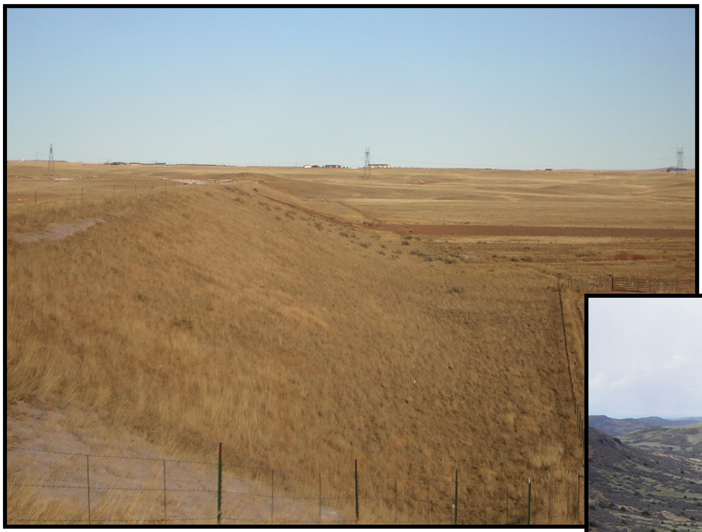
Boxelder B-2: Dam Breach Analysis

Larimer County, Colorado
March 2009



Boxelder B-2 embankment

Wellington

watershed

USDA Natural Resources Conservation Service
Colorado State Office

Steven E. Yochum, PE
Hydrologist
2150 Centre Ave
Bldg. A, Ste. 116
Fort Collins, CO 80526
970-295-5657

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

Lakewood, Colorado

March 3, 2009

BOXELDER B-2: DAM BREACH ANALYSIS

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

Summary: Predictions have been made of the likely extent and timing of flood flow resulting from a catastrophic breach of the Boxelder B-2 flood retention structure. This report details the dam breach analysis performed on the reservoir for the purpose of reevaluating the hazard classification and for use in an emergency action plan.

PREPARED BY: ________________________________ **DATE:** __________

STEVEN E. YOCHUM, PE
Hydrologist
970-295-5657, steven.yochum@co.usda.gov

CONCURRED: ________________________________ **DATE:** __________

JOHN ANDREWS, PE
Colorado State Conservation Engineer
720-544-2834, john.andrews@co.usda.gov

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INTRODUCTION

This report details the methods and results of a dam breach analysis performed on the Boxelder B-2 Dam of Larimer County, Colorado. This analysis was performed primarily to update 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-2 dam (NID ID: CO01406) is an earthen-embankment, typically-dry, flood retention structure. The structure is located on Boxelder Creek at approximately 5570 feet, 8.5 miles upstream of Wellington. This is a dams-in-series situation – Boxelder watershed structures B-5 and B-6 are located upstream (Figure 1). This structure and the companion structures upstream provide substantial flood-reduction benefits to the communities downstream, most notably Wellington.

Average precipitation within the reservoir's 108.5 square mile watershed varies from 15 to 17 inches, according to PRISM. Reservoirs B-5 and B-6 have watershed areas of 18.9 and 15.0 square miles, respectively. The B-2 embankment has a maximum height of about 59.4 feet, with a crest elevation of 5574.4 feet, original ground elevation of 5515 feet and embankment length of 7500 feet. The maximum storage, with the water surface elevation at the crest of the embankment, is 12,000 acre-feet. The emergency spillway is a 250 foot wide concrete chute located on the embankment. At the emergency spillway crest elevation of 5564.35 feet the associated reservoir storage is 6500 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 cross-section surveying of the most populated portion of the floodway, the town of 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 the I-25 crossings over the Boxelder and especially 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 APPENDIXES A and B. Valley cross-sections in the vicinity of Wellington are provided in APPENDIX C.**

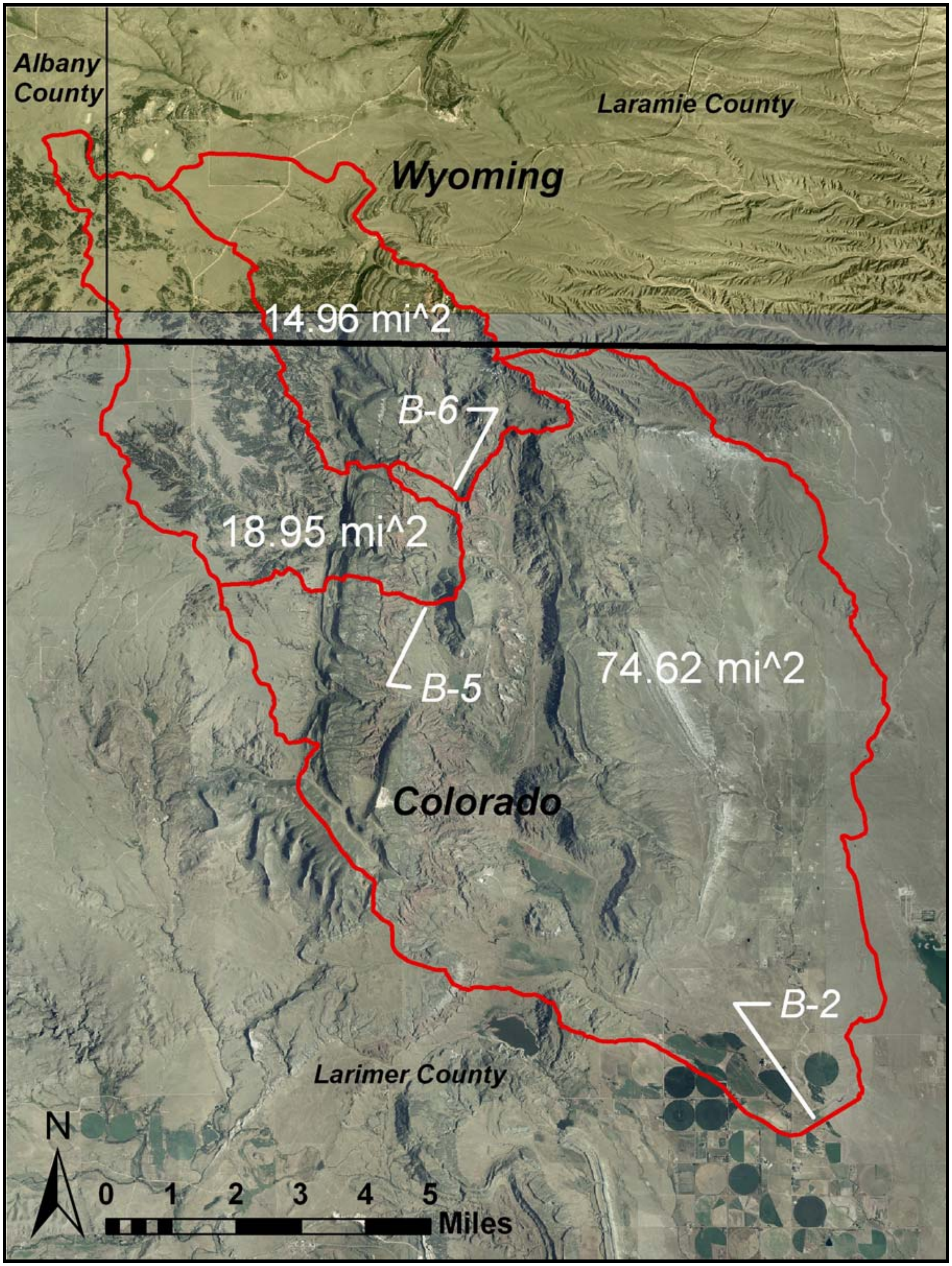

Figure 1: Boxelder B-2, B-5 and B-6 reservoir watersheds.

BREACH HYDROGRAPH DEVELOPMENT

As mentioned in Froehlich (1995), 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 (5564.35 feet, breach volume = 6500 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-2 structure are provided in Figures 2 through 4. As illustrated in Figures 2 and 3, the upstream and downstream faces are not armored by rock and are 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 resulting enhanced erosion around the grass clumps.

Figure 2: Upstream face of B-2 embankment.

Figure 3: Downstream face of B-2 embankment

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Figure 4: B-2 embankment. Aerial photography taken summer of 2005.

The methods used to develop possible hydrograph characteristics are 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; breach formation time using Froehlich, MacDonald and Langridge-Monoplolis, U.S. Bureau of Reclamation, and Von Thun and Gillette; and Fread's physically-based BREACH model. 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 (5564.35 feet).

	(cfs)	Time (hours)	(acre-feet)
NRCS peak flow	54,900	----	6500
Froehlich (1995) peak flow	67,000	----	6500
Kirkpatrick (1977) peak flow	41,000	----	6500
U.S. Bureau of Reclamation (1982) peak flow	101,000	----	6500
MacDonald and Langridge-Monoplolis (1984) peak flow	285,000	----	6500
Evan (1986) peak flow	116,000	----	6500
Fread BREACH	27,300	2.73	6680
breach geometry prediction (in HEC-RAS)	83,700	1.00	6580
Median:	**75,000**		

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

$$Q_{\max} = 1100 B_r^{1.35} \quad (1)$$

where

$$B_r = \frac{V_s H_w}{A} \quad (2)$$

But the peak flow is not to be less than

$$Q_{\max} = 3.2 H_w^{2.5} \quad (3)$$

and need not exceed

$$Q_{\max} = 65 H_w^{1.85} \quad (4)$$

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). With H_w = 49.4 feet, V_s = 6500 acre-feet and A = 226,000 ft^2, the peak discharge is 1770 cfs (bias low due to large embankment length), should not be less than 54,900 cfs but not in excess of 88,400 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.)

$$Q_p = 0.607 V_w^{0.295} H_b^{1.24} \quad (5)$$

where V_w is the reservoir volume at time of failure (8,018,000 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 (15.0 m). Using this equation, a peak discharge of 67,000 cfs (1898 cms) is estimated.

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

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

Using this equation, a peak discharge of 41,000 cfs (1160 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 101,000 cfs (2860 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 285,000 cfs (8060 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 116,000 cfs (3280 cms) is estimated.

There was a substantial range in the breach peak flow time estimates, from 41,000 to 285,000 cfs. The average and median values were 111,000 and 84,000 cfs, respectively.

BREACH Model

As discussed in Fread (1988), BREACH is a physically-based deterministic computational model that simulates breach characteristics (geometry, formation time) and the resulting hydrograph from a breached embankment dam. The model uses the sediment transport capacity of unsteady, uniform (not rapidly-varying) flow, using a modified (for steep channels) Meyer-Peter Muller methodology.

The analysis was preformed using the BOSS BREACH version 1.1 software. The failure water surface elevation was assumed to be at the crest of the emergency spillway with a piping failure initiated at the downstream toe at the maximum cross section. Dam dimensions, spillway rating curve and reservoir pool volume were taken from as-built drawings. Geotechnical parameters were assumed using typical values published in Design of Small Dams (USBR 1987) for the classification of the soil materials used to build the Boxelder B-2 embankment. Bare earth cover on the embankment was assumed.

Results of this analysis provided a peak flow of 27,300 cfs with a time to peak of 2.73 hours. This time to peak is longer than breach formation times of 21 of the 23 embankment failures documented in Froehlich (2008). Typically, actual breach formation times are typically between 0.5 and 1.5 hours. For example, the Big Bay embankment failure of 2004 (14,200 ac-ft) failed through piping, with a breach formation time of 0.92 hours (Yochum et al. 2008). It is often the case that breach formation time corresponds to time to peak, but for relatively small structures the flow may peak before a full breach is obtained (Wahl 2004). Hence, the most-appropriate time to peak value may be actually shorter than the times documented in Froehlich (2008).

This method provides substantially lower peak flow estimates than the NRCS and Froehlich peak flow estimates, due to the geometry of the embankment and material characteristics of the embankment. The embankment width is a sensitive parameter in the BREACH model however Froehlich (2008) found that embankment width was not a significant variable in the prediction of average breach width from actual dam failures. (Material composition was not included in the regression, due to limited data availability). The dataset of actual failures is limited but this was an interesting finding nonetheless.

While this model does provide an estimate of the failure characteristics and resulting floodwave using the principles of erosion, hydraulics and sediment transport, the results of such a deterministic analyses should not be considered the most accurate prediction of an embankment failure. BREACH uses a modified Meyer-Peter Muller sediment transport model. This model is dated, with newer methods considered more accurate. No matter what the model, however, it should always be remembered that any sediment transport model is lucky to predict transport within an order of magnitude of reality. This is especially a concern in the extreme case of a reservoir embankment failure, a scenario that is a substantial extrapolation from the conditions used to derive the original Meyer-Peter Muller sediment transport model. Such lack of accuracy is why the BREACH model, like any method available at this time for predicting a breach flow hydrograph from an embankment failure, should be compared to a number of other methods to check for consistency in results. In cases where any method is an outlier, the results should be viewed with suspicion.

Breach Formation Time

A breach formation time estimate was developed using a number of methods, as documented in Wahl (2004). A summary of results are provided in Table 2. It is not the case that these equations are 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 (5564.35 feet).

Method	Formation Time (hours)
BREACH model (Fread 1988)	2.73
Froehlich (1995b)	0.99
MacDonald and Langridge-Monoplolis (1984)	0.87
U.S. Bureau of Reclamation (1988)	0.50
Von Thun and Gillette (A) highly erodible (1990)	0.23
Von Thun and Gillette (A) erosion resistant (1990)	0.55
Von Thun and Gillette (B) highly erodible (1990)	0.66
Von Thun and Gillette (B) erosion resistant (1990)	1.33
Median:	**0.77**

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 = 8,018,000 m^3 and H_b = 15.1 m, the breach formation time is estimated to be 0.99 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 a reservoir volume of 8,018,000 m^3 and depth of water of 15.1 m, V_{er} is 42,904 and the breach formation time is 0.87 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

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

This method predicts an average breach width of 45.3 meters (149 ft) and formation time of 0.50 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 (11) is for highly-erodible materials and equation (12) is for erosion-resistant embankment materials. This method predicts the formation time as 0.23 and 0.55 hours.

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

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

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

where equation (14) is for highly-erodible materials and equation (13) 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 42.7 in this circumstance (Wahl 1998).

This method predicts a average breach width of 80.5 meters (264 ft) and the formation time as 0.66 and 1.33 hours.

There was a substantial range in the breach formation time estimates, from 0.21 hours to 2.73 hours. The median value was 0.76 hours. Shorter breach formation time estimates can lead greater unsteady flow model instability – a less conservative 1.0 hour breach formation time was 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 (5564.35 feet).

Method	Average Breach Width (hours)
BREACH model (Fread 1988)	26.3
Froehlich (1995b)	160
U.S. Bureau of Reclamation (1988)	149
Von Thun and Gillette (1990)	264
Median:	**155**

The BREACH model computes an unusually-low breach width of 26.3 feet. This breach width is less than all but 4 of the 70 breaches documented in Froehlich (2008). For 3 of the 4 actual failures with narrower breach widths, the dams had substantially less breach heights (12 to 20 feet, as opposed to 49.4 feet for Boxelder B-2). One actual failure (Lambert Lake, TN) had similar characteristics as Boxelder B-2 BREACH analysis: 25 feet average breach width with a breach height of 46.9 feet.

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 8,018,000 m^3 and depth of water of 14.1 m, this method predicts an average breach width of 48.9 m (160 feet).

As developed from Equation (14), the U.S. Bureau of Reclamation (1988) predicts an average breach width of 45.3 meters (149 ft).

Von Thun and Gillette (1990) provides average breach width from equation 19. This method predicts a average breach width of 77.9 meters (256 feet).

There was a substantial range in the average breach width estimates, from 26.3 to 256 feet. The median value was 155 feet. A breach hydrograph was developed for a scenario with a 155 feet wide average breach width, side slopes of 0.9 and formation time of 1.0 hour.

The breach geometry and formation time were inputted into HEC-RAS unsteady and the resulting hydrograph was developed assuming a sine wave progression (Brunner 2006).

Given this breach geometry and formation time, the model simulates a breach hydrograph with a peak at 83,700 cfs (at the embankment: station 500,000).

Selected Breach Hydrograph

Many potential breach hydrographs can be computed from the results of the numerous equations and methods summarized above. At first glace, the results of the only deterministic method, the BREACH model, is appealing. However, due to the assumed soil properties, lack of conservativeness, substantial expected inaccuracies in the Meyer-Peter Muller sediment transport model, and the assumption of uniform instead of rapidly-varying flow, the results of the BREACH analysis is not considered the most likely or appropriate method. But of the many statistical prediction equations covered, what combination of peak and formation time or breach geometry and formation time should be used?

Reviewing the data, there is a very wide range of peak flow predicted using the various methodologies, from 27,300 cfs (BREACH) to 285,000 cfs (MacDonald and Langridge-Monoplolis peak flow). However, selection of the most appropriate peak flow becomes more clear when comparing the median peak flow from the prediction equations (84,000 cfs) with the results of the HEC-RAS breach simulation using the median average breach width and the chosen breach formation time (83,900 cfs). The results using these independent methodologies are surprisingly similar. Due to this similarity and the reasonableness of the result, it was decided to use the breach hydrograph as developed by the HEC-RAS breach simulation, with a peak of 83,900 cfs, formation time of 1.0 hours, and volume of 6580 acre-feet. This is a lower peak flow at Boxelder B-2 than the 112,000 cfs used in a previous analysis (SCS 1980).

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.

This is not the first analysis of a breach hydrograph released from the Boxelder B-2 embankment – an analysis was performed using TR-66 before the structure was built (SCS 1980). A key weakness with the TR-66 model is the tendency of the simplified Att-Kin routing methodology to overestimate attenuation (SCS 1985). As detailed in Yochum et al. (2008), an investigation into the actual failure of Big Bay reservoir in 2004, a comparison of multiple routing approaches (HEC-RAS, WinTR-20, TR-66) with measured high water marks indicated that TR-66 over-attenuated this particular floodwave, providing unreliable results other than within the first couple miles of analysis, agreeing with the previously-documented known limitations (SCS 1985).

Computation Methodology

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 and Goodwell, 2002), 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 and Goodwell, 2002). In differential form, it is:

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

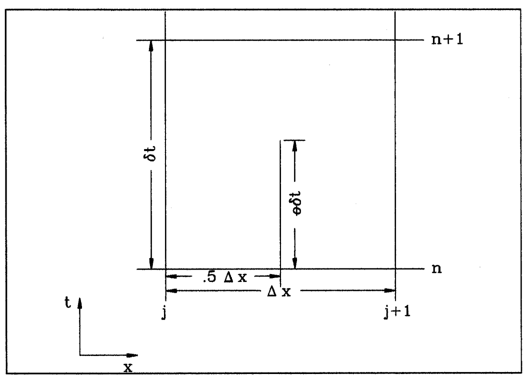
$$S_f = \frac{Q|Q|n^2}{2.208R^{4/3}A^2}$$

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 and Goodwell, 2002). 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: $\frac{\partial f}{\partial t} \approx \frac{\Delta f}{\Delta t} = \frac{0.5(\Delta f_{j+1} + \Delta f_j)}{\Delta t}$

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

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:

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

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 and Goodwell 2006).

For a more comprehensive presentation of the solution equations and techniques used in HEC-RAS, please see the HEC-RAS Hydraulic Reference Manual (Brunner and Goodwell, 2002).

Boxelder B-2 HEC-RAS Model

Using sections developed in HEC-GeoRAS, an ArcGIS extension, and geometry developed from both a 10-meter DEM (based in 7.5-minutes USGS quadrangles) and supplemental surveyed cross sections in the vicinity of Wellington, an unsteady flow model was developed from the Boxelder B-2 structure to the Cache la Poudre River. After troubleshooting the model by varying the time step, cross section spacing (through interpolation), theta, water surface calculation tolerance, and simplifying the geometry from the surveyed cross sections, a relatively stable breach model was developed that provided reasonable estimates of peak discharge and water surface elevations. The model still has a few relatively-insignificant points of instability, where results are a bit unusual, but these points are isolated and do not produce outliers -- they do not appear to significantly impact the results of the model.

The model assumes a piping failure, with an initial water surface at the crest of the emergency spillway (5564.35 feet). The road crossing at Washington Avenue (dual culverts), Cleveland Avenue (bridge) and Jefferson Avenue (bridge) in Wellington 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 = 0.0029) and an initial flow of 2000 cfs was assumed at all sections.

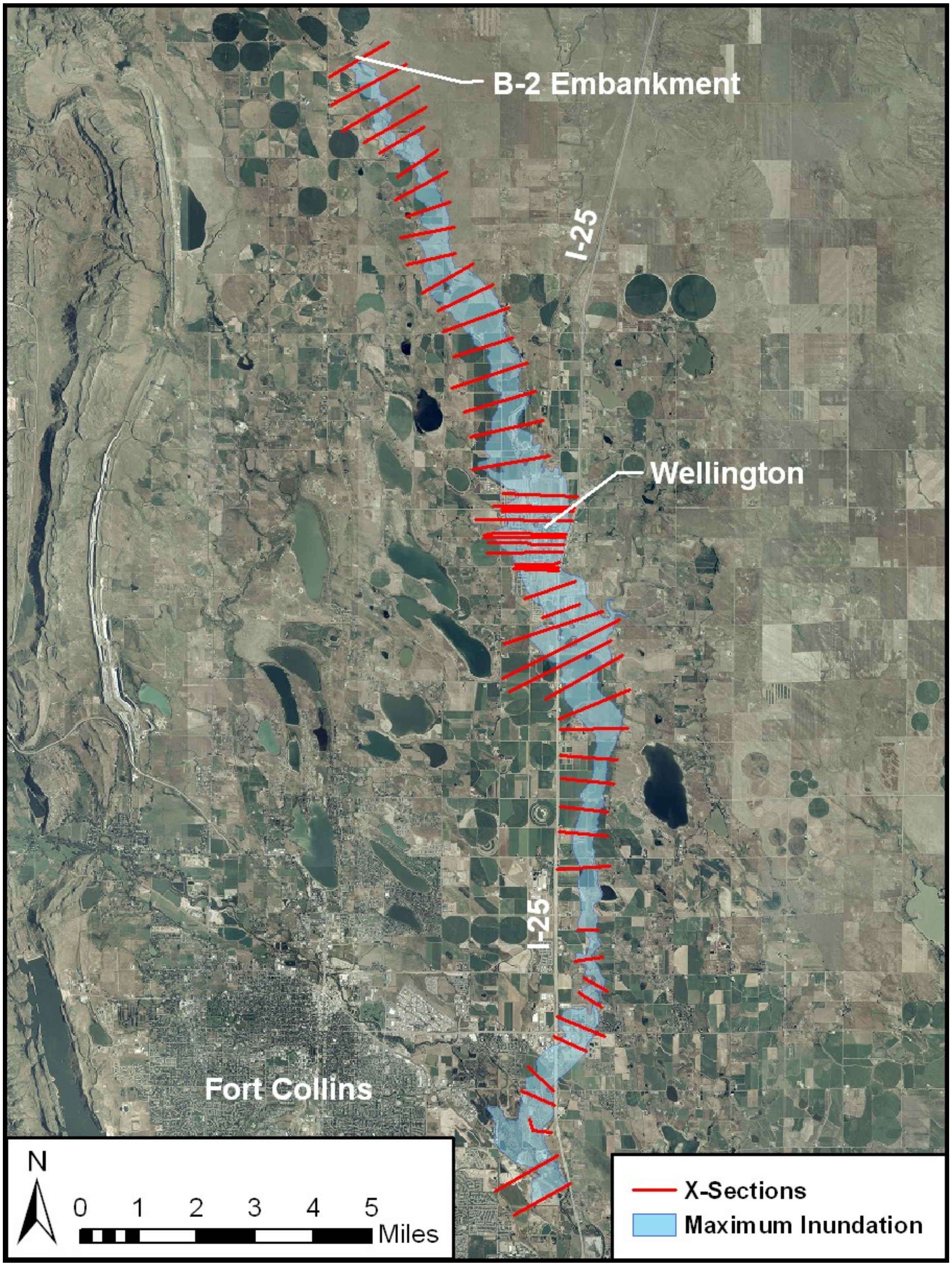


Figure 5: Inundation extent to the Cache la Poudre River, Boxelder B-2 breach analysis.

INUNDATION EXTENT AND TIMING

This analysis provides a prediction of the extent and timing of flooding from a catastrophic breach of the Boxelder B-2 dam embankment. The extent of the expected inundation is shown in Figure 5. 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.

Starting with a peak flow of 83,000 cfs at B-2, the flow attenuates to 75,000 cfs at the northern limit of Wellington, 72,000 cfs at the southern limit of Wellington, 56,000 cfs in the Eastern suburbs of Fort Collins (I-25 and Mulberry Street), and 52,000 cfs at the Poudre River. Based upon 1980 to 2007 data in the Poudre River at the Boxelder, this discharge corresponds to about 4.5-times the 100-year flood event of 11,200 cfs. (See frequency analysis in Appendix D).

Table 4 provides the model results at each non-interpolated cross section. Figure 6 illustrates the routed breach hydrographs at 6 points within the analysis extent. The extent of inundation with expected depth*velocity products greater than 7 (shown in Appendix A) 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**.

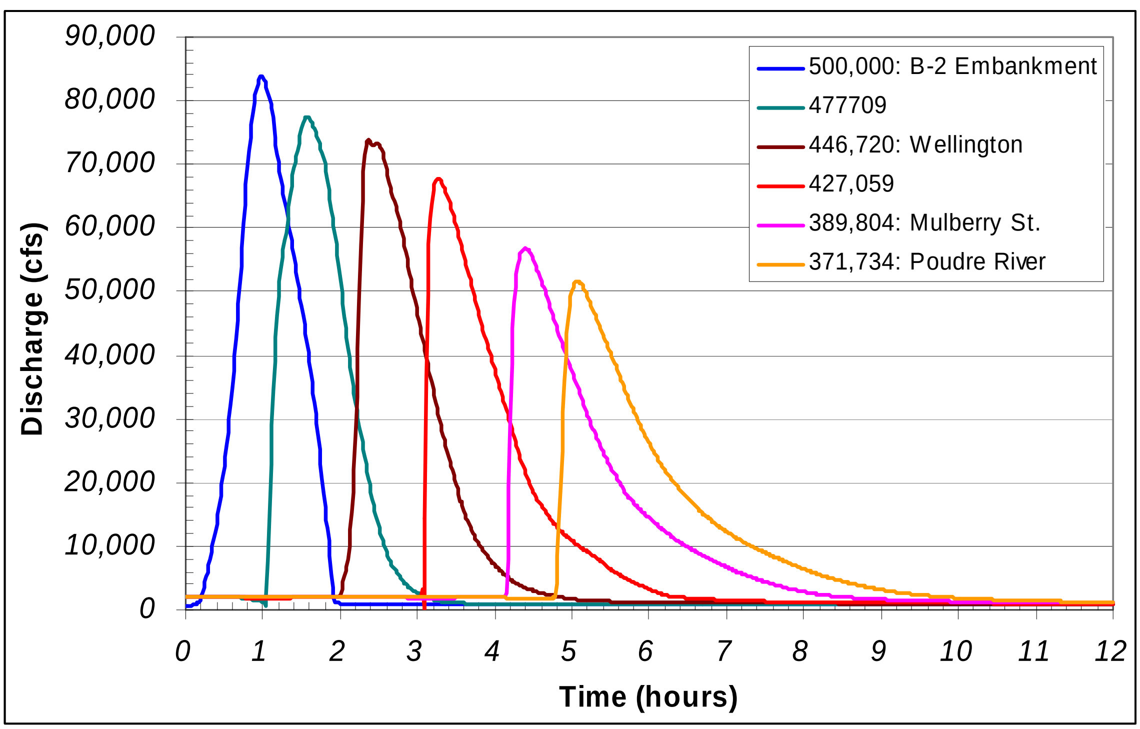


Figure 6: Breach hydrographs.

Table 4: Breach analysis results at maximum water surface elevation, Boxelder B-2 Breach analysis.

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Energy Grade Slope (ft/ft)	Velocity Channel (ft/s)	Velocity Left (ft/s)	Velocity Right (ft/s)	Froude Number Channel	Froude Number X-Sect.
500000	83,000	5532.06	0.00245	10.8	1.0	5.0	0.56	0.65
497436	79,400	5514.78	0.00641	14.2	4.7	6.2	0.86	0.90
494549	79,200	5493.96	0.00516	8.7	12.2	1.3	0.53	0.80
492205	79,100	5480.42	0.00682	10.8	6.2	9.3	0.59	0.77
490697	78,900	5469.83	0.00557	10.4	8.0	5.9	0.56	0.69
487682	78,700	5444.00	0.00544	11.8	1.5	2.5	0.77	0.81
485669	78,600	5432.36	0.00505	12.5	6.7	6.5	0.76	0.79
482974	78,400	5418.63	0.00498	13.6	8.6	5.0	0.77	0.82
480425	77,900	5403.76	0.00412	12.3	6.7	6.6	0.70	0.80
477709	77,500	5384.96	0.00346	11.5	7.5	5.1	0.64	0.65
475666	77,200	5373.85	0.00597	8.8	6.8	6.8	0.74	0.73
472611	76,900	5355.12	0.00672	11.5	3.6	3.7	0.72	0.98
470081	76,700	5339.15	0.00522	8.1	6.9	6.2	0.53	0.64
467295	76,300	5323.45	0.00599	11.3	7.6	6.8	0.68	0.75
464427	76,100	5307.49	0.00748	7.8	4.1	7.6	0.60	0.75
459947	75,900	5287.90	0.00603	10.0	4.0	8.2	0.59	0.78
456307	75,700	5270.44	0.00463	10.3	6.8	5.4	0.58	0.68
452885	75,100	5252.73	0.01189	16.2	5.5	4.4	0.97	1.36
449555	79,000	5228.84	0.00499	7.2	5.8	9.5	0.50	0.69
448257	74,600	5226.24	0.00052	3.9	2.2	3.0	0.18	0.21
448146	74,500	5225.65	0.00419	8.8	4.4	4.7	0.49	0.54
448096				**Washington Avenue**				
448046	74,500	5223.90	0.03584	22.5	9.6	7.3	1.39	1.71
447756	74,200	5221.29	0.00619	8.4	5.2	2.6	0.58	0.51
446720	73,700	5215.46	0.00594	7.4	5.3	2.5	0.53	0.52
445224	70,700	5208.33	0.00291	6.0	3.0	3.8	0.41	0.37
444922	68,900	5207.37	0.00260	7.8	2.6	5.0	0.37	0.36
444872				**Cleveland Avenue**				
444822	73,100	5205.93	0.00944	14.1	3.9	7.9	0.70	0.62
444196	73,100	5200.81	0.00600	5.8	3.9	3.2	0.40	0.41
443457	73,100	5196.62	0.00657	7.9	4.7	3.2	0.57	0.48
442217	72,500	5191.43	0.00338	7.1	5.0	3.3	0.43	0.41
442023	72,500	5190.75	0.00214	7.0	6.0	3.4	0.33	0.46
441973				**Jefferson Avenue**				
441923	72,500	5187.02	0.04106	25.0	15.3	0.1	1.38	1.84
441735	72,500	5186.40	0.00445	7.6	7.4	2.6	0.52	0.64
439439	71,400	5176.57	0.00247	6.7	5.9	2.6	0.37	0.45
437144	69,300	5165.01	0.00916	12.4	7.0	4.3	0.72	0.77
435761	69,100	5154.57	0.00616	6.9	6.0	3.9	0.71	0.62
433682	68,800	5142.25	0.00380	8.2	5.3	3.2	0.61	0.59
432250	68,600	5135.67	0.00755	9.1	6.9	5.3	0.81	0.78
430414	68,400	5122.41	0.00574	9.0	6.6	5.9	0.73	0.72
427059	67,600	5106.04	0.00317	7.4	5.1	5.7	0.56	0.52
423410	65,900	5093.66	0.00330	7.5	4.6	5.4	0.53	0.54
420354	65,000	5076.75	0.00602	11.3	7.4	8.3	0.66	0.74
418041	64,500	5062.78	0.00363	11.2	8.2	5.7	0.65	0.65
415458	63,600	5051.50	0.00505	12.9	6.6	6.7	0.76	0.76
413179	63,000	5040.70	0.00524	9.3	7.2	4.7	0.55	0.57
409732	61,700	5021.75	0.00231	8.7	8.0	5.6	0.52	0.52
403799	59,200	5000.75	0.00610	13.2	7.3	8.6	0.82	0.78
400648	58,800	4983.73	0.00345	11.4	6.8	6.5	0.64	0.62
396353	58,200	4972.24	0.00721	10.9	6.9	10.2	0.64	0.76
393922	57,100	4956.95	0.00569	10.3	3.3	7.5	0.58	0.69
389804	56,800	4939.92	0.00491	6.9	4.5	3.9	0.47	0.44
387547	56,000	4931.33	0.00570	8.3	4.8	4.4	0.51	0.46
382443	55,000	4910.23	0.00581	11.5	7.2	7.0	0.78	0.76
380358	54,700	4899.86	0.00365	6.2	5.4	5.6	0.43	0.52
377351	54,200	4887.78	0.00654	8.4	3.9	7.8	0.58	0.70
374605	53,100	4870.54	0.00318	6.9	3.7	3.8	0.42	0.44
371734	51,600	4863.91	0.00291	5.5	2.0	3.9	0.43	0.47

The probable inundation extent and timing is provided on the inundation maps of Appendix A. Tables imbedded within these plots indicate peak discharge at each section, maximum depth and velocities, and breach wave timing and steepness at the provided sections. For Wellington, points with computed depth*velocity values are included, with a product of 7 being assumed as a threshold for endangering life. Maximum inundation depths are indicated in plots provided in Appendix B. Additionally, a few selected cross sections in the vicinity of Wellington, the community most threatened by a failure of Boxelder B-2, have been provided in Appendix C. These sections include the water surface elevation, structures and relevant hydraulic characteristics of the peak flow.

SUMMARY AND CONCULSIONS

A comprehensive approach was implemented to develop a most likely breach hydrograph of the Boxelder B-2 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; breach formation time using Froehlich, MacDonald and Langridge-Monoplolis, U.S. Bureau of Reclamation, and Von Thun and Gillette; and Fread's physically-based BREACH model. After reviewing the results of various methods that provided a wide range of peak flow values, the selection of a most likely breach hydrograph from a piping failure (with an initial water surface elevation at the crest of the emergency spillway) became clear when observing similarity in the results developed using independent methodologies. Through professional judgment, it was decided to use the breach hydrograph as developed by the HEC-RAS breach simulation, with a peak of 83,900 cfs, formation time of 1.0 hours, and volume of 6580 acre-feet. This is a lower peak flow at Boxelder B-2 than the 112,000 cfs used in a previous analysis (SCS 1980).

The breach hydrograph was routed using HEC-RAS 4.0 from the embankment to the confluence with the Poudre River, 20 miles downstream. According to the model, the flow attenuates to 75,000 cfs at the northern limit of Wellington, 72,000 cfs at the southern limit of Wellington, 56,000 cfs in the eastern suburbs of Fort Collins, and 52,000 cfs at the Poudre River. This final discharge corresponds to about 4.5-times the 100-year flood event of 11,200 cfs.

In 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-2 structure be increased from its current significant level to a high hazard.

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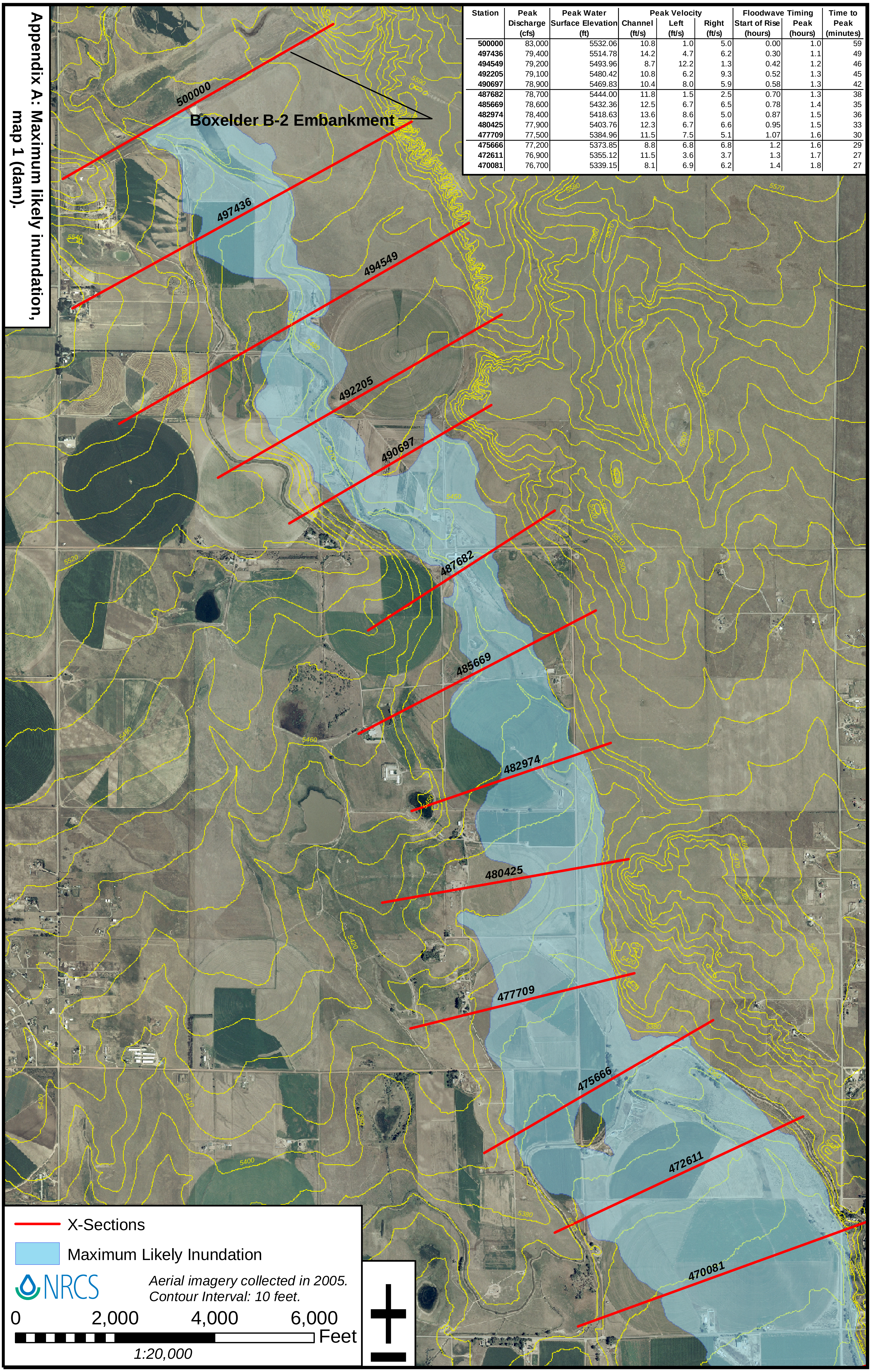
APPENDIX A: Maximum Likely Inundation

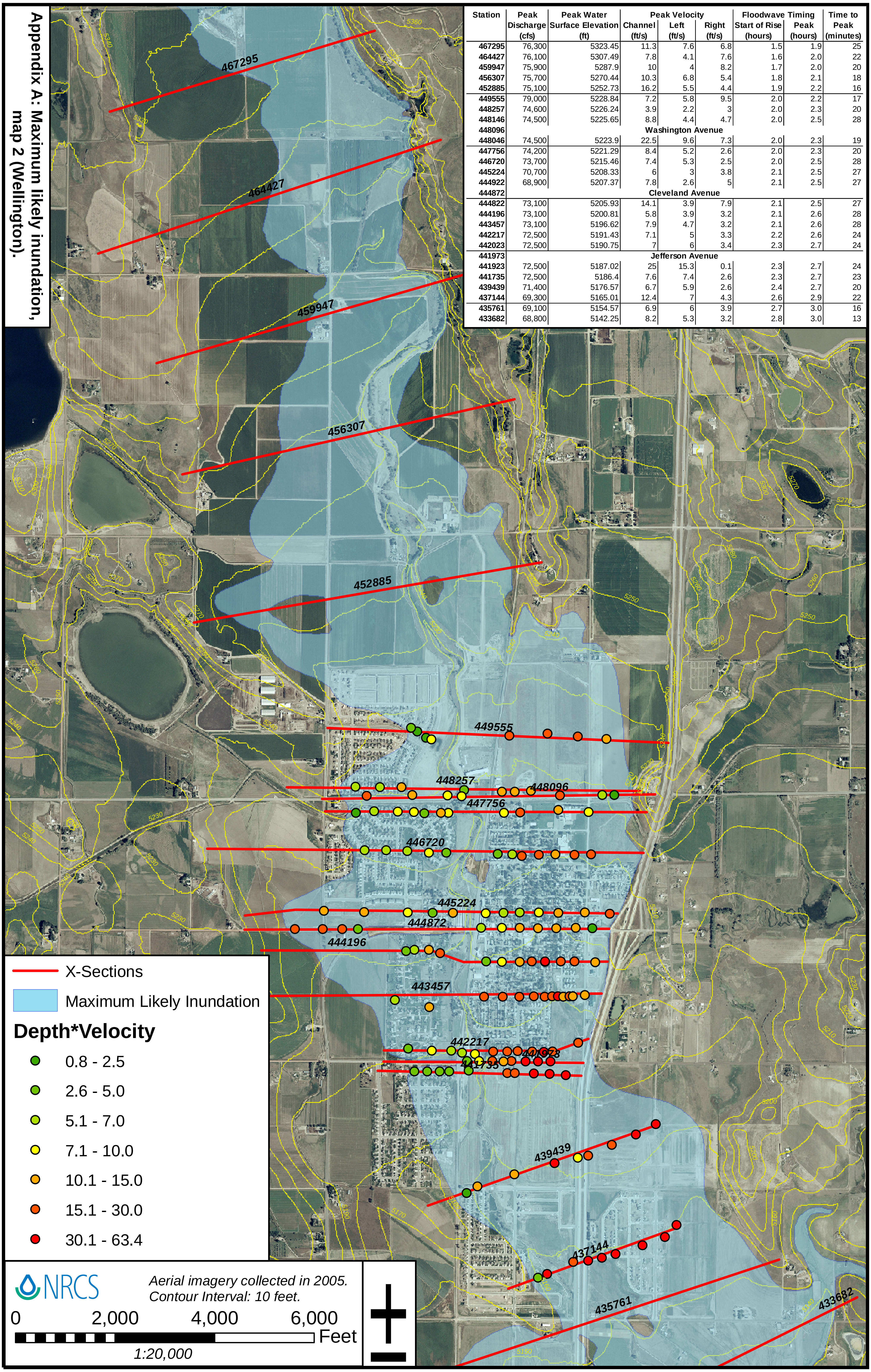
Appendix A: Maximum likely inundation, map 1 (dam).



Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodwave Timing Start of Rise (hours)	Floodwave Timing Peak (hours)	Time to Peak (minutes)
500000	83,000	5532.06	10.8	1.0	5.0	0.00	1.0	59
497436	79,400	5514.78	14.2	4.7	6.2	0.30	1.1	49
494549	79,200	5493.96	8.7	12.2	1.3	0.42	1.2	46
492205	79,100	5480.42	10.8	6.2	9.3	0.52	1.3	45
490697	78,900	5469.83	10.4	8.0	5.9	0.58	1.3	42
487682	78,700	5444.00	11.8	1.5	2.5	0.70	1.3	38
485669	78,600	5432.36	12.5	6.7	6.5	0.78	1.4	35
482974	78,400	5418.63	13.6	8.6	5.0	0.87	1.5	36
480425	77,900	5403.76	12.3	6.7	6.6	0.95	1.5	33
477709	77,500	5384.96	11.5	7.5	5.1	1.07	1.6	30
475666	77,200	5373.85	8.8	6.8	6.8	1.2	1.6	29
472611	76,900	5355.12	11.5	3.6	3.7	1.3	1.7	27
470081	76,700	5339.15	8.1	6.9	6.2	1.4	1.8	27

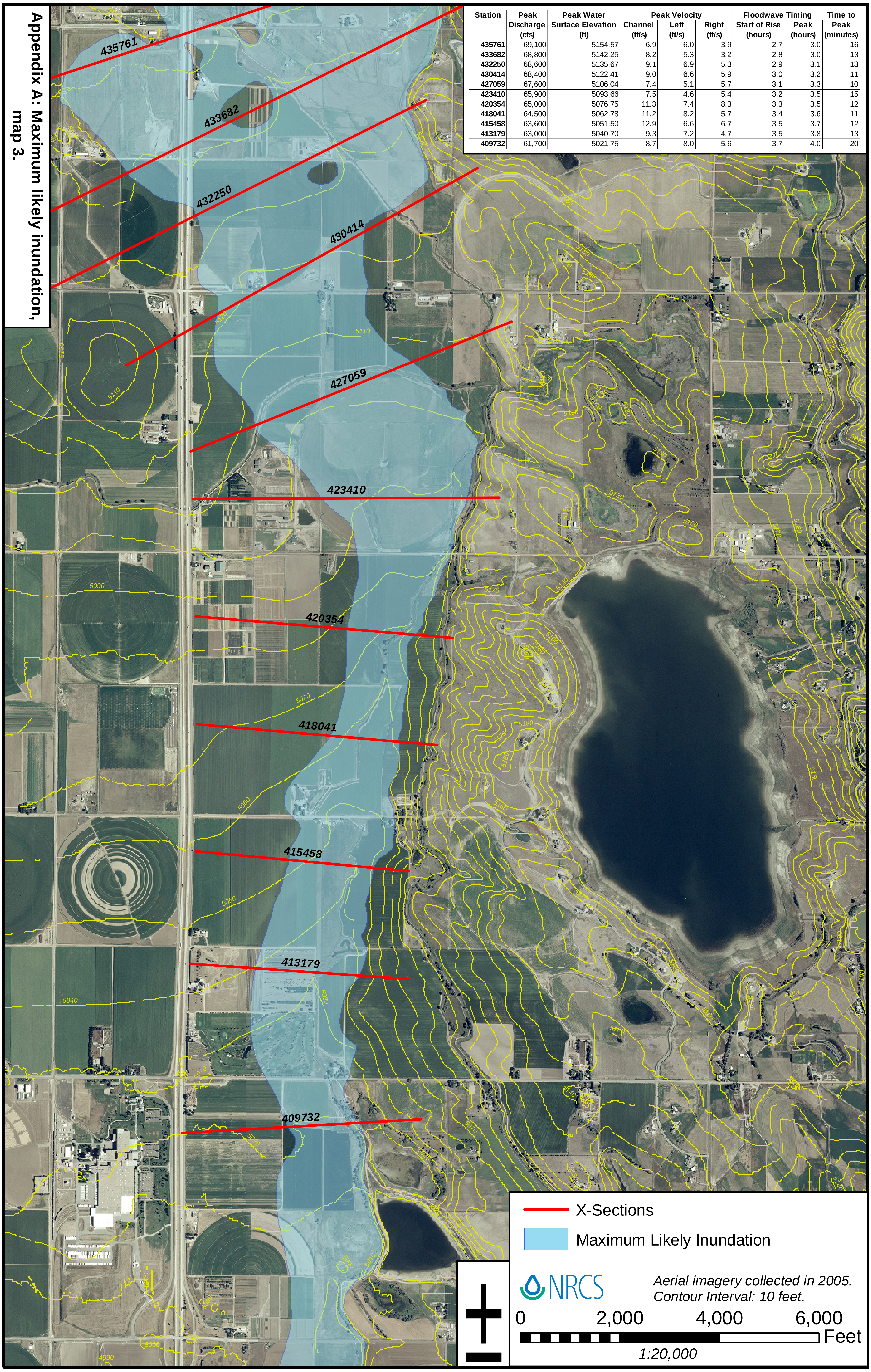
Appendix A: Maximum likely inundation, map 2 (Wellington).

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodwave Timing Start of Rise (hours)	Floodwave Timing Peak (hours)	Time to Peak (minutes)
467295	76,300	5323.45	11.3	7.6	6.8	1.5	1.9	25
464427	76,100	5307.49	7.8	4.1	7.6	1.6	2.0	22
459947	75,900	5287.9	10	4	8.2	1.7	2.0	20
456307	75,700	5270.44	10.3	6.8	5.4	1.8	2.1	18
452885	75,100	5252.73	16.2	5.5	4.4	1.9	2.2	16
449555	79,000	5228.84	7.2	5.8	9.5	2.0	2.2	17
448257	74,600	5226.24	3.9	2.2	3	2.0	2.3	20
448146	74,500	5225.65	8.8	4.4	4.7	2.0	2.5	28
448096	**Washington Avenue**							
448046	74,500	5223.9	22.5	9.6	7.3	2.0	2.3	19
447756	74,200	5221.29	8.4	5.2	2.6	2.0	2.3	20
446720	73,700	5215.46	7.4	5.3	2.5	2.0	2.5	28
445224	70,700	5208.33	6	3	3.8	2.1	2.5	27
444922	68,900	5207.37	7.8	2.6	5	2.1	2.5	27
444872	**Cleveland Avenue**							
444822	73,100	5205.93	14.1	3.9	7.9	2.1	2.5	27
444196	73,100	5200.81	5.8	3.9	3.2	2.1	2.6	28
443457	73,100	5196.62	7.9	4.7	3.2	2.1	2.6	28
442217	72,500	5191.43	7.1	5	3.3	2.2	2.6	24
442023	72,500	5190.75	7	6	3.4	2.3	2.7	24
441973	**Jefferson Avenue**							
441923	72,500	5187.02	25	15.3	0.1	2.3	2.7	24
441735	72,500	5186.4	7.6	7.4	2.6	2.3	2.7	23
439439	71,400	5176.57	6.7	5.9	2.6	2.4	2.7	20
437144	69,300	5165.01	12.4	7	4.3	2.6	2.9	22
435761	69,100	5154.57	6.9	6	3.9	2.7	3.0	16
433682	68,800	5142.25	8.2	5.3	3.2	2.8	3.0	13



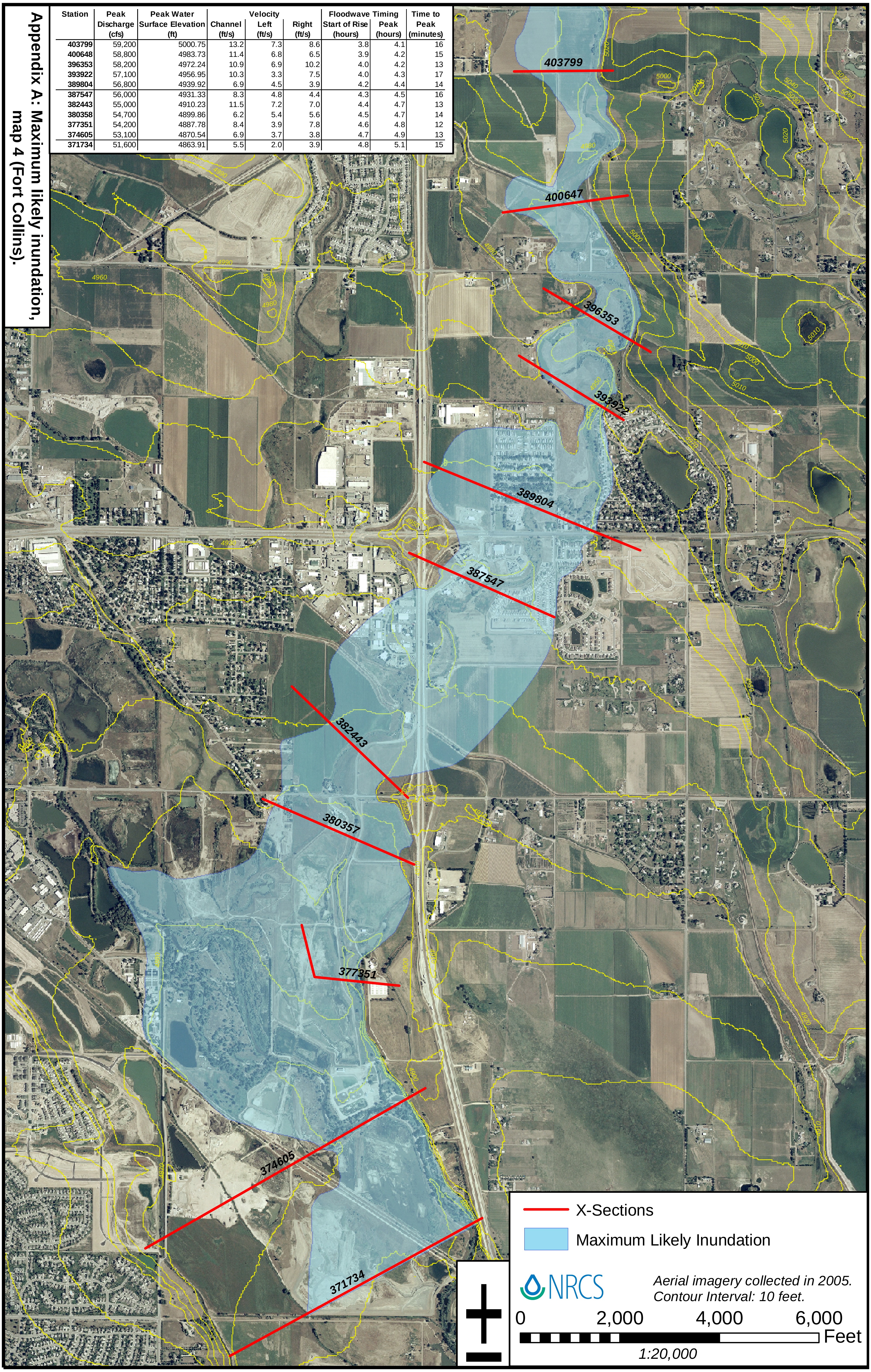
Appendix A: Maximum likely inundation, map 3.

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodwave Timing Start of Rise (hours)	Floodwave Timing Peak (hours)	Time to Peak (minutes)
435761	69,100	5154.57	6.9	6.0	3.9	2.7	3.0	16
433682	68,800	5142.25	8.2	5.3	3.2	2.8	3.0	13
432250	68,600	5135.67	9.1	6.9	5.3	2.9	3.1	13
430414	68,400	5122.41	9.0	6.6	5.9	3.0	3.2	11
427059	67,600	5106.04	7.4	5.1	5.7	3.1	3.3	10
423410	65,900	5093.66	7.5	4.6	5.4	3.2	3.5	15
420354	65,000	5076.75	11.3	7.4	8.3	3.3	3.5	12
418041	64,500	5062.78	11.2	8.2	5.7	3.4	3.6	11
415458	63,600	5051.50	12.9	6.6	6.7	3.5	3.7	12
413179	63,000	5040.70	9.3	7.2	4.7	3.5	3.8	13
409732	61,700	5021.75	8.7	8.0	5.6	3.7	4.0	20

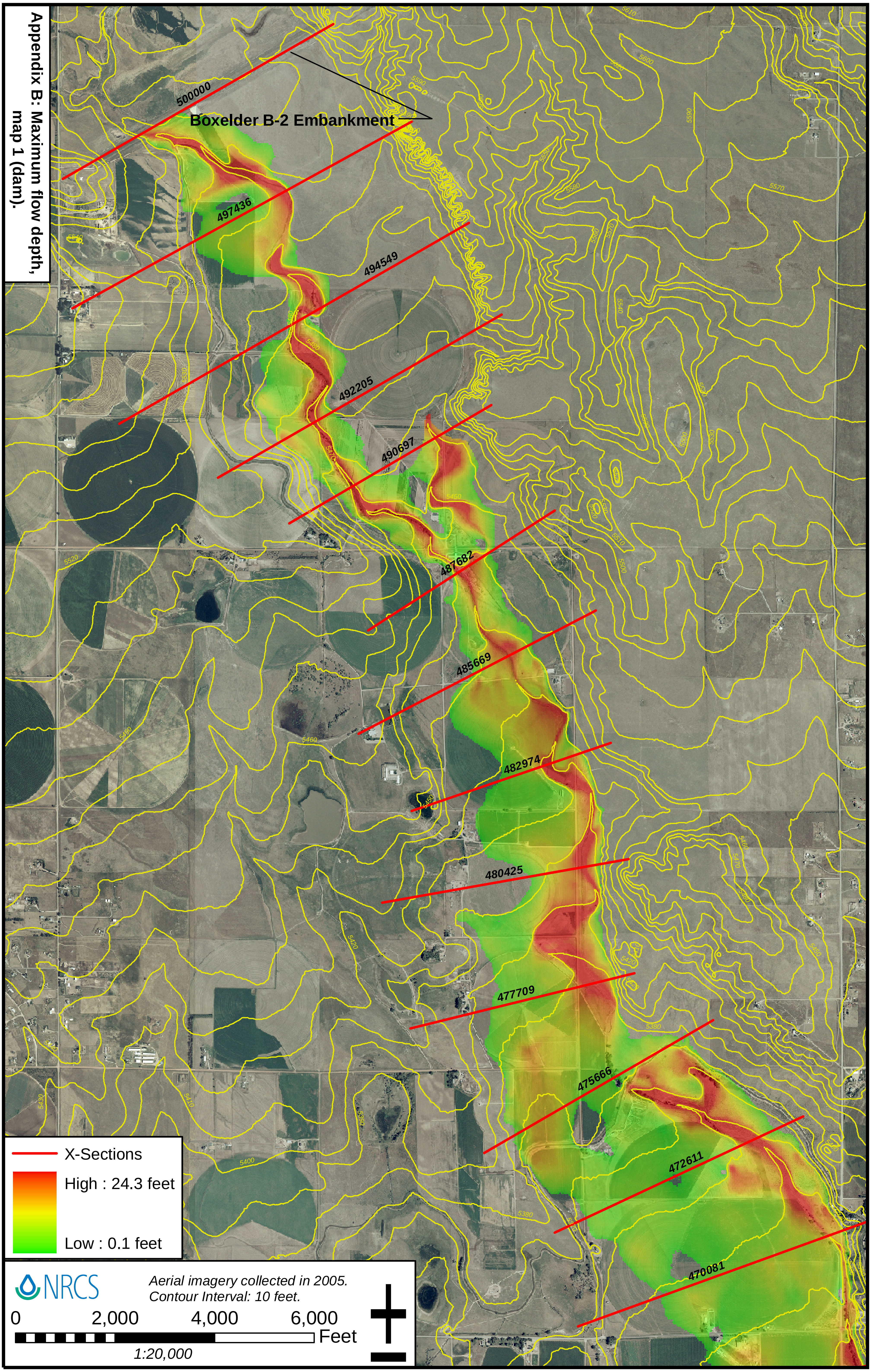


Appendix A: Maximum likely inundation, map 4 (Fort Collins).

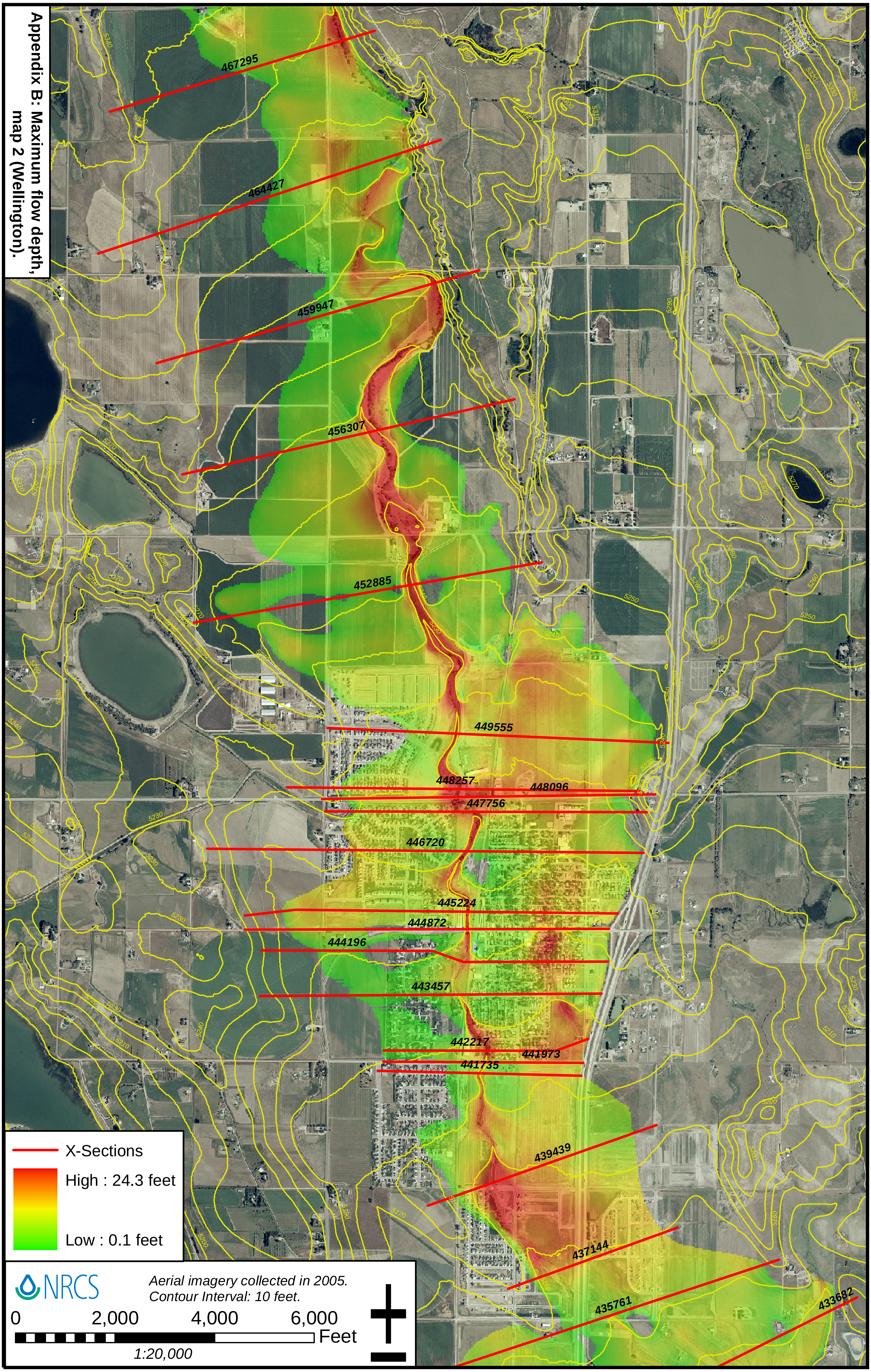
Station	Peak Discharge (cfs)	Peak Water Surface Elevation (ft)	Velocity Channel (ft/s)	Velocity Left (ft/s)	Velocity Right (ft/s)	Floodwave Timing Start of Rise (hours)	Floodwave Timing Peak (hours)	Time to Peak (minutes)
403799	59,200	5000.75	13.2	7.3	8.6	3.8	4.1	16
400648	58,800	4983.73	11.4	6.8	6.5	3.9	4.2	15
396353	58,200	4972.24	10.9	6.9	10.2	4.0	4.2	13
393922	57,100	4956.95	10.3	3.3	7.5	4.0	4.3	17
389804	56,800	4939.92	6.9	4.5	3.9	4.2	4.4	14
387547	56,000	4931.33	8.3	4.8	4.4	4.3	4.5	16
382443	55,000	4910.23	11.5	7.2	7.0	4.4	4.7	13
380358	54,700	4899.86	6.2	5.4	5.6	4.5	4.7	14
377351	54,200	4887.78	8.4	3.9	7.8	4.6	4.8	12
374605	53,100	4870.54	6.9	3.7	3.8	4.7	4.9	13
371734	51,600	4863.91	5.5	2.0	3.9	4.8	5.1	15



APPENDIX B: Maximum Flow Depth

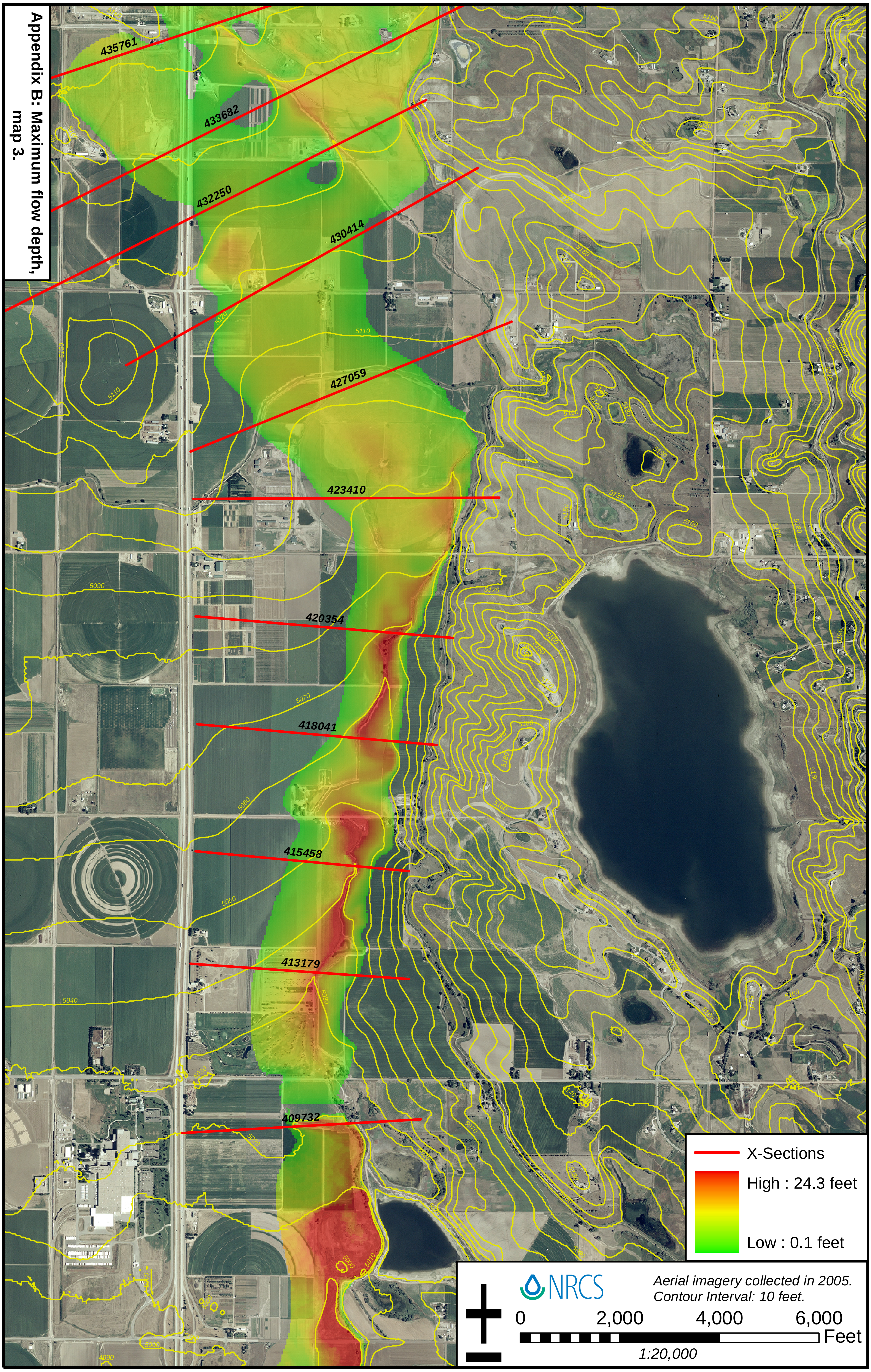


Appendix B: Maximum flow depth, map 1 (dam).

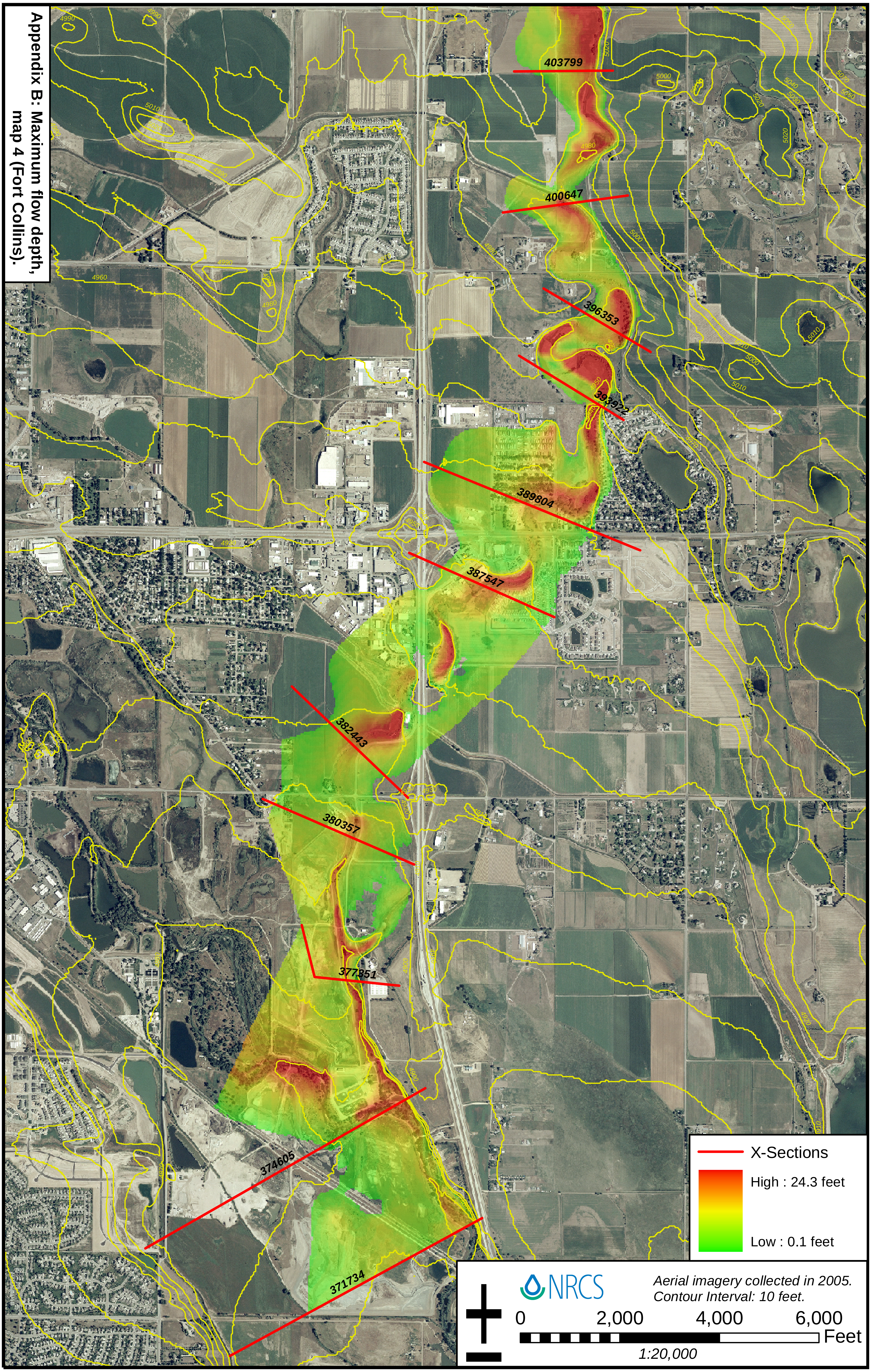


Appendix B: Maximum flow depth, map 2 (Wellington).
467295
464427
459947
456307
452885
449555
448257
448096
447756
446720
445224
444872
444196
443457
442217
441973
441735
439439
437144
435761
433682
X-Sections
High : 24.3 feet
Low : 0.1 feet
NRCS
Aerial imagery collected in 2005.
Contour Interval: 10 feet.
0
2,000
4,000
6,000
Feet
1:20,000

Appendix B: Maximum flow depth, map 3.



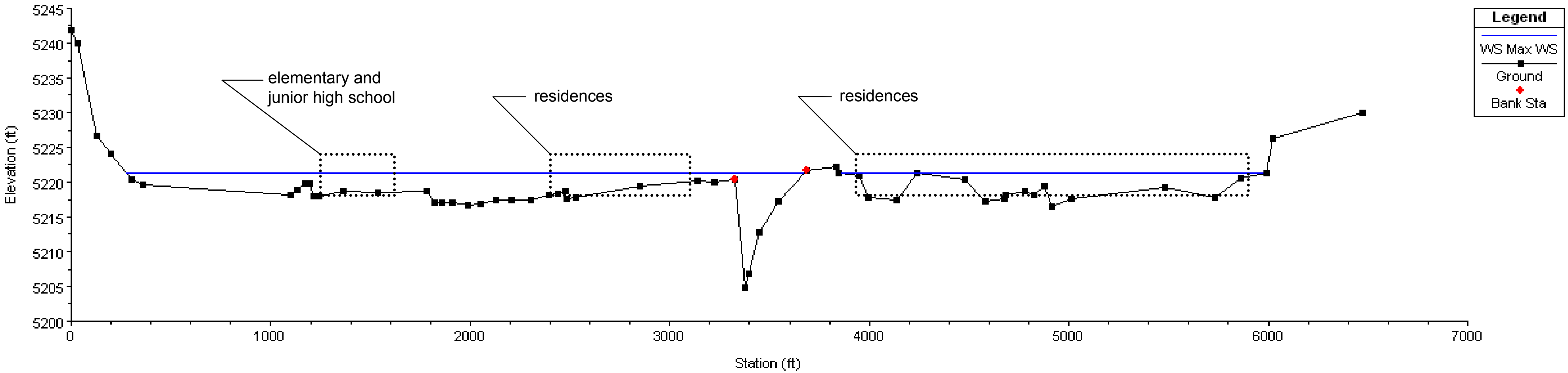
Appendix B: Maximum flow depth, map 4 (Fort Collins).



APPENDIX C: Valley Cross Sections

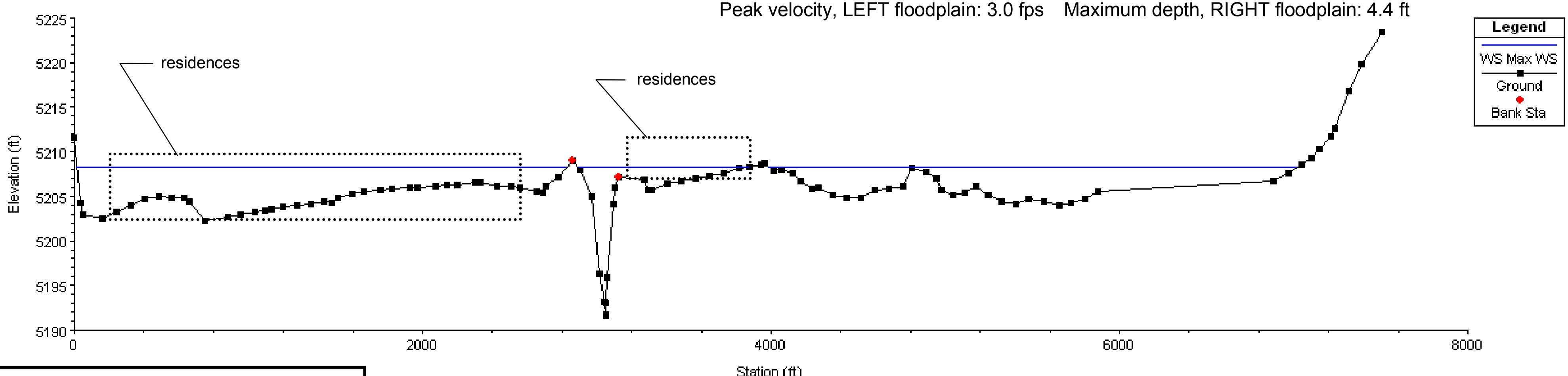
Section 447756

Peak Discharge: 74,200 cfs
Peak WSEL: 5221.29 ft
Peak velocity, channel: 8.4 fps
Peak velocity, LEFT floodplain: 5.2 fps
Peak velocity, RIGHT floodplain: 2.6 fps
Maximum depth, channel: 16.5 ft
Maximum depth, LEFT floodplain: 4.7 ft
Maximum depth, RIGHT floodplain: 4.9 ft



Section 445224

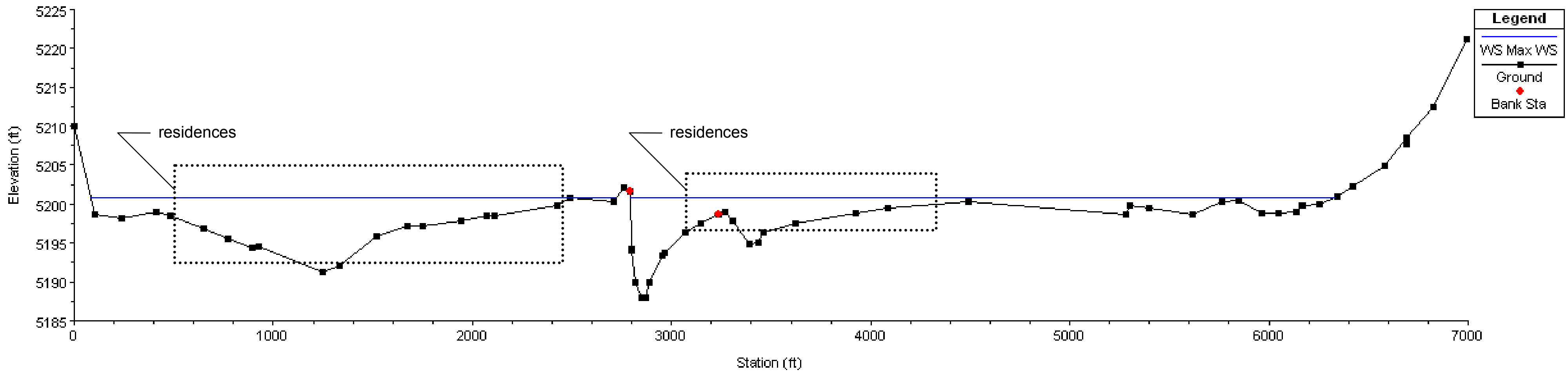
Peak Discharge: 70,700 cfs
Peak WSEL: 5208.33 ft
Peak velocity, channel: 6.0 fps
Peak velocity, LEFT floodplain: 3.0 fps
Peak velocity, RIGHT floodplain: 3.8 fps
Maximum depth, channel: 16.8 ft
Maximum depth, LEFT floodplain: 6.0 ft
Maximum depth, RIGHT floodplain: 4.4 ft



Appendix C: Valley cross sections in Wellington, upstream.

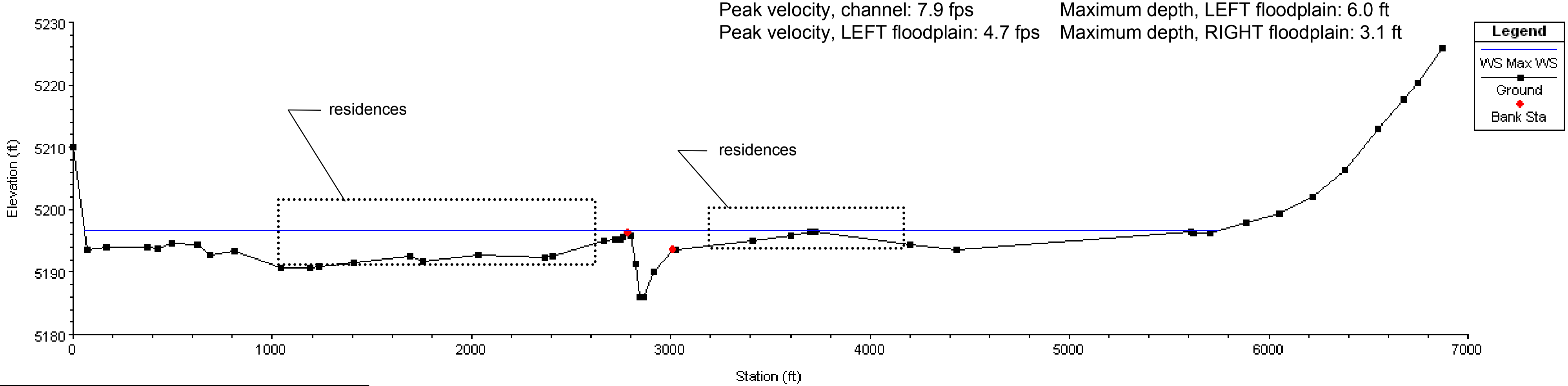
Section 444196

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 443457

Peak Discharge: 73,100 cfs
Peak WSEL: 5196.62 ft
Peak velocity, channel: 7.9 fps
Peak velocity, LEFT floodplain: 4.7 fps
Peak velocity, RIGHT floodplain: 3.2 fps
Maximum depth, channel: 10.6 ft
Maximum depth, LEFT floodplain: 6.0 ft
Maximum depth, RIGHT floodplain: 3.1 ft



Appendix C: Valley cross sections in Wellington, downstream.

APPENDIX D: 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

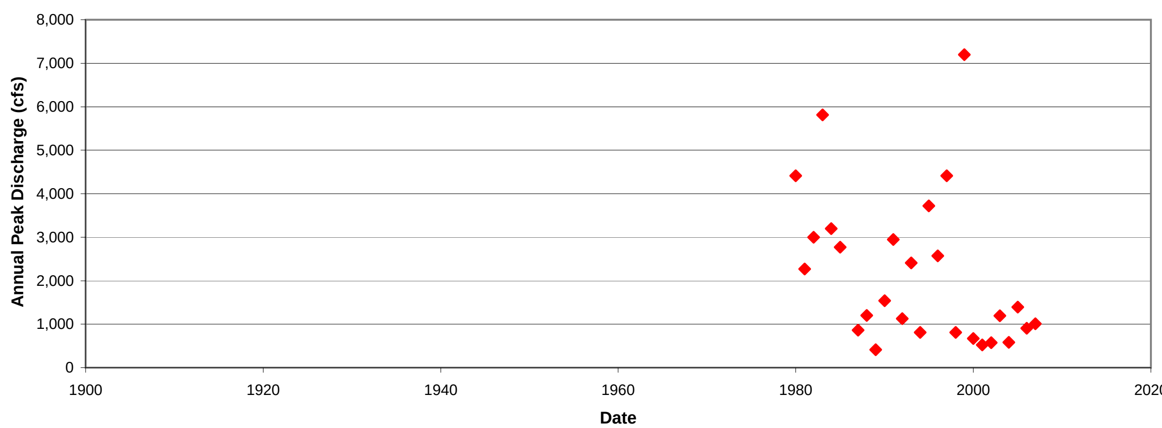
Generalized Skew Coefficient[3]:
Variance of Generalized Skew[3]:
A: -0.319127
B: 0.904663
station skew: 0.135910
MSE Station Skew: 0.19526925
Weighted skew coefficient[1]: 0

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	----	----	----

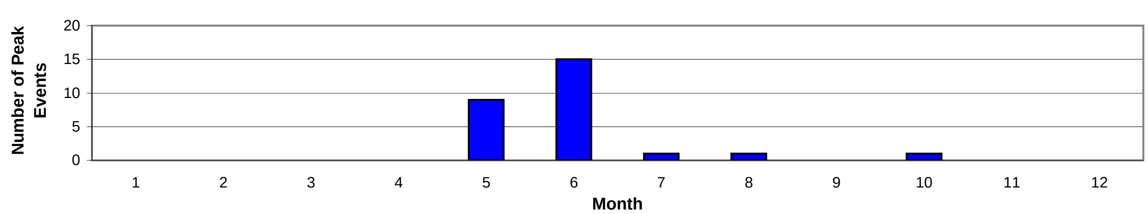
(1) Station and generalized skews must be between -2.00 and +3.00 in this spreadsheet.
(2) Considering the relatively short length of most gage records, less frequent peak estimates need to be used with considerable care.
(3) Computed one of four ways (see "generalized skew coefficient" worksheet): Mean 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

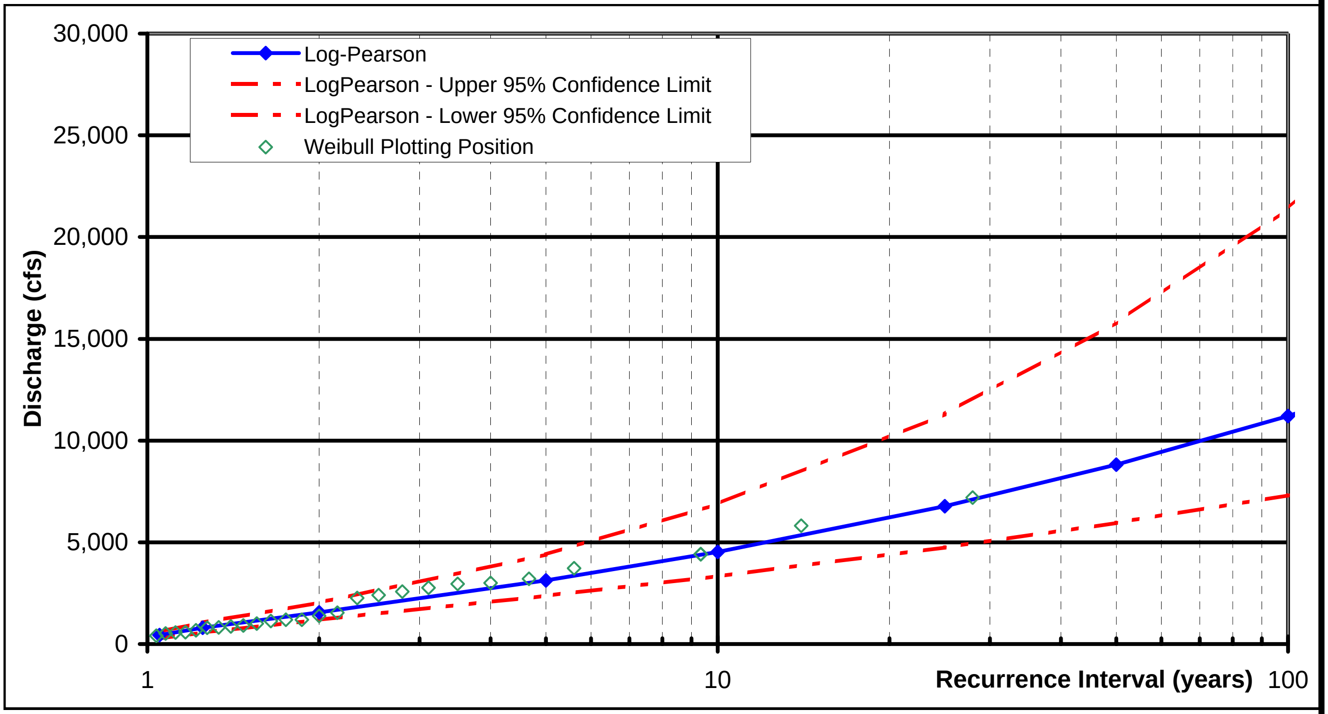
Station ID: *06752280* **Latitude, Longitude:** *40-33-07 105-00-39*
Drainage Area (mi^2): *1244* **County:** *Larimer*
Number of low outliers eliminated: *0* **State:** *Colorado*

	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

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