Boxelder B-4: Dam Breach Analysis

Larimer County, Colorado
June 2009

Boxelder B-4 embankment

residence at section 184295

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

(This page intentionally blank)

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

Lakewood, Colorado

June 2, 2009

BOXELDER B-4: DAM BREACH ANALYSIS

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

Summary: Predictions have been made of the likely extent and timing of flood flow resulting from a catastrophic breach of the Boxelder B-4 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. Most substantially, four residences will be threatened with substantial damage, with a depth*velocity product greater than 7. Due to this potential for loss of life, it is recommended that the hazard classification of this structure be increased from its current significant level to a high hazard classification. If these few structures were relocated or removed, retaining a significant hazard classification may be possible.

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

NRCS ***Helping People Help the Land***

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

(This page intentionally blank)

INTRODUCTION

This report details the methods and results of a dam breach analysis performed on the Boxelder B-4 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 communities impacted are primarily rural, though a small portion of the eastern suburbs of Fort Collins are in the breach floodway.

The Boxelder B-4 dam (NID ID: CO00524) is an earthen-embankment, typically-dry, flood retention structure. The structure is located on Indian Creek at approximately 5400 feet in elevation. This structure provides substantial flood-reduction benefits to the dispersed homes and ranches downstream.

Average precipitation within the reservoir's 13.72 square mile watershed is 15 inches, according to PRISM. The B-4 embankment has a maximum height of about 30.3 feet, with a crest elevation of 5408.3 feet, original ground elevation at the downstream toe of about 5378 feet and embankment length of 2400 feet. The maximum storage, with the water surface elevation at the crest of the embankment, is 2420 acre-feet. The emergency spillways are two parallel, 200 foot wide, earthen spillways on the left abutment. At the emergency spillway crest elevation of 5401.0 feet the associated reservoir storage is 1270 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 in the upper portion of the floodway. 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 APPENDIXES A. Valley cross-sections in the vicinity of four threatened homes are provided in APPENDIX B.**

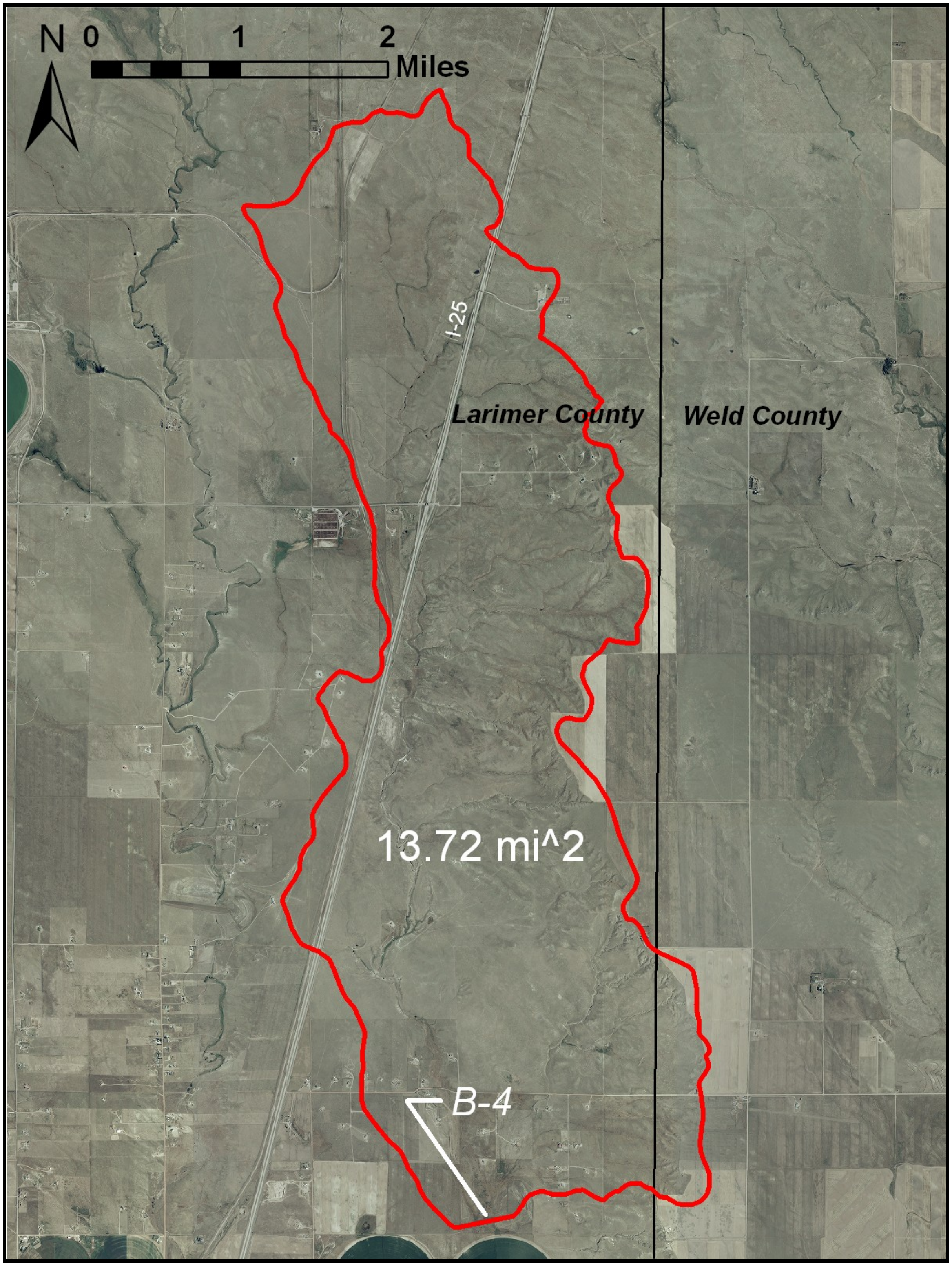


Figure 1: Boxelder B-4 reservoir watershed.

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 (5401 feet, breach volume = 1270 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-4 structure are provided in Figures 2 through 4. It is relevant to note the rather flat embankment slopes of 3.5:1 upstream and 3:1 downstream. 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 resulting enhanced erosion around the grass clumps.

Figure 2: Upstream face of B-4 embankment. Slope = 3.5:1

Figure 3: Downstream face of B-4 embankment. Slope = 3:1.

.

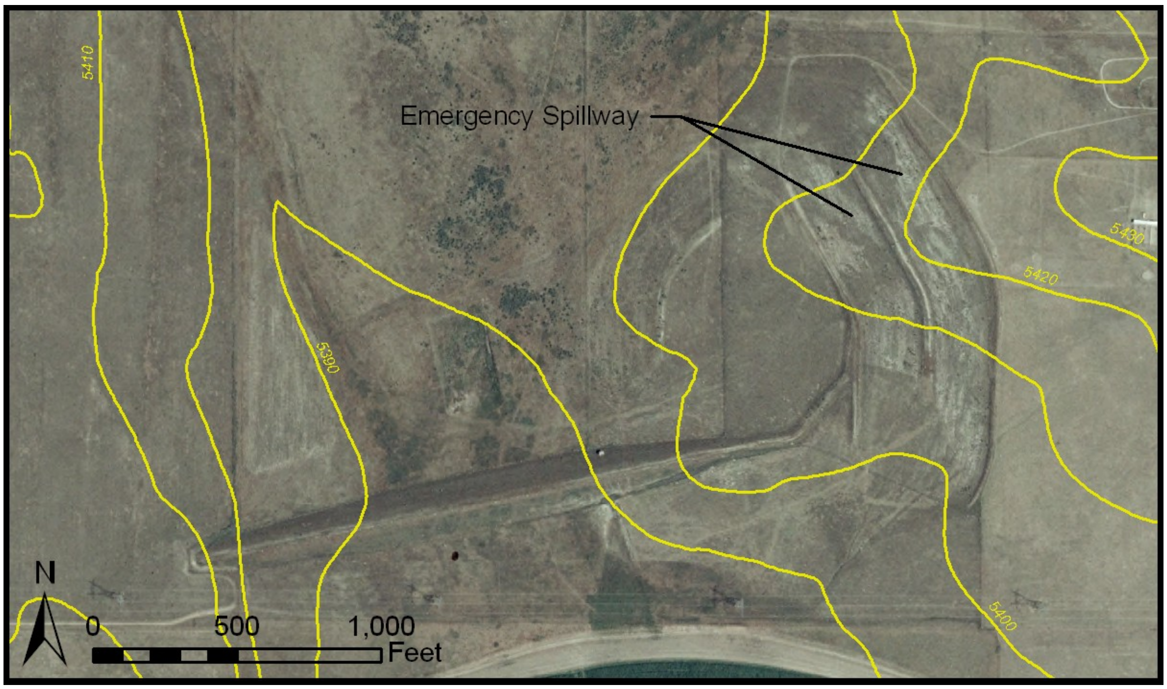


Figure 4: B-4 embankment. Aerial photography taken summer of 2005. Pre-construction 10-foot contour intervals shown.

The methods used to develop possible hydrograph characteristics are peak flow equations developed by NRCS, Froehlich, Kirkpatrick, and the U.S. Bureau of Reclamation; 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 (5401 feet).

Method	Peak Flow (cfs)	Volume (acre-feet)
NRCS peak flow	25,000	1270
Froehlich (1995) peak flow	16,000	1270
Kirkpatrick (1977) peak flow	6,400	1270
U.S. Bureau of Reclamation (1982) peak flow	25,000	1270
MacDonald and Langridge-Monoplolis (1984) peak flow	106,000	1270
Evan (1986) peak flow	49,000	1270
Fread BREACH	7,000	1250
breach geometry prediction (in HEC-RAS)	14,600	1260
Median:	**20,500**	

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). With H_w = 23 feet, V_s = 1270 acre-feet and A = 26,000 ft^2, the peak discharge is 1290 cfs (biased low due to large embankment length), should not be less than 8200 cfs but not in excess of 21,500 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 (7.0 m). Using this equation, a peak discharge of 16,000 cfs (460 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 6400 cfs (180 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 25,000 cfs (700 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 106,000 cfs (3010 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 49,000 cfs (1380 cms) is estimated.

There was a substantial range in the breach peak flow time estimates, from 6400 to 125,000 cfs. The average and median values were 42,700 and 23,300 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 (<u>not</u> 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-4 embankment. Bare earth cover on the embankment was assumed.

Results of this analysis provided a peak flow of 6960 cfs with a time to peak of 1.89 hours. This method provides substantially lower peak flow estimates than most of the statistically-based 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 is 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 (5401 feet).

Method	Formation Time (hours)
BREACH model (Fread 1988)	1.89
Froehlich (1995b)	0.85
MacDonald and Langridge-Monoplolis (1984)	0.44
U.S. Bureau of Reclamation (1988)	0.23
Von Thun and Gillette (A) highly erodible (1990)	0.11
Von Thun and Gillette (A) erosion resistant (1990)	0.39
Von Thun and Gillette (B) highly erodible (1990)	0.40
Von Thun and Gillette (B) erosion resistant (1990)	1.27
Median:	**0.42**

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 = 1,567,000 m^3 and H_b = 7.0 m, the breach formation time is estimated to be 0.85 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-4 embankment characteristics, V_{er} is 6770 and the breach formation time is 0.44 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 21.0 meters (69 ft) and formation time of 0.23 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:

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

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

where equation (15) is for highly-erodible materials and equation (16) is for erosion-resistant embankment materials. This method predicts the formation time as 0.11 and 0.39 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 (14) is for highly-erodible materials and equation (13) is for erosion-resistant embankment materials. The average breach width (B_{avg}) is:

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

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 35.8 meters (120 ft) and the formation time as 0.40 and 1.27 hours.

There was a substantial range in the breach formation time estimates, from 0.11 hours to 1.83 hours. The median value was 0.42 hours. Due to the relatively flat slopes of the embankment (3:1 and 3.5:1), and considering that this parameter is not included in the regressions, a longer breach formation time is most reasonable. A 50 minute breach formation time 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 (hours)
BREACH model (Fread 1988)	16
Froehlich (1995b)	82
U.S. Bureau of Reclamation (1988)	69
Von Thun and Gillette (1990)	120
Median:	**76**

The BREACH model computes an unusually-low breach width of 16 feet. This breach width is less than all but 2 of the 70 breaches documented in Froehlich (2008). One actual failure (Buckhaven No. 2, TN) had similar characteristics as Boxelder B-2 BREACH analysis: 15 feet average breach width with a breach height of 20 feet – considering this, the BREACH results appear to be possible, though unlikely.

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 1,567,000 m^3 and depth of water of 7.0 m, this method predicts an average breach width of 25.1 m (82 feet).

As developed from Equation (14), the U.S. Bureau of Reclamation (1988) predicts an average breach width of 21.0 meters (69 ft).

Von Thun and Gillette (1990) provides average breach width from equation 19. This method predicts an average breach width of 35.8 meters (120 ft)

There was a substantial range in the average breach width estimates, from 16 to 120 feet. The median value was 76 feet. A breach hydrograph was developed for a scenario with a 76 feet wide average breach width, side slopes of 0.9 and formation time of 50 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 2006). Given this breach geometry and formation time, the model simulates a breach hydrograph with a peak at 14,700 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. 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.

Reviewing the results of the various analyses, there is a very wide range of peak flow predicted using the various methodologies, from 6400 cfs (Kirkpatrick) to 106,000 cfs (MacDonald and Langridge-Monoplolis peak flow). Comparing the low value to the maximum emergency spillway flow (13,400 cfs) indicates that it may likely be too low. Considering the other results, the high value seems very excessive.

Using professional judgment, it was decided to use the breach geometry prediction HEC-RAS model output, with the median width of 76 feet, side slopes of 0.9:1, and, considering the rather-flat slopes of the embankment, a formation time of 50 minutes (0.83 hours). These parameters produce a peak breach flow of 14,700 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

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:

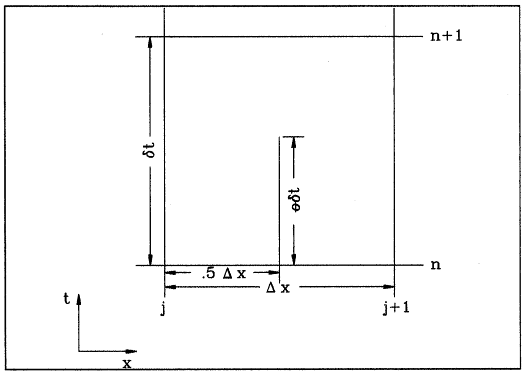
$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0$$

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

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: $\dfrac{\partial f}{\partial t} \approx \dfrac{\Delta f}{\Delta t} = \dfrac{0.5(\Delta f_{j+1} + \Delta f_j)}{\Delta t}$

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:

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

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-4 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 upper portions of the model, an unsteady flow model was developed from the Boxelder B-4 structure to the Cache la Poudre River. This hydraulic model was inherently stable, and provides reasonable estimates of peak discharge and water surface elevations given the limited geometric data available.

The model assumes a piping failure, with an initial water surface at the crest of the emergency spillway (5401.0 feet). No road crossings were modeled as bridges or culverts, including the I-25 crossing, due to limited geometric data availability (I-25) and expected insignificant impacts (lesser crossings). Additionally, the effects of several farm ponds on Indian Creek were not modeled.

A normal depth boundary condition assumption was made at the downstream limit of the model (slope = 0.0029) and an initial flow of 500 cfs was assumed at all sections.

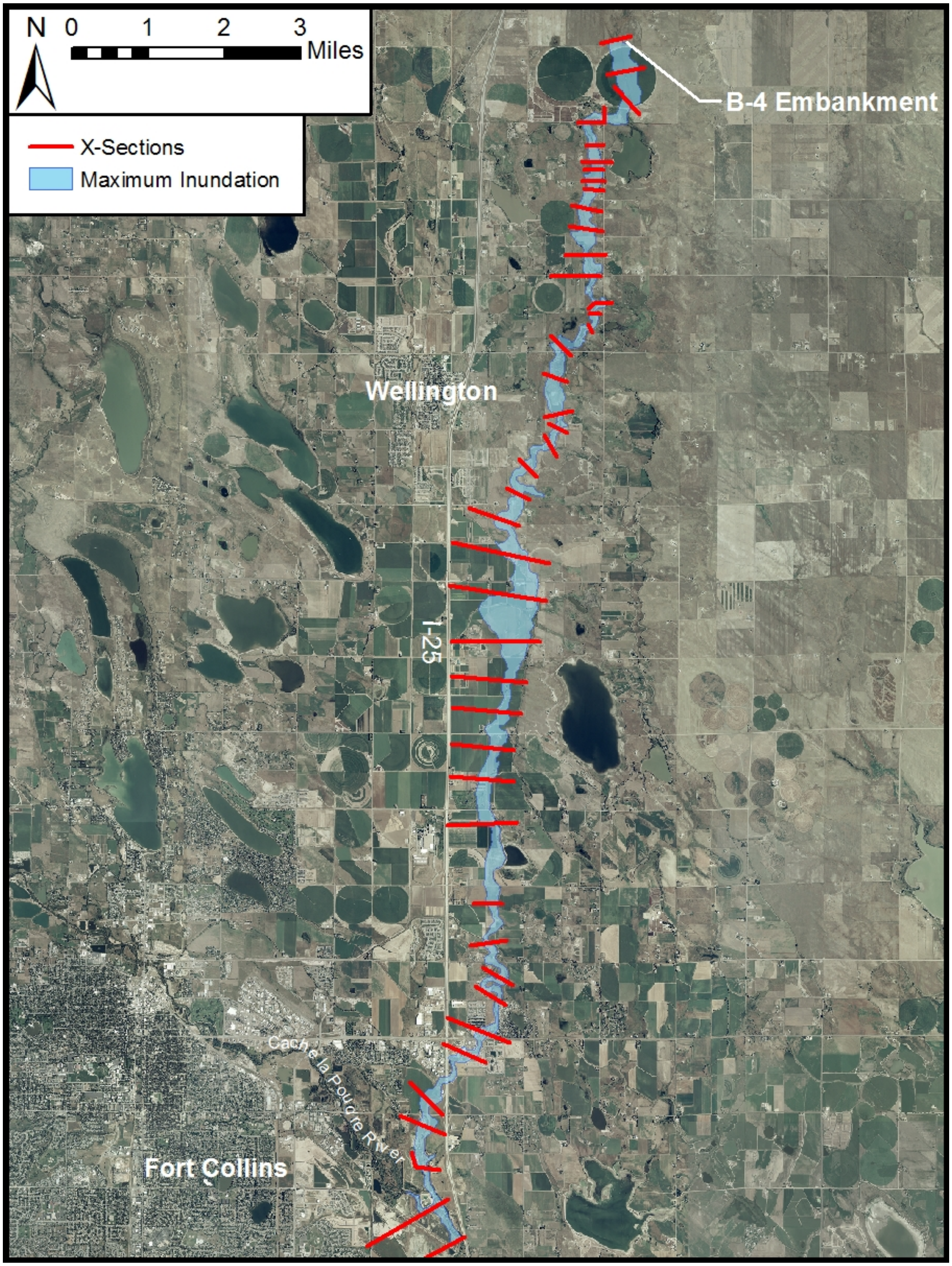


Figure 5: Inundation extent to the Cache la Poudre River, Boxelder B-4 breach analysis.

INUNDATION EXTENT AND TIMING

This analysis provides a prediction of the extent and timing of flooding from a catastrophic breach of the Boxelder B-4 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 14,700 cfs at B-4, the flow attenuates to 12,500 cfs at the confluence with Boxelder Creek, 8900 cfs in the Eastern suburbs of Fort Collins (I-25 and Mulberry Street), and 7500 cfs at the Poudre River. Based upon 1980 to 2007 data in the Poudre River at the Boxelder, this discharge corresponds to about the 30-year flood event. (See frequency analysis in Appendix C).

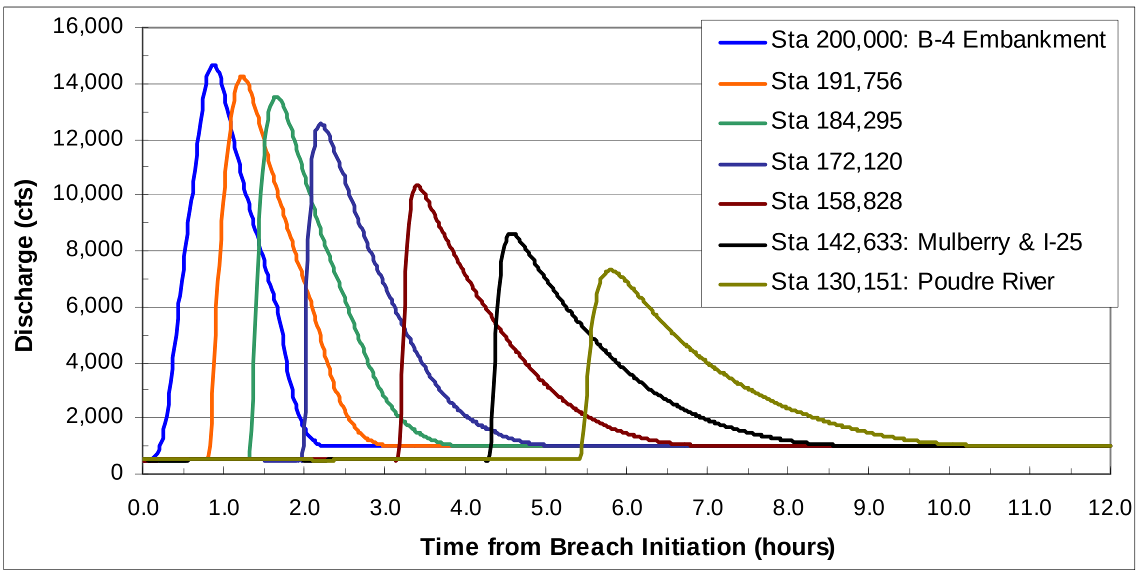


Figure 6: Breach hydrographs.

Table 4 provides the model results at each non-interpolated cross section. Figure 6 illustrates the routed breach hydrographs at 7 points within the analysis extent. 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 modeled sections. In the vicinity of residences potentially inundated, 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 the vicinity of homes threatened by destruction have been provided in **Appendix B**. These sections include the water surface elevation, structures and relevant hydraulic characteristics of the peak flow.

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. Most substantially, four residences will be threatened with substantial damage (depth*velocity > 7), with peak flow times (counted from initiation of failure) ranging from 1.2 hours to 3.4 hours.

A number of other structures are projected to be inundated to a minor degree. The relatively few homes projected to be substantially impacted by a failure could warrant a significant hazard classification, instead of high hazard, but NRCS and the State of Colorado do not factor in the number of potential depth into a hazard classification (unlike the U.S. Bureau of Restoration). Due to this potential for loss of life, **it is recommended that the hazard classification of this structure be increased from its current significant level to a high hazard classification**. If these few structures were relocated or removed, retaining a significant hazard classification may be possible for the Boxelder B-4 structure.

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

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (feet)	Energy Grade Slope (ft/ft)	Peak Velocity			Froude Number	
				Channel (ft/s)	Left (ft/s)	Right (ft/s)	Channel	X-S
200000	14,700	5389.63	0.00699	8.2	5.5	4.7	0.77	0.78
198402	14,600	5372.51	0.00565	6.3	3.8	3.9	0.67	0.66
196647	14,500	5357.01	0.00799	8.9	4.8	5.2	0.83	0.84
195140	14,500	5338.00	0.00414	6.6	1.5	4.3	0.54	0.56
194135	14,400	5330.60	0.00638	8.8	4.9	3.0	0.76	0.78
192936	14,300	5317.39	0.00564	8.3	5.4	5.1	0.71	0.72
192131	14,300	5310.86	0.00711	7.1	2.7	4.8	0.65	0.67
191756	14,300	5308.07	0.00377	7.1	3.1	4.4	0.55	0.54
191189	14,200	5303.26	0.00703	6.8	6.0	2.9	0.55	0.72
190779	14,200	5298.62	0.00698	7.8	6.3		0.56	0.71
189730	14,200	5286.50	0.00389	6.6	2.9	4.0	0.59	0.61
188679	14,100	5281.06	0.00695	9.7	3.9	5.1	0.59	0.77
186804	14,000	5259.77	0.00261	5.8	3.5	3.9	0.41	0.45
185720	13,800	5254.08	0.00753	7.5	1.6	1.5	0.62	0.72
184295	13,600	5239.33	0.00223	4.5	3.4	2.4	0.32	0.39
183800	13,500	5236.73	0.00526	7.6	3.6	2.9	0.52	0.55
182879	13,500	5229.25	0.00828	11.0	9.2	6.2	0.68	0.72
181013	13,400	5217.06	0.00863	7.0	2.9	4.2	0.67	0.7
179466	13,400	5199.98	0.00771	6.2	2.7	4.7	0.59	0.63
177423	13,300	5179.08	0.00844	8.8	0.6		0.84	0.84
176695	13,200	5170.89	0.00295	6.9	4.9	5.0	0.45	0.49
175700	13,100	5166.74	0.00410	7.4	3.6	4.4	0.61	0.64
173959	12,900	5156.87	0.00332	6.4	3.4	3.5	0.55	0.56
172120	12,600	5146.70	0.00295	7.4	4.0	5.7	0.54	0.54
170647	12,300	5137.42	0.00330	5.8	2.7	2.9	0.53	0.56
168832	12,100	5124.50	0.00444	5.7	2.6	2.7	0.59	0.62
166626	11,700	5110.01	0.00447	4.4	3.1	3.0	0.56	0.53
164321	11,200	5090.35	0.00479	5.5	2.0	2.5	0.56	0.64
162205	11,000	5071.57	0.00613	6.1	1.3	1.7	0.61	0.63
160626	10,800	5057.79	0.00285	4.9	2.3	2.1	0.46	0.48
158828	10,500	5047.34	0.00444	6.1		3.1	0.54	0.59
157251	10,300	5036.03	0.00601	5.5	3.4	2.4	0.49	0.53
154993	10,100	5016.37	0.00243	4.3	1.6	2.4	0.44	0.45
151037	9,400	4996.18	0.00624	5.7	4.4	5.4	0.54	0.63
148992	9,300	4978.55	0.00392	5.4	0.6	1.9	0.52	0.58
146966	9,200	4967.95	0.00580	6.0	2.9	4.6	0.51	0.59
144400	9,000	4951.22	0.01046	9.3			0.71	0.71
142633	8,900	4936.47	0.00524	3.3	1.5	1.3	0.34	0.35
141075	8,600	4926.78	0.00441	3.9	1.2	1.2	0.38	0.4
137600	8,300	4906.66	0.00510	5.0	3.9	2.7	0.52	0.56
136172	8,200	4896.86	0.00324	4.3	3.3	2.4	0.4	0.49
134084	8,000	4884.67	0.00546	5.0	1.2	3.5	0.48	0.58
132197	7,800	4865.80	0.00208	3.2	1.2	1.5	0.3	0.31
130151	7,500	4860.52	0.00474	3.8		3.3	0.5	0.52

SUMMARY AND CONCLUSIONS

A comprehensive approach was implemented to develop a most likely breach hydrograph of the Boxelder B-4 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. Through professional judgment, it was decided to use the breach hydrograph as developed by the HEC-RAS breach simulation, with a peak of 14,700 cfs, formation time of 50 minutes, and volume of 1260 acre-feet.

The breach hydrograph was routed using HEC-RAS 4.0 from the embankment to the confluence with the Poudre River, 16 miles downstream. According to the model, the flow attenuates to 12,500 cfs at the confluence with Boxelder Creek, 8900 cfs in the Eastern suburbs of Fort Collins (I-25 and Mulberry Street), and 7500 cfs at the Poudre River. This final discharge corresponds to about a 30-year flood event.

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. Most substantially, four residences will be threatened with substantial damage, with a depth*velocity product greater than 7. Due to this potential for loss of life, it is recommended that the hazard classification of this structure be increased from its current significant level to a high hazard classification. If these few structures were relocated or removed, retaining a significant hazard classification may be possible.

REFERENCES

Brunner, Gary W., Goodwell, Chris R. 2002 *HEC-RAS River Analysis System, Hydraulic Reference Manual* US Army Corps of Engineers, Hydraulic Engineering Center (HEC), CPD-69.

Brunner, Gary W. 2006 *HEC-RAS River Analysis System, User's Manual* US Army Corps of Engineers, Hydraulic Engineering Center (HEC), CPD-68.

Fread, D.L. 1988 "BREACH: An Erosion Model for Earthen Dam Failures" National Oceanic and Atmospheric Administration, National Weather Service, Hydrologic Research Laboratory.

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," *ASCE 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.

U.S. Bureau of Reclamation (USBR) 1987 "Design of Small Dams" U.S. Department of Interior, Bureau of Reclamation.

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

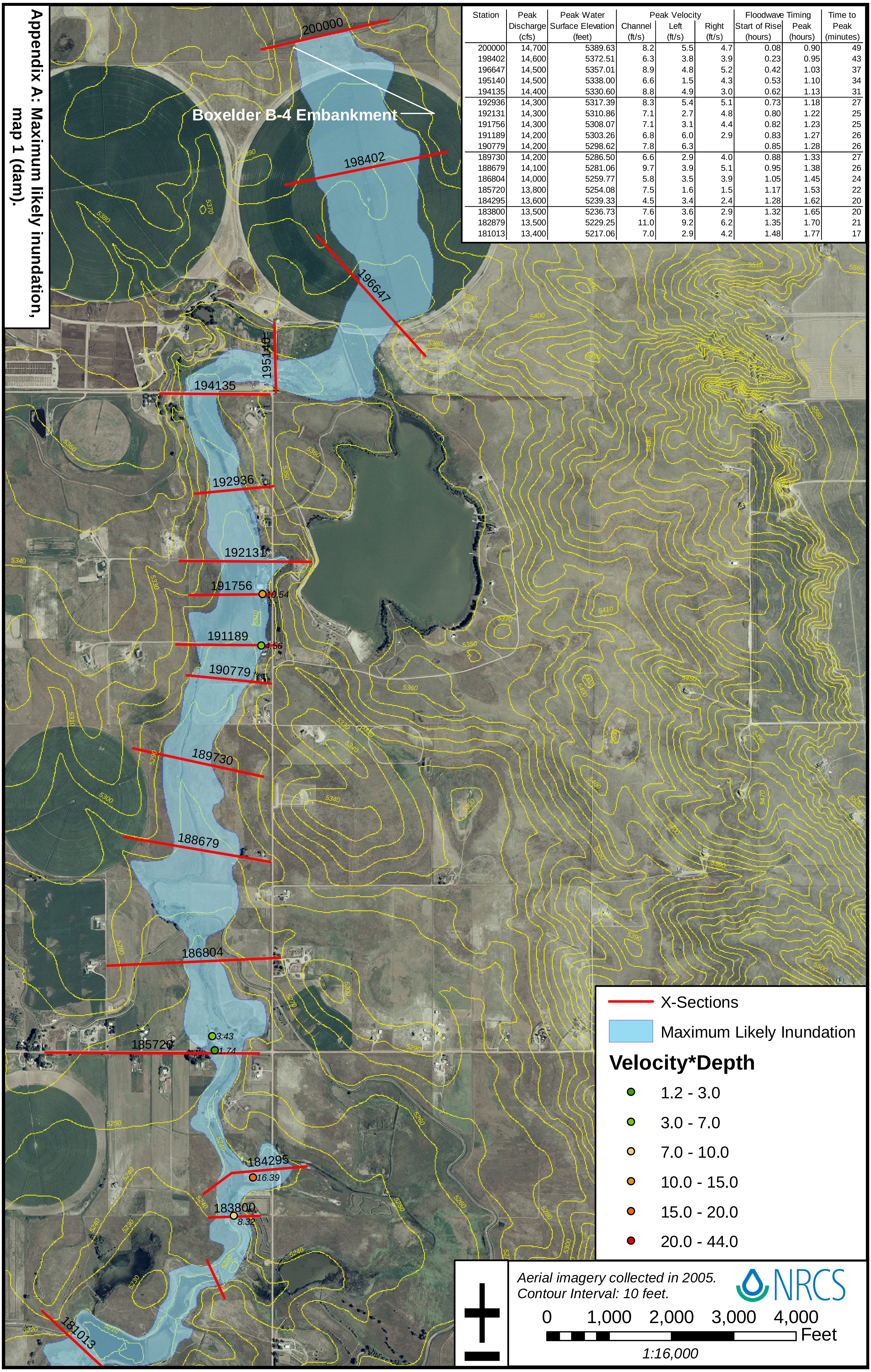
Wahl, Tony L. 1998 *Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment* U.S. Department of Interior, Bureau of Reclamation, Dam Safety Office, DSO-98-004.

(This page intentionally blank)

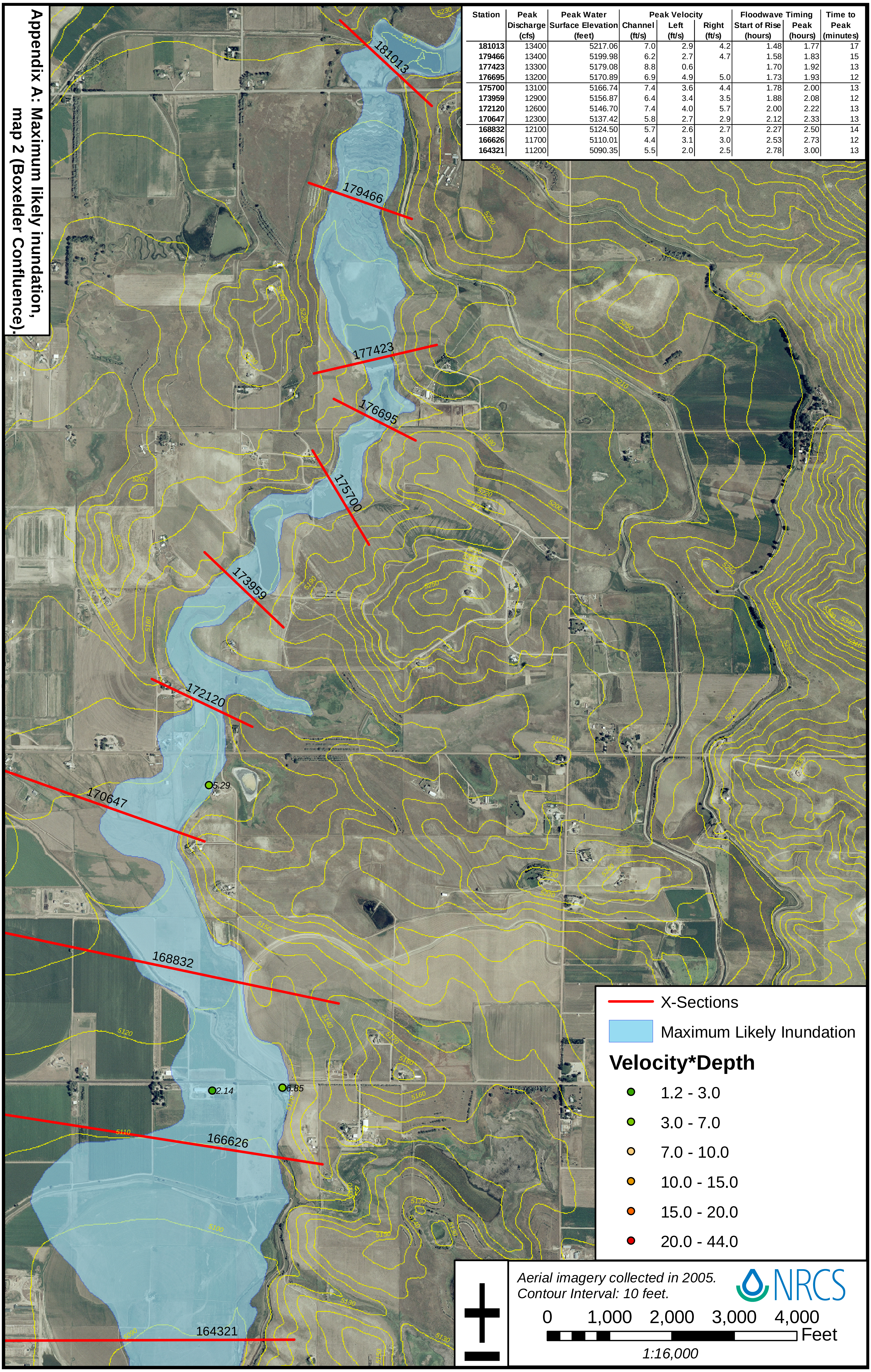
APPENDIX A: Maximum Likely Inundation

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

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (feet)	Peak Velocity			Floodwave Timing		Time to Peak (minutes)
			Channel (ft/s)	Left (ft/s)	Right (ft/s)	Start of Rise (hours)	Peak (hours)	
200000	14,700	5389.63	8.2	5.5	4.7	0.08	0.90	49
198402	14,600	5372.51	6.3	3.8	3.9	0.23	0.95	43
196647	14,500	5357.01	8.9	4.8	5.2	0.42	1.03	37
195140	14,500	5338.00	6.6	1.5	4.3	0.53	1.10	34
194135	14,400	5330.60	8.8	4.9	3.0	0.62	1.13	31
192936	14,300	5317.39	8.3	5.4	5.1	0.73	1.18	27
192131	14,300	5310.86	7.1	2.7	4.8	0.80	1.22	25
191756	14,300	5308.07	7.1	3.1	4.4	0.82	1.23	25
191189	14,200	5303.26	6.8	6.0	2.9	0.83	1.27	26
190779	14,200	5298.62	7.8	6.3		0.85	1.28	26
189730	14,200	5286.50	6.6	2.9	4.0	0.88	1.33	27
188679	14,100	5281.06	9.7	3.9	5.1	0.95	1.38	26
186804	14,000	5259.77	5.8	3.5	3.9	1.05	1.45	24
185720	13,800	5254.08	7.5	1.6	1.5	1.17	1.53	22
184295	13,600	5239.33	4.5	3.4	2.4	1.28	1.62	20
183800	13,500	5236.73	7.6	3.6	2.9	1.32	1.65	20
182879	13,500	5229.25	11.0	9.2	6.2	1.35	1.70	21
181013	13,400	5217.06	7.0	2.9	4.2	1.48	1.77	17



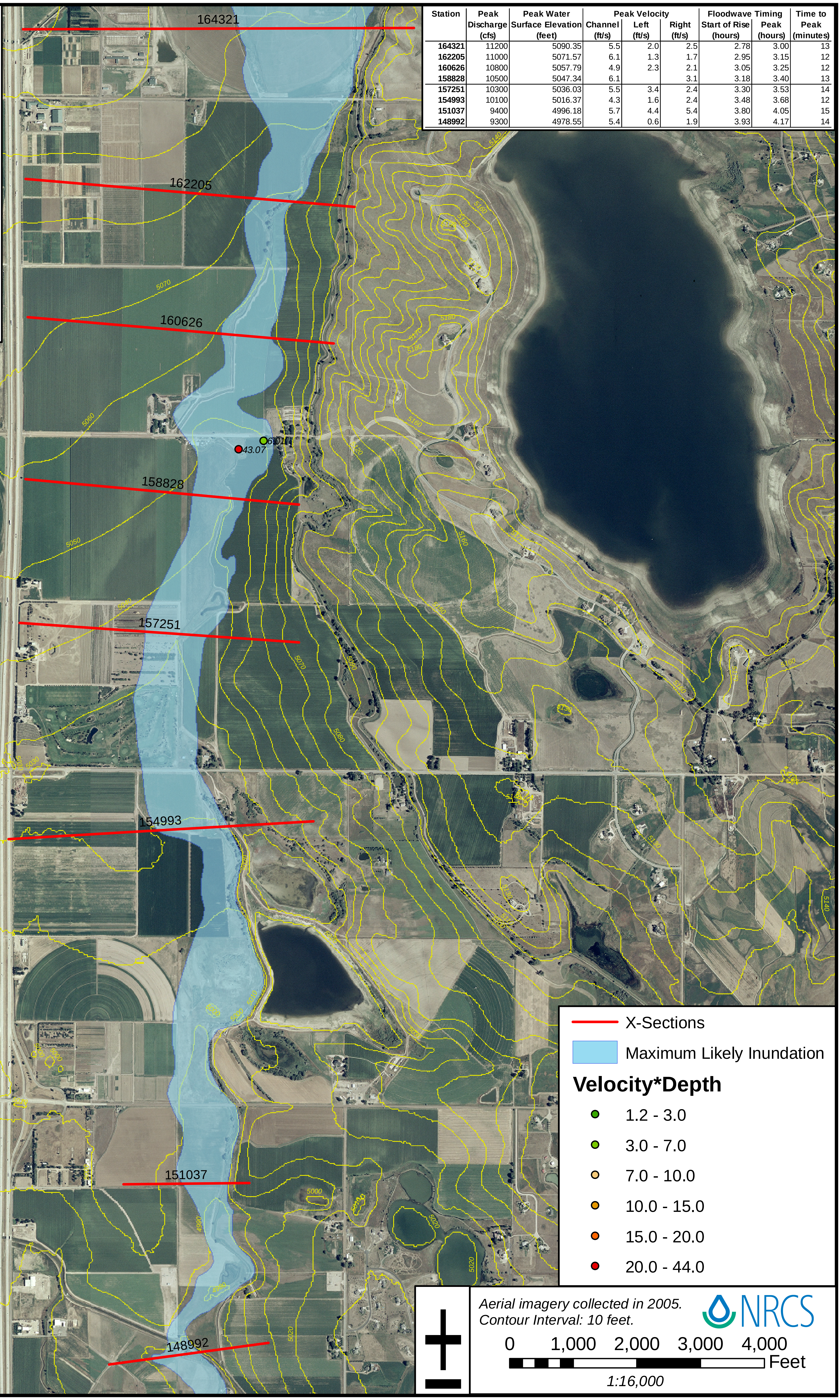
Appendix A: Maximum likely inundation, map 2 (Boxelder Confluence).



Station	Peak Discharge (cfs)	Peak Water Surface Elevation (feet)	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)
181013	13400	5217.06	7.0	2.9	4.2	1.48	1.77	17
179466	13400	5199.98	6.2	2.7	4.7	1.58	1.83	15
177423	13300	5179.08	8.8	0.6		1.70	1.92	13
176695	13200	5170.89	6.9	4.9	5.0	1.73	1.93	12
175700	13100	5166.74	7.4	3.6	4.4	1.78	2.00	13
173959	12900	5156.87	6.4	3.4	3.5	1.88	2.08	12
172120	12600	5146.70	7.4	4.0	5.7	2.00	2.22	13
170647	12300	5137.42	5.8	2.7	2.9	2.12	2.33	13
168832	12100	5124.50	5.7	2.6	2.7	2.27	2.50	14
166626	11700	5110.01	4.4	3.1	3.0	2.53	2.73	12
164321	11200	5090.35	5.5	2.0	2.5	2.78	3.00	13

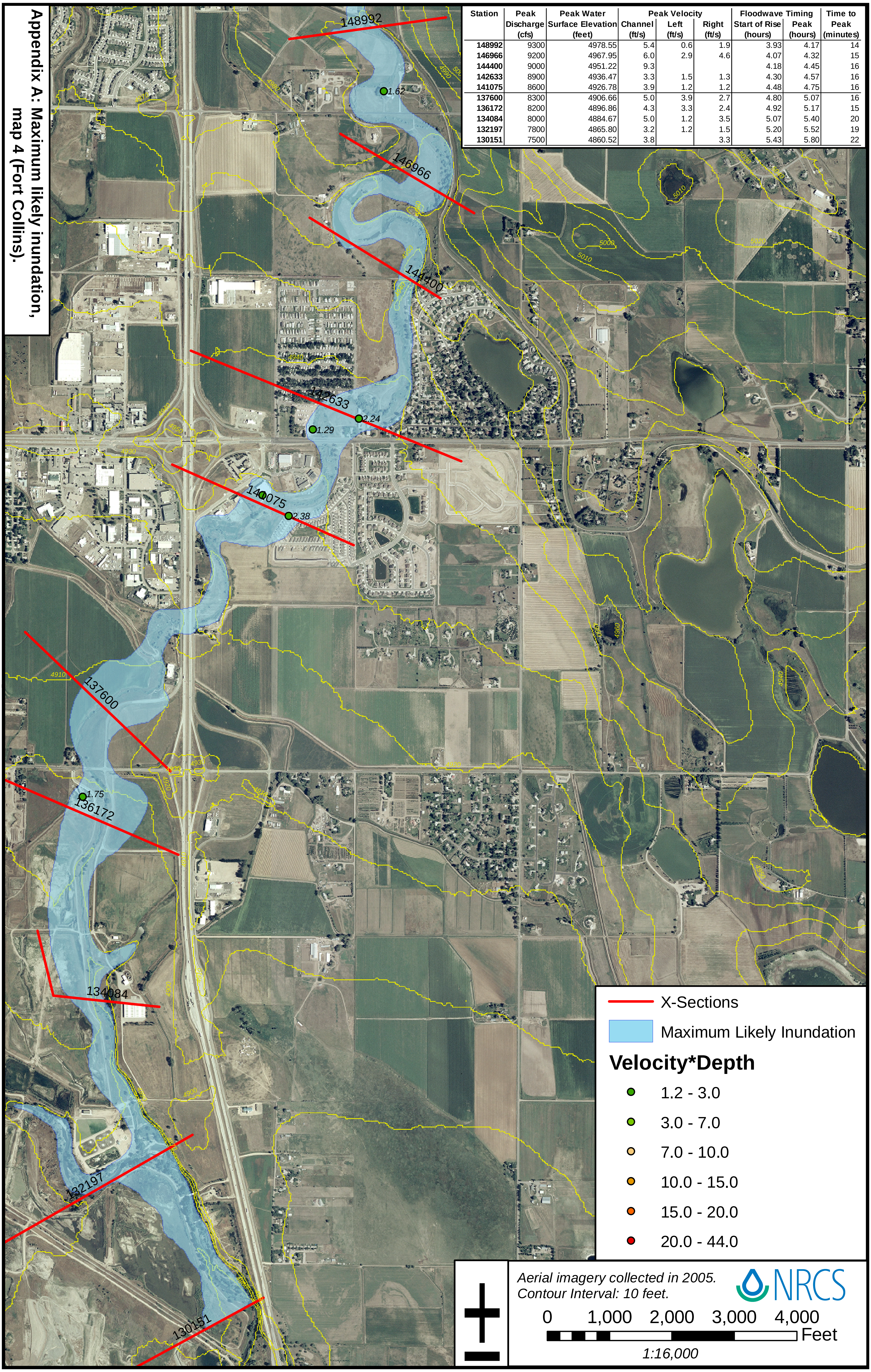
Appendix A: Maximum likely inundation, map 3.

Station	Peak Discharge (cfs)	Peak Water Surface Elevation (feet)	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)
164321	11200	5090.35	5.5	2.0	2.5	2.78	3.00	13
162205	11000	5071.57	6.1	1.3	1.7	2.95	3.15	12
160626	10800	5057.79	4.9	2.3	2.1	3.05	3.25	12
158828	10500	5047.34	6.1		3.1	3.18	3.40	13
157251	10300	5036.03	5.5	3.4	2.4	3.30	3.53	14
154993	10100	5016.37	4.3	1.6	2.4	3.48	3.68	12
151037	9400	4996.18	5.7	4.4	5.4	3.80	4.05	15
148992	9300	4978.55	5.4	0.6	1.9	3.93	4.17	14



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

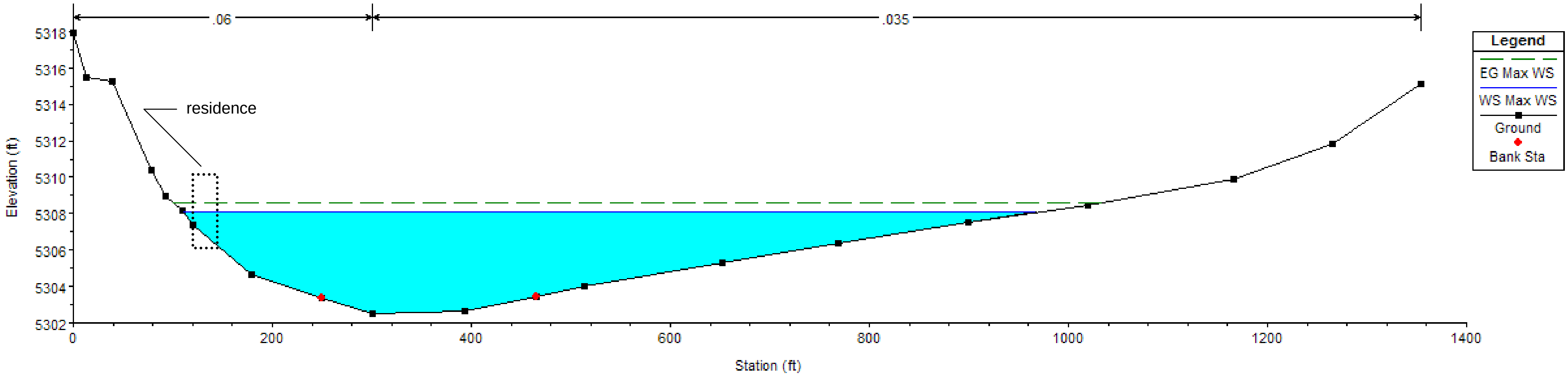
Station	Peak Discharge (cfs)	Peak Water Surface Elevation (feet)	Peak Velocity Channel (ft/s)	Peak Velocity Left (ft/s)	Peak Velocity Right (ft/s)	Floodwave Timing Start of Rise (hours)	Floodwave Timing Peak (hours)	Time to Peak (minutes)
148992	9300	4978.55	5.4	0.6	1.9	3.93	4.17	14
146966	9200	4967.95	6.0	2.9	4.6	4.07	4.32	15
144400	9000	4951.22	9.3			4.18	4.45	16
142633	8900	4936.47	3.3	1.5	1.3	4.30	4.57	16
141075	8600	4926.78	3.9	1.2	1.2	4.48	4.75	16
137600	8300	4906.66	5.0	3.9	2.7	4.80	5.07	16
136172	8200	4896.86	4.3	3.3	2.4	4.92	5.17	15
134084	8000	4884.67	5.0	1.2	3.5	5.07	5.40	20
132197	7800	4865.80	3.2	1.2	1.5	5.20	5.52	19
130151	7500	4860.52	3.8		3.3	5.43	5.80	22



APPENDIX B: Valley Cross Sections

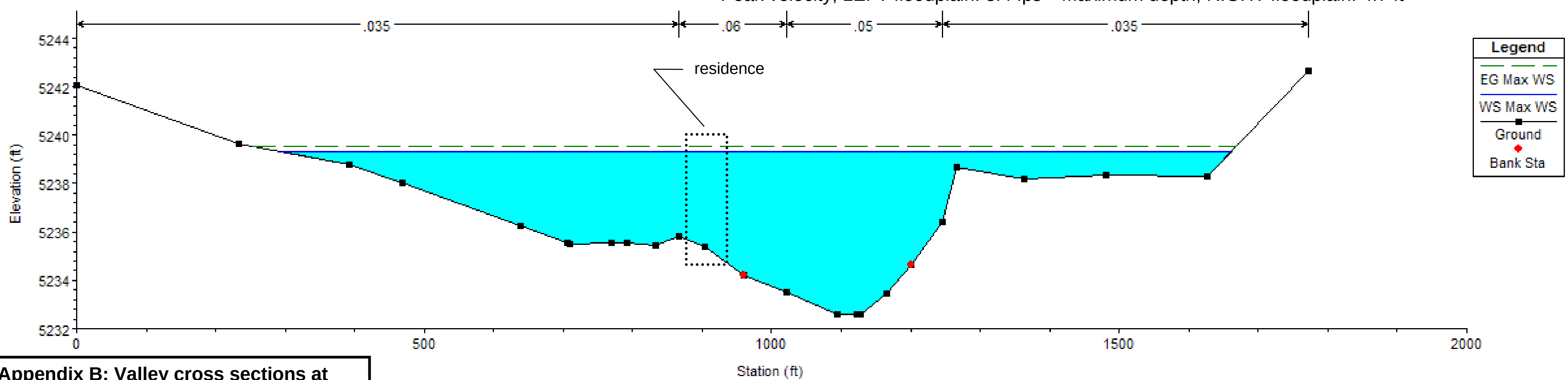
Section 191756

Peak Discharge: 14,300 cfs
Peak WSEL: 5308.07 ft
Peak velocity, channel: 7.1 fps
Peak velocity, LEFT floodplain: 3.1 fps
Peak velocity, RIGHT floodplain: 4.4 fps
Maximum depth, channel: 5.6 ft
Maximum depth, LEFT floodplain: 4.7 ft
Maximum depth, RIGHT floodplain: 4.6 ft



Section 184295

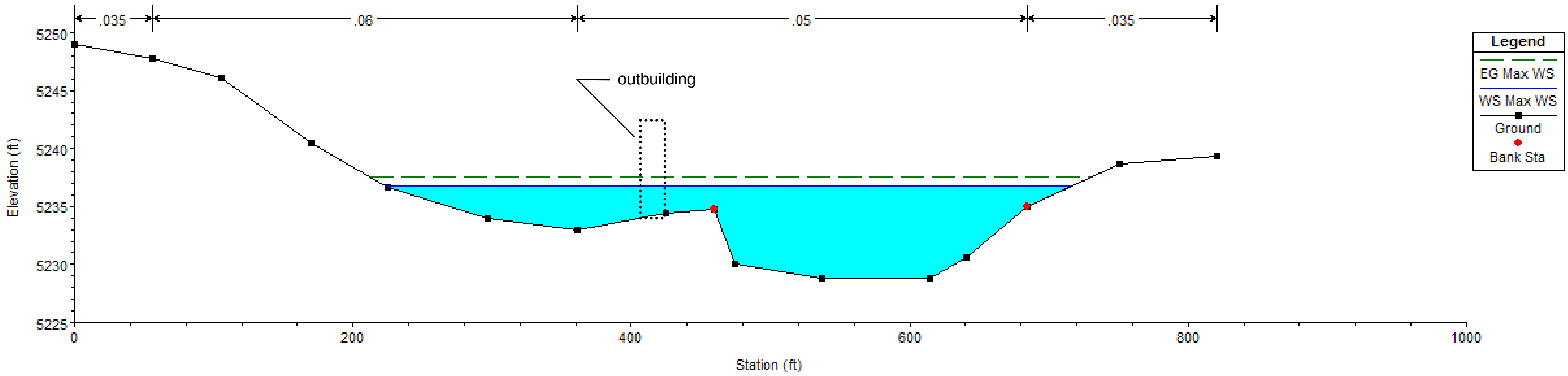
Peak Discharge: 13,600 cfs
Peak WSEL: 5239.33 ft
Peak velocity, channel: 4.5 fps
Peak velocity, LEFT floodplain: 3.4 fps
Peak velocity, RIGHT floodplain: 2.4 fps
Maximum depth, channel: 6.8 ft
Maximum depth, LEFT floodplain: 5.1 ft
Maximum depth, RIGHT floodplain: 4.7 ft



Appendix B: Valley cross sections at threatened structures.

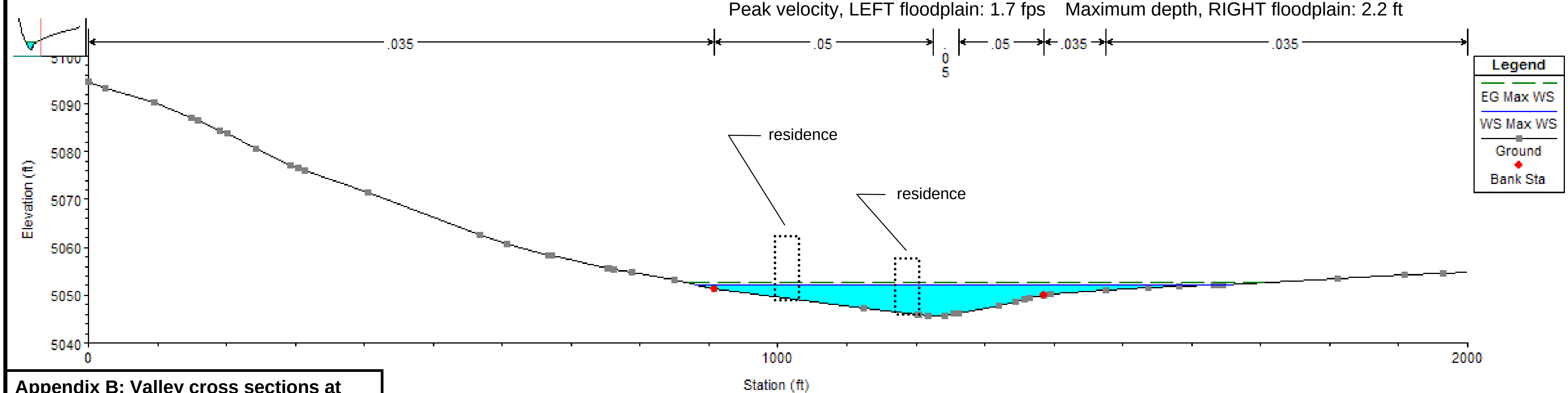
Section 183800

Peak Discharge: 13,500 cfs
Peak WSEL: 5236.73 ft
Peak velocity, channel: 7.6 fps
Peak velocity, LEFT floodplain: 3.6 fps
Peak velocity, RIGHT floodplain: 2.9 fps
Maximum depth, channel: 7.9 ft
Maximum depth, LEFT floodplain: 3.7 ft
Maximum depth, RIGHT floodplain: 1.8 ft



Section 159598*

Peak Discharge: 10,500 cfs
Peak WSEL: 5052.23 ft
Peak velocity, channel: 4.9 fps
Peak velocity, LEFT floodplain: 1.7 fps
Peak velocity, RIGHT floodplain: 3.1 fps
Maximum depth, channel: 6.5 ft
Maximum depth, LEFT floodplain: 1.0 ft
Maximum depth, RIGHT floodplain: 2.2 ft



Appendix B: Valley cross sections at threatened structures.

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) ----

With Weighted Generalized Skew

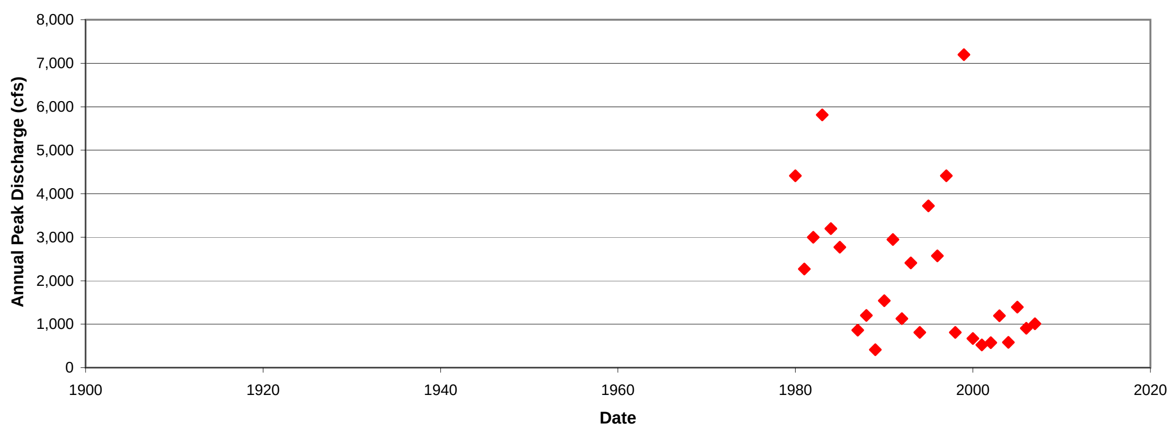
Generalized Skew Coefficient(3):
Variance of Generalized Skew(3):
A: -0.319127
B: 0.904663
station skew: 0.135910
MSE Station Skew: 0.19526925
Weighted skew coefficient(1): 0

Recurrence Interval(2) (years)	Percent Chance	K-Value	Ln(Q)	Peak(4) Discharge (cfs)	95% Confidence Limits Upper (cfs)	95% Confidence Limits Lower (cfs)
Without Generalized Skew						
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						
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	----	----	----

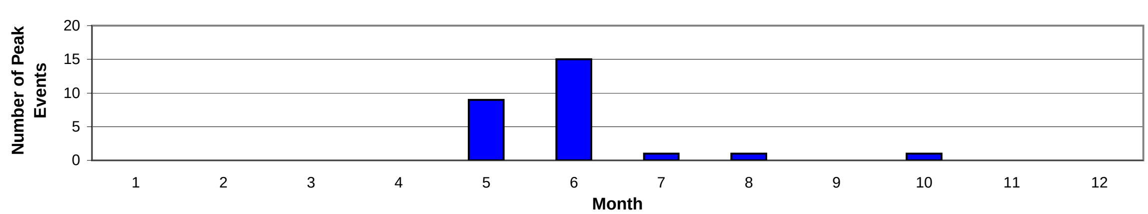
(1) Station and generalized skews must be between -2.00 and +3.00 in this spreadsheet.
(2) Considering the relatively short length of most gage records, less frequent peak estimates need to be used with considerable care.
(3) Computed one of four ways (see "generalized skew coefficient" worksheet): Mean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

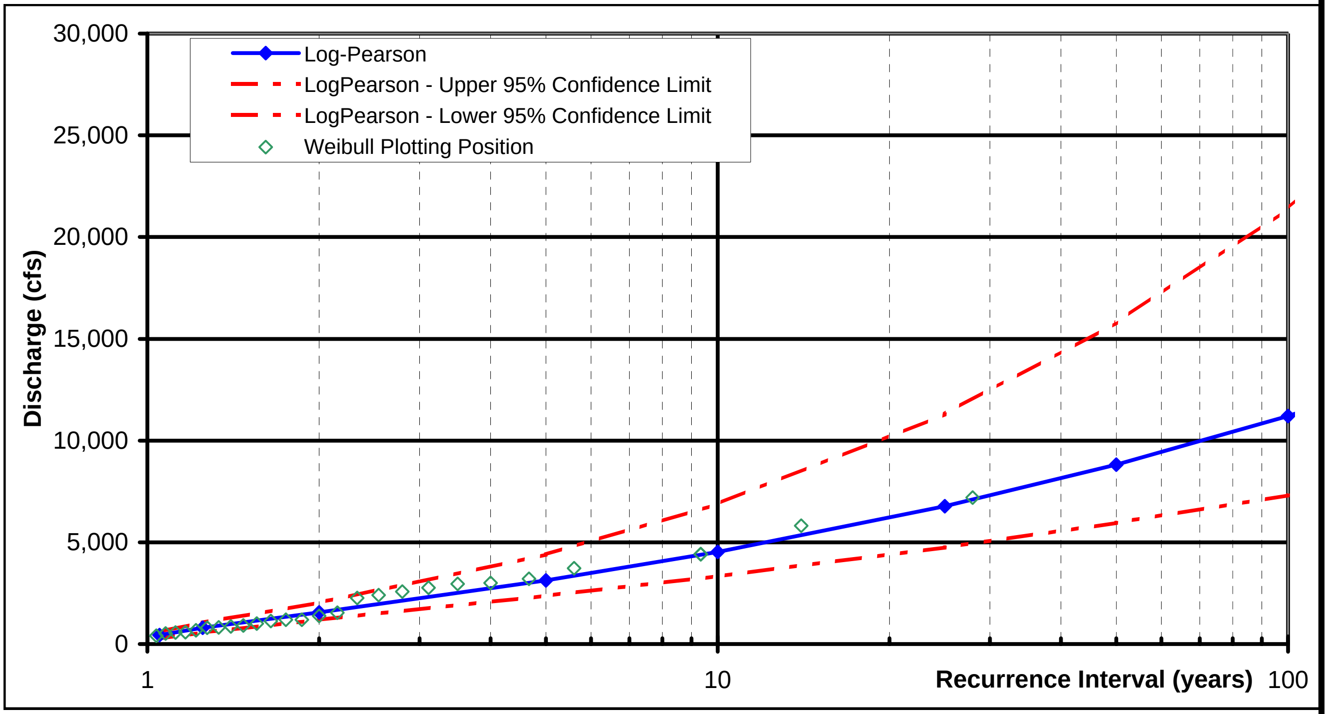
Station ID: *06752280* **Latitude, Longitude:** *40-33-07 105-00-39*
Drainage Area (mi^2): *1244* **County:** *Larimer*
Number of low outliers eliminated: *0* **State:** *Colorado*

	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