1237	1253	16
184496	51,700	5372.7	8.9	6.4	4.5	1240	1255	15
183785	51,600	5369.8	11.2	7.2	4.3	1241	1256	15
183195	51,600	5364.0	17.4	6.8	7.0	1242	1257	15
182437	46,000	5362.7	1.6	1.3	1.4	1244	1303	19
181975	44,800	5360.5	13.2	8.2	6.7	1245	1309	24
179435	44,600	5340.5	9.2	5.3	5.4	1247	1311	24
178738	44,500	5339.1	7.0	5.2	4.5	1249	1312	23
177976	44,500	5335.5	8.5	7.7	6.9	1252	1313	21
177483	44,400	5331.1	10.5	8.1	6.8	1254	1314	20
176563	29,600	5324.0	7.9	6.2	5.1	1255	1314	19
175163	29,600	5313.5	8.1	5.3	6.2	1259	1316	17
176110	14,800	5307.6	6.4	5.4	2.0	1255	1314	19
175146	14,600	5305.5	3.8	2.6	1.7	1258	1317	19
174208	14,100	5304.5	0.8	2.0	0.6	1302	1320	18

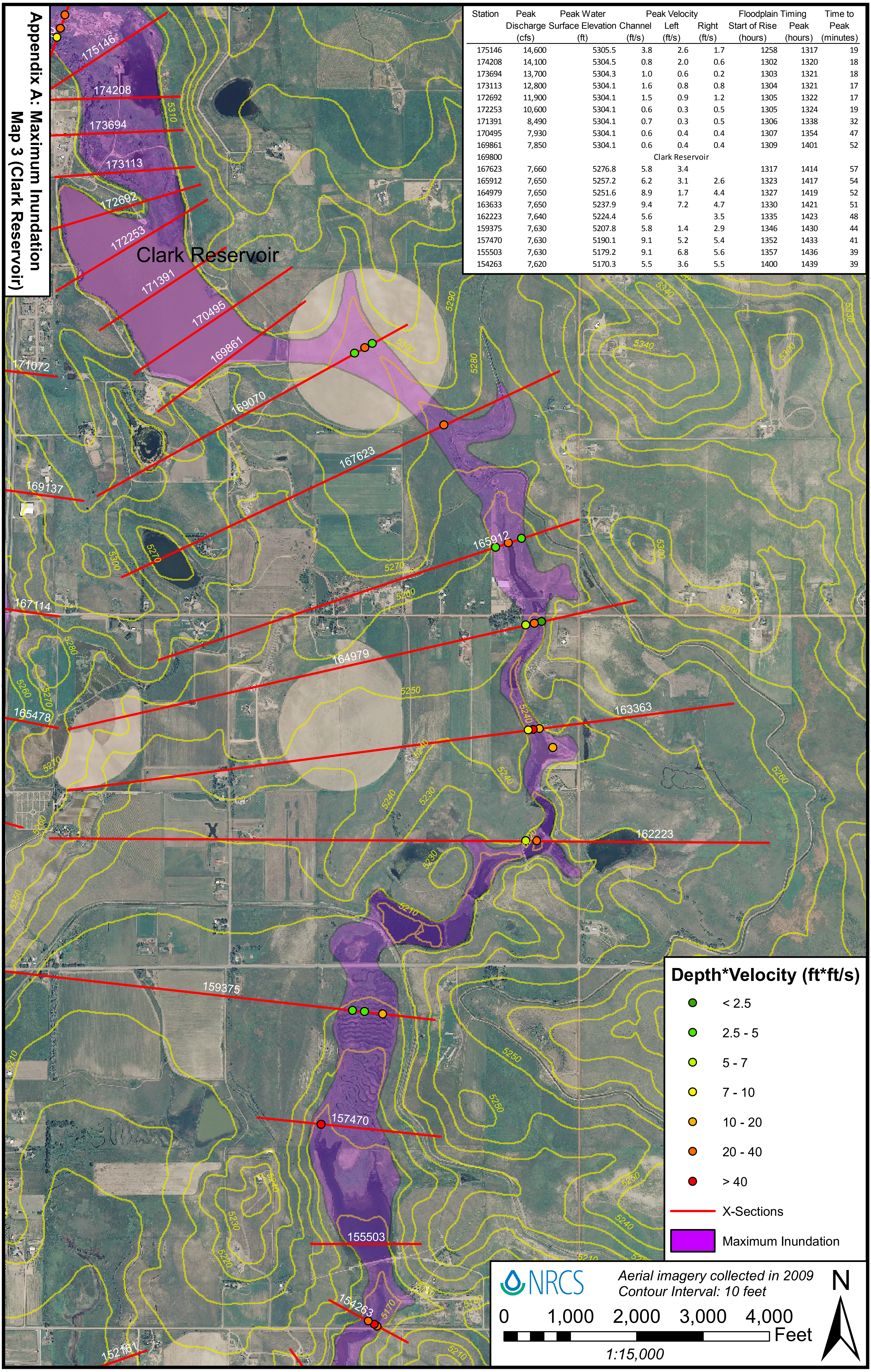
Appendix A: Maximum Inundation Map 2 (Wellington)

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodplain Timing Start of Rise (hours)	Floodplain Timing Peak (hours)	Time to Peak (minutes)
175163	29,600	5313.5	8.1	5.3	6.2	1259	1316	17
174094	29,500	5306.7	6.5	5.2	4.4	1303	1319	16
172612	29,300	5298.5	7.3	4.7	4.3	1308	1323	15
171072	29,200	5289.3	4.2	6.0	4.3	1314	1327	13
169137	29,100	5276.6	7.4	5.3	4.7	1321	1331	10
167114	28,900	5264.4	8.8	4.7	5.2	1326	1335	9
165478	28,800	5253.0	5.0	6.8	4.2	1332	1339	7
164063	28,700	5239.7	7.4	4.1	6.7	1336	1342	6
162173	24,400	5226.2	6.0	3.2	10.0	1340	1346	6
161002	24,300	5224.4	2.0	1.0	1.7	1341	1407	26
160800		Washington Avenue						
160350	24,200	5219.4	7.2	3.3	1.6	1342	1409	27
159489	24,200	5213.8	5.4	3.2	1.3	1344	1413	29
158007	23,600	5206.6	5.1	2.1	2.3	1348	1416	28
157650		Cleveland Avenue						
156984	23,400	5198.5	6.2	2.5	1.4	1349	1426	37
156227	23,400	5195.0	5.5	2.8	1.5	1353	1431	38
154947	23,300	5190.0	1.9	2.1	0.9	1402	1436	34
154468	23,300	5184.3	7.8	3.4	5.1	1406	1437	31

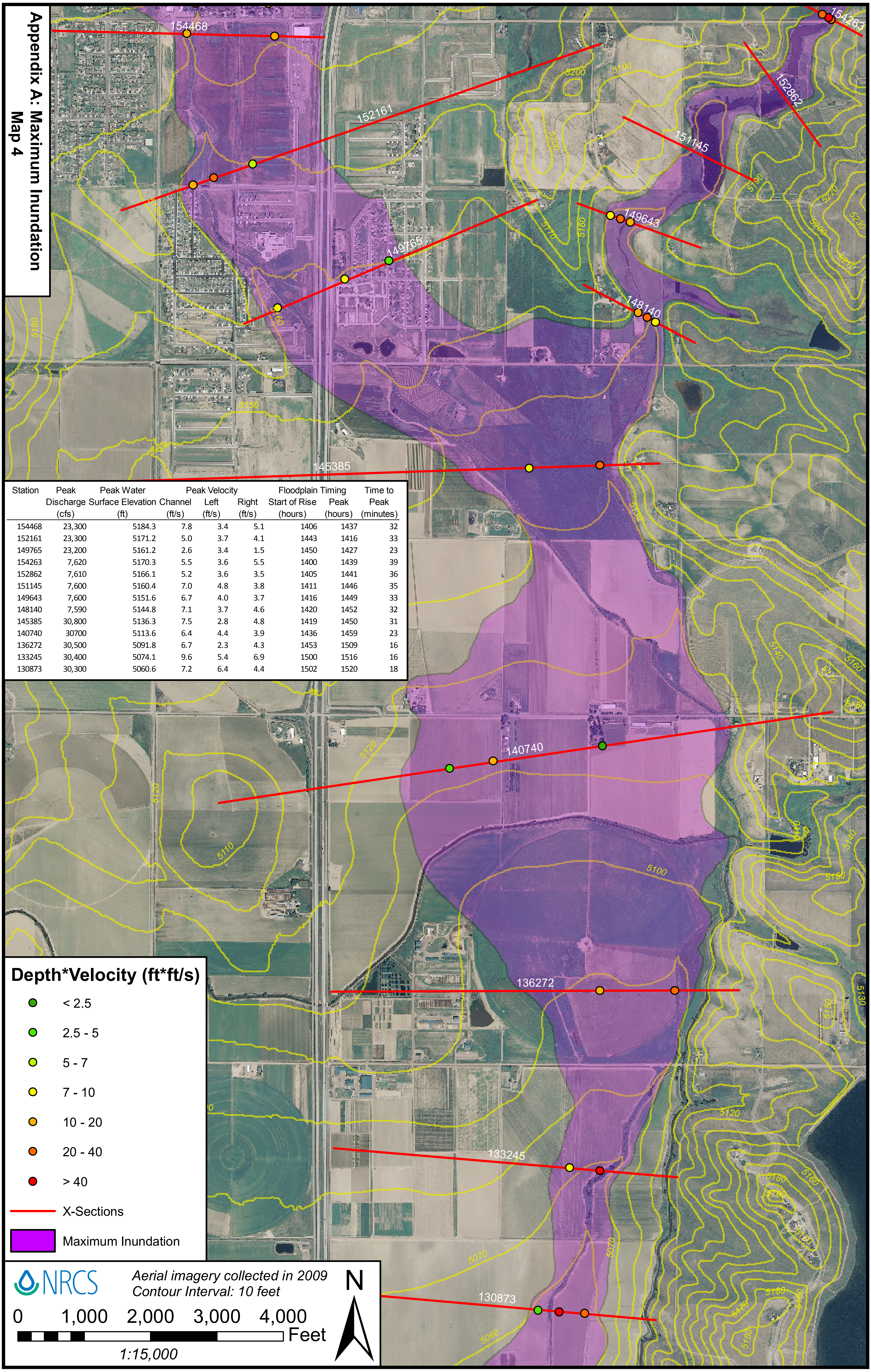


Appendix A: Maximum Inundation Map 3 (Clark Reservoir)

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Left (ft/s)	Right (ft/s)	Floodplain Timing Start of Rise (hours)	Peak (hours)	Time to Peak (minutes)
175146	14,600	5305.5	3.8	2.6	1.7	1258	1317	19
174208	14,100	5304.5	0.8	2.0	0.6	1302	1320	18
173694	13,700	5304.3	1.0	0.6	0.2	1303	1321	18
173113	12,800	5304.1	1.6	0.8	0.8	1304	1321	17
172692	11,900	5304.1	1.5	0.9	1.2	1305	1322	17
172253	10,600	5304.1	0.6	0.3	0.5	1305	1324	19
171391	8,490	5304.1	0.7	0.3	0.5	1306	1338	32
170495	7,930	5304.1	0.6	0.4	0.4	1307	1354	47
169861	7,850	5304.1	0.6	0.4	0.4	1309	1401	52
169800	Clark Reservoir							
167623	7,660	5276.8	5.8	3.4		1317	1414	57
165912	7,650	5257.2	6.2	3.1	2.6	1323	1417	54
164979	7,650	5251.6	8.9	1.7	4.4	1327	1419	52
163633	7,650	5237.9	9.4	7.2	4.7	1330	1421	51
162223	7,640	5224.4	5.6		3.5	1335	1423	48
159375	7,630	5207.8	5.8	1.4	2.9	1346	1430	44
157470	7,630	5190.1	9.1	5.2	5.4	1352	1433	41
155503	7,630	5179.2	9.1	6.8	5.6	1357	1436	39
154263	7,620	5170.3	5.5	3.6	5.5	1400	1439	39

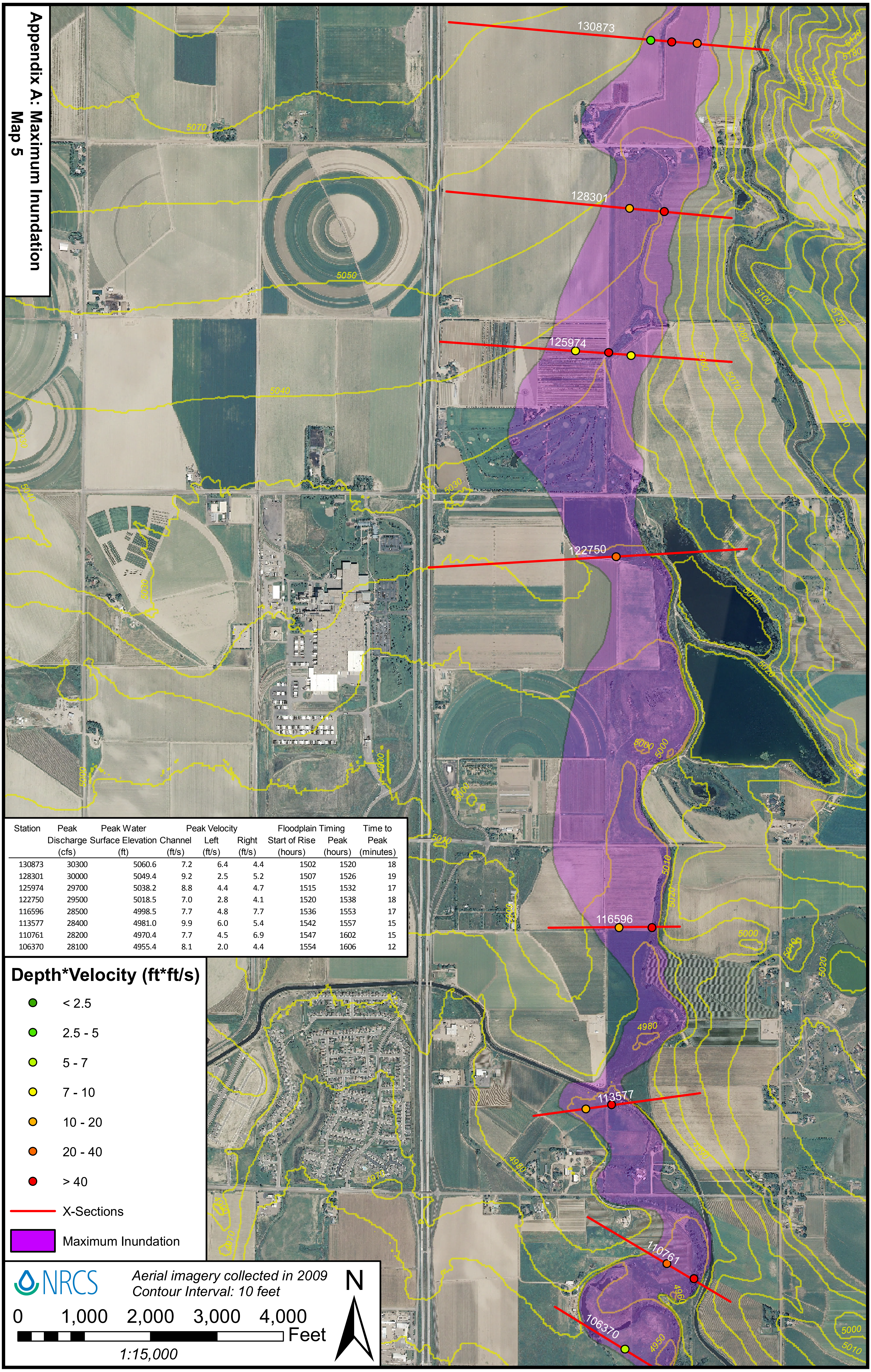


Appendix A: Maximum Inundation Map 4



Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodplain Timing Start of Rise (hours)	Floodplain Timing Peak (hours)	Time to Peak (minutes)
154468	23,300	5184.3	7.8	3.4	5.1	1406	1437	32
152161	23,300	5171.2	5.0	3.7	4.1	1443	1416	33
149765	23,200	5161.2	2.6	3.4	1.5	1450	1427	23
154263	7,620	5170.3	5.5	3.6	5.5	1400	1439	39
152862	7,610	5166.1	5.2	3.6	3.5	1405	1441	36
151145	7,600	5160.4	7.0	4.8	3.8	1411	1446	35
149643	7,600	5151.6	6.7	4.0	3.7	1416	1449	33
148140	7,590	5144.8	7.1	3.7	4.6	1420	1452	32
145385	30,800	5136.3	7.5	2.8	4.8	1419	1450	31
140740	30700	5113.6	6.4	4.4	3.9	1436	1459	23
136272	30,500	5091.8	6.7	2.3	4.3	1453	1509	16
133245	30,400	5074.1	9.6	5.4	6.9	1500	1516	16
130873	30,300	5060.6	7.2	6.4	4.4	1502	1520	18

Appendix A: Maximum Inundation
Map 5



Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodplain Timing Start of Rise (hours)	Floodplain Timing Peak (hours)	Time to Peak (minutes)
130873	30300	5060.6	7.2	6.4	4.4	1502	1520	18
128301	30000	5049.4	9.2	2.5	5.2	1507	1526	19
125974	29700	5038.2	8.8	4.4	4.7	1515	1532	17
122750	29500	5018.5	7.0	2.8	4.1	1520	1538	18
116596	28500	4998.5	7.7	4.8	7.7	1536	1553	17
113577	28400	4981.0	9.9	6.0	5.4	1542	1557	15
110761	28200	4970.4	7.7	4.5	6.9	1547	1602	15
106370	28100	4955.4	8.1	2.0	4.4	1554	1606	12

Depth*Velocity (ft*ft/s)

- < 2.5
- 2.5 - 5
- 5 - 7
- 7 - 10
- 10 - 20
- 20 - 40
- > 40

X-Sections

Maximum Inundation

NRCS

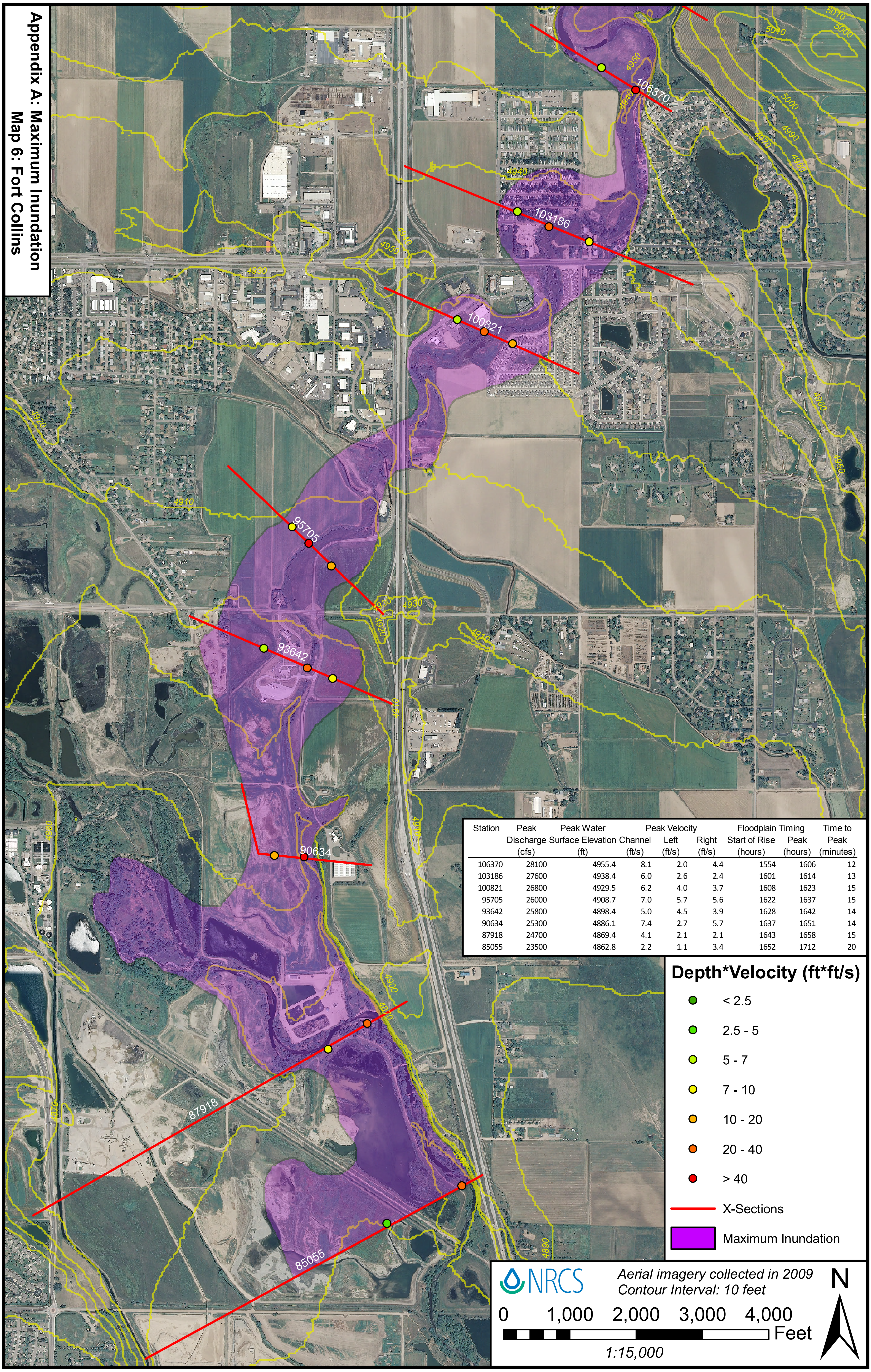
Aerial imagery collected in 2009
Contour Interval: 10 feet

0 1,000 2,000 3,000 4,000 Feet

1:15,000

N

Appendix A: Maximum Inundation
Map 6: Fort Collins

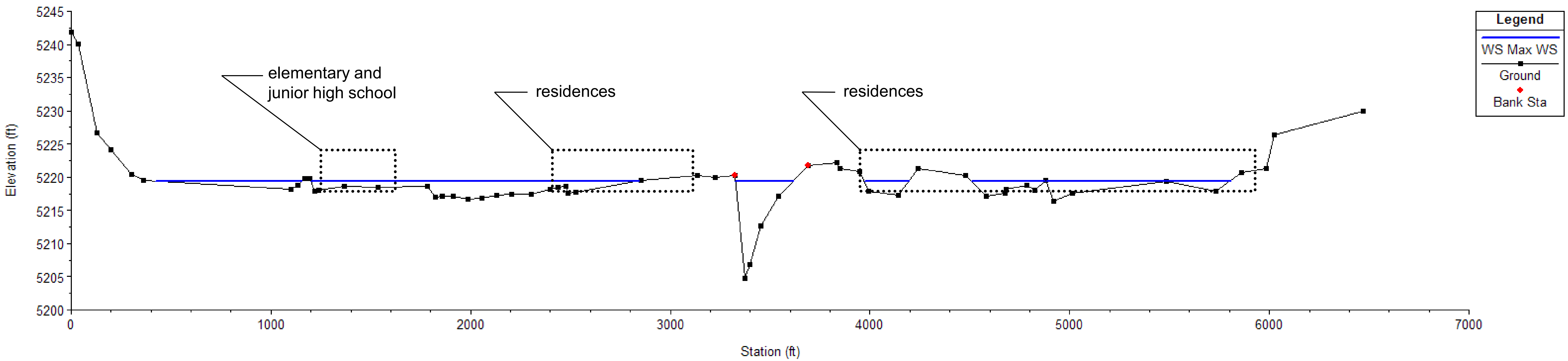


Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodplain Timing Start of Rise (hours)	Floodplain Timing Peak (hours)	Time to Peak (minutes)
106370	28100	4955.4	8.1	2.0	4.4	1554	1606	12
103186	27600	4938.4	6.0	2.6	2.4	1601	1614	13
100821	26800	4929.5	6.2	4.0	3.7	1608	1623	15
95705	26000	4908.7	7.0	5.7	5.6	1622	1637	15
93642	25800	4898.4	5.0	4.5	3.9	1628	1642	14
90634	25300	4886.1	7.4	2.7	5.7	1637	1651	14
87918	24700	4869.4	4.1	2.1	2.1	1643	1658	15
85055	23500	4862.8	2.2	1.1	3.4	1652	1712	20

APPENDIX B: Valley Cross Sections, Wellington

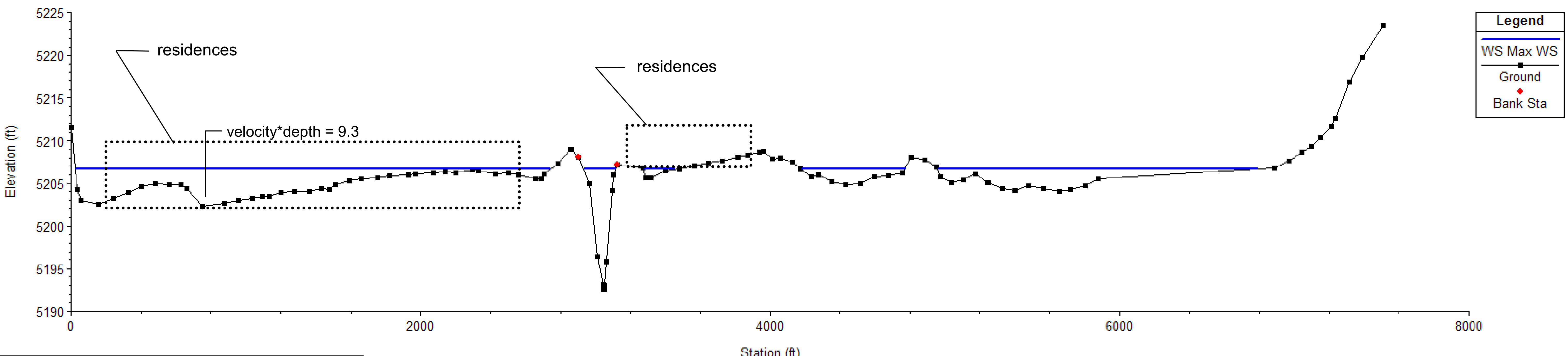
Section 160350

Peak Discharge: 24,200 cfs
Peak WSEL: 5219.44 ft
Peak velocity, channel: 7.1 fps
Peak velocity, LEFT floodplain: 3.3 fps
Peak velocity, RIGHT floodplain: 1.6 fps
Maximum depth, channel: 14.7 ft
Maximum depth, LEFT floodplain: 2.8 ft
Maximum depth, RIGHT floodplain: 3.0 ft



Section 158007

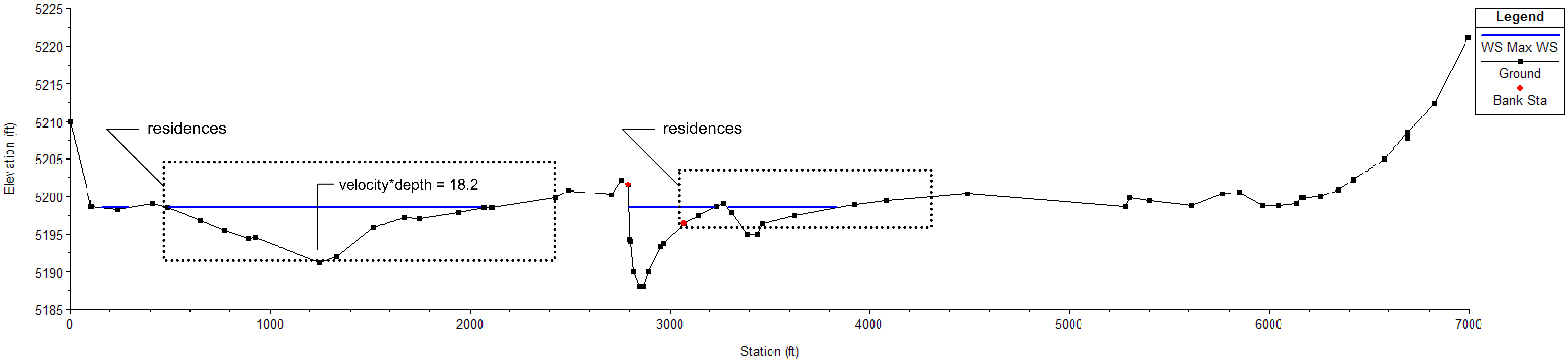
Peak Discharge: 23,600 cfs
Peak WSEL: 5206.64 ft
Peak velocity, channel: 5.1 fps
Peak velocity, LEFT floodplain: 2.1 fps
Peak velocity, RIGHT floodplain: 2.3 fps
Maximum depth, channel: 14.1 ft
Maximum depth, LEFT floodplain: 4.3 ft
Maximum depth, RIGHT floodplain: 2.7 ft



Appendix B: Valley cross sections in Wellington, upstream.

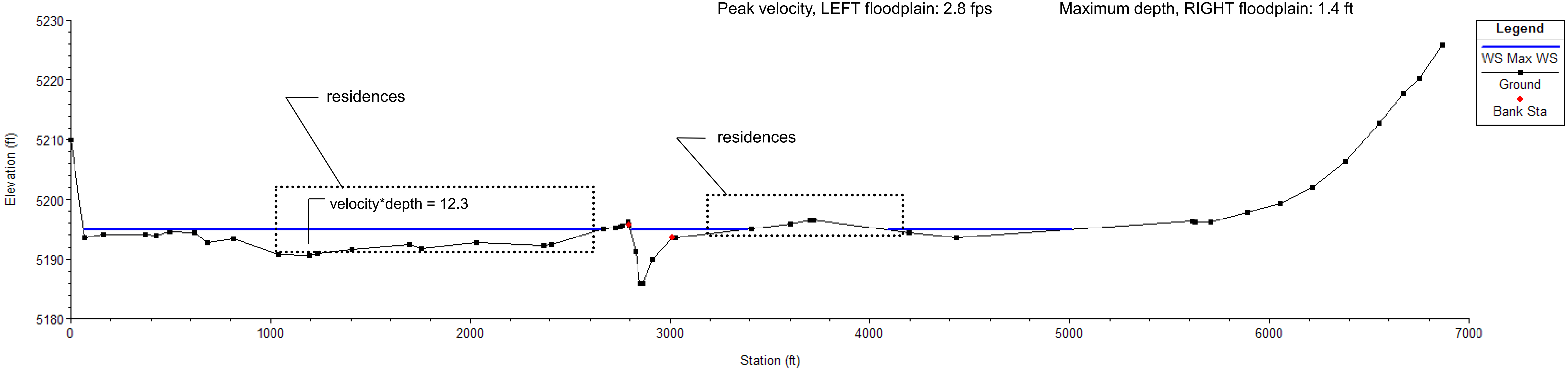
Section 156984

Peak Discharge: 73,100 cfs
Peak WSEL: 5200.81 ft
Peak velocity, channel: 5.8 fps
Peak velocity, LEFT floodplain: 3.9 fps
Peak velocity, RIGHT floodplain: 3.2 fps
Maximum depth, channel: 12.8 ft
Maximum depth, LEFT floodplain: 9.6 ft
Maximum depth, RIGHT floodplain: 5.9 ft



Section 156227

Peak Discharge: 23,400 cfs
Peak WSEL: 5194.97 ft
Peak velocity, channel: 5.5 fps
Peak velocity, LEFT floodplain: 2.8 fps
Peak velocity, RIGHT floodplain: 1.5 fps
Maximum depth, channel: 9.0 ft
Maximum depth, LEFT floodplain: 4.3 ft
Maximum depth, RIGHT floodplain: 1.4 ft



Appendix B: Valley cross sections in Wellington, downstream.

APPENDIX C: Cache la Poudre River Flow Frequency

Log-Pearson Frequency Analysis Spreadsheet, Version 2.3, 1/2005.

Project: *Boxelder Breach Studies*
Streamgage: *Cache la Poudre River above Boxelder*
Date: *2/3/2009* **Performed By:** *SEY*

Without Generalized Skew

Average: 7.3731
Standard Deviation: 0.80523295
Skew Coefficient[1]: 0.13591037

Length of systematic record: 27
Number of historic peaks: 0
Length of Data Record: 27
Length of Historic Record:[5] ----

Recurrence Interval[2] (years)	Percent Chance	K-Value	Ln(Q)	Peak[4] Discharge (cfs)	95% Confidence Limits Upper (cfs)	95% Confidence Limits Lower (cfs)
200	0.5	2.703	9.5499	14,000	28,400	8,810
100	1	2.426	9.3264	11,200	21,400	7,300
50	2	2.126	9.0847	8,820	15,800	5,950
25	4	1.797	8.8199	6,770	11,300	4,740
10	10	1.295	8.4160	4,520	6,880	3,330
5	20	0.834	8.0445	3,120	4,400	2,380
2	50	-0.023	7.3547	1,560	2,030	1,200
1.25	80	-0.847	6.6907	805	1,060	569
1.05	95	-1.605	6.0805	437	612	271

With Weighted Generalized Skew

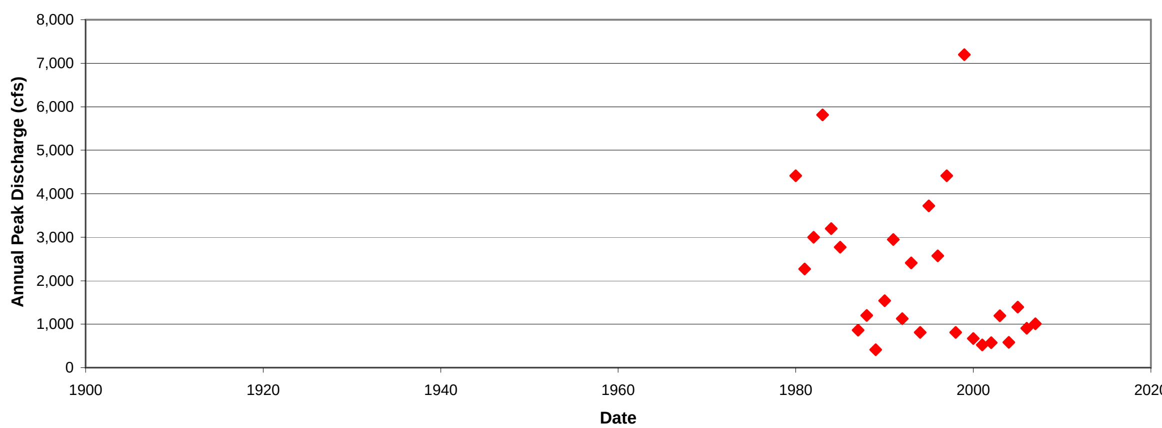
Generalized Skew Coefficient[3]:
Variance of Generalized Skew[3]:
A: -0.319127
B: 0.904663
station skew: 0.135910
MSE Station Skew: 0.19526925
Weighted skew coefficient[1]: 0

Recurrence Interval[2] (years)	Percent Chance	K-Value	Ln(Q)	Peak[4] Discharge (cfs)	95% Confidence Limits Upper (cfs)	95% Confidence Limits Lower (cfs)
200	0.5	2.576	9.4473	----	----	----
100	1	2.326	9.2460	----	----	----
50	2	2.054	9.0270	----	----	----
25	4	1.751	8.7830	----	----	----
10	10	1.282	8.4054	----	----	----
5	20	0.842	8.0511	----	----	----
2	50	0.000	7.3731	----	----	----
1.25	80	-0.842	6.6951	----	----	----
1.05	95	-1.645	6.0485	----	----	----

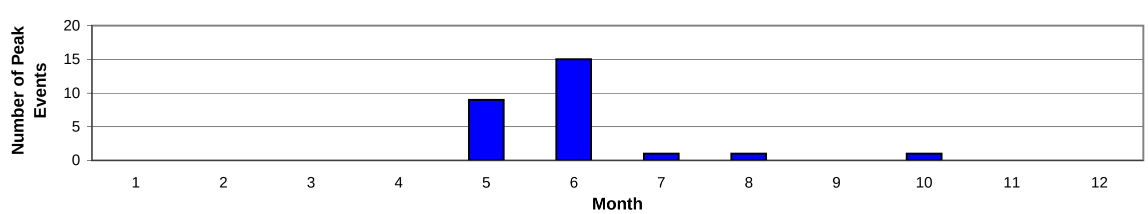
(1) Station and generalized skews must be between -2.00 and +3.00 in this spreadsheet.
(2) Considering the relatively short length of most gage records, less frequent peak estimates need to be used with considerable care.
(3) Computed one of four ways (see "generalized skew coefficient" worksheet): Mean and variance (standard deviation^2) of station skews coefficients in region; skew isolines drawn on a map or regions; skew prediction equations; read from Plate 1 of Bulletin 17B (reproduced in this spreadsheet), with Variance of Generalized Skew = 0.302.
(4) Results are automatically rounded to three significant figures, the dominant number of significant figures in the K-Value table.
(5) Historic frequency analysis assumes that intervening years reflect systematic record.

Comments:

Data Plot:



Peak Timing:



Log-Pearson Frequency Analysis Spreadsheet, Version 2.3, 1/2005.

Project: *Boxelder Breach Studies*
Streamgage: *Cache la Poudre River above Boxelder*
Date: *2/3/2009* **Performed By:** *SEY*

Input Data

Station ID: *06752280* **Latitude, Longitude:** *40-33-07* *105-00-39*
Drainage Area (mi^2): *1244* **County:** *Larimer*
Number of low outliers eliminated: *0* **State:** *Colorado*

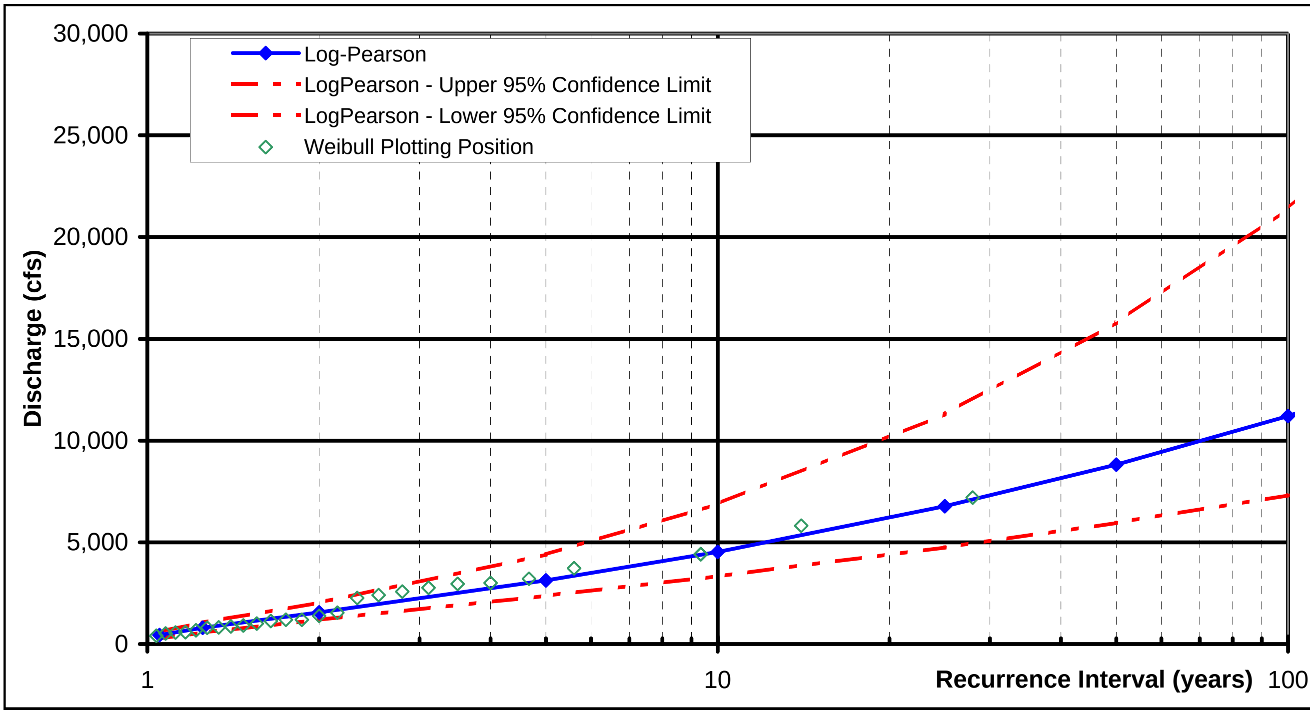
	Date	Discharge (cfs)	Historic?	Outlier?		Date	Discharge (cfs)	Historic?	Outlier?		Date	Discharge (cfs)	Historic?	Outlier?
1	05/25/1980	4,410	n	n	51	----	----	n	n	101	----	----	n	n
2	06/08/1981	2,270	n	n	52	----	----	n	n	102	----	----	n	n
3	06/30/1982	3,000	n	n	53	----	----	n	n	103	----	----	n	n
4	06/21/1983	5,810	n	n	54	----	----	n	n	104	----	----	n	n
5	05/25/1984	3,200	n	n	55	----	----	n	n	105	----	----	n	n
6	06/09/1985	2,770	n	n	56	----	----	n	n	106	----	----	n	n
7	05/23/1987	865	n	n	57	----	----	n	n	107	----	----	n	n
8	06/11/1988	1,200	n	n	58	----	----	n	n	108	----	----	n	n
9	05/31/1989	409	n	n	59	----	----	n	n	109	----	----	n	n
10	06/12/1990	1,540	n	n	60	----	----	n	n	110	----	----	n	n
11	06/02/1991	2,950	n	n	61	----	----	n	n	111	----	----	n	n
12	06/24/1992	1,130	n	n	62	----	----	n	n	112	----	----	n	n
13	06/19/1993	2,410	n	n	63	----	----	n	n	113	----	----	n	n
14	06/01/1994	808	n	n	64	----	----	n	n	114	----	----	n	n
15	06/18/1995	3,720	n	n	65	----	----	n	n	115	----	----	n	n
16	06/16/1996	2,570	n	n	66	----	----	n	n	116	----	----	n	n
17	07/29/1997	4,410	n	n	67	----	----	n	n	117	----	----	n	n
18	06/04/1998	811	n	n	68	----	----	n	n	118	----	----	n	n
19	05/01/1999	7,200	n	n	69	----	----	n	n	119	----	----	n	n
20	05/17/2000	673	n	n	70	----	----	n	n	120	----	----	n	n
21	05/30/2001	521	n	n	71	----	----	n	n	121	----	----	n	n
22	05/31/2002	573	n	n	72	----	----	n	n	122	----	----	n	n
23	05/30/2003	1,190	n	n	73	----	----	n	n	123	----	----	n	n
24	06/18/2004	583	n	n	74	----	----	n	n	124	----	----	n	n
25	06/04/2005	1,390	n	n	75	----	----	n	n	125	----	----	n	n
26	10/31/2005	904	n	n	76	----	----	n	n	126	----	----	n	n
27	08/02/2007	1,010	n	n	77	----	----	n	n	127	----	----	n	n
28	----	----	n	n	78	----	----	n	n	128	----	----	n	n
29	----	----	n	n	79	----	----	n	n	129	----	----	n	n
30	----	----	n	n	80	----	----	n	n	130	----	----	n	n
31	----	----	n	n	81	----	----	n	n	131	----	----	n	n
32	----	----	n	n	82	----	----	n	n	132	----	----	n	n
33	----	----	n	n	83	----	----	n	n	133	----	----	n	n
34	----	----	n	n	84	----	----	n	n	134	----	----	n	n
35	----	----	n	n	85	----	----	n	n	135	----	----	n	n
36	----	----	n	n	86	----	----	n	n	136	----	----	n	n
37	----	----	n	n	87	----	----	n	n	137	----	----	n	n
38	----	----	n	n	88	----	----	n	n	138	----	----	n	n
39	----	----	n	n	89	----	----	n	n	139	----	----	n	n
40	----	----	n	n	90	----	----	n	n	140	----	----	n	n
41	----	----	n	n	91	----	----	n	n	141	----	----	n	n
42	----	----	n	n	92	----	----	n	n	142	----	----	n	n
43	----	----	n	n	93	----	----	n	n	143	----	----	n	n
44	----	----	n	n	94	----	----	n	n	144	----	----	n	n
45	----	----	n	n	95	----	----	n	n	145	----	----	n	n
46	----	----	n	n	96	----	----	n	n	146	----	----	n	n
47	----	----	n	n	97	----	----	n	n	147	----	----	n	n
48	----	----	n	n	98	----	----	n	n	148	----	----	n	n
49	----	----	n	n	99	----	----	n	n	149	----	----	n	n
50	----	----	n	n	100	----	----	n	n	150	----	----	n	n

Log-Pearson Frequency Analysis Spreadsheet, Version 2.3, 1/2005.

Project: *Boxelder Breach Studies*
Streamgage: *Cache la Poudre River above Boxelder*
Date: *2/3/2009* **Performed By:** *SEY*

Discharge-Frequency, with Gage Skew

Cache la Poudre River above Boxelder



Discharge-Frequency, with Weighted Generalized Skew

Cache la Poudre River above Boxelder