May 24, 2010
W-P Project No. 12018A

Mr. Ed Wojnowski, Town Administrator
Town of Newmarket
186 Main Street
Newmarket, New Hampshire 03857

Subject: Macallen Dam - Preliminary Report
Dam Breach Analysis

Dear Mr. Wojnowski:

The purpose of this letter report is to summarize the results of the dam breach modeling conducted on the Macallen Dam in May of 2010. The breach analysis is part of Wright-Pierce's overall assessment of the dam, which includes structural inspection and analysis, drafting of an Emergency Action Plan and preparation of a permit application to increase the discharge capacity of the dam.

Background

The Town of Newmarket has requested Wright-Pierce to perform preliminary engineering studies to confirm the hazard classification and provide initial inundation mapping of the Macallen Dam (DES Dam No. 177.01) for future incorporation into the Emergency Action Plan. In general, the Macallen Dam was classified by the DES as a "Significant Hazard" (Class B) dam in March of 2008 and Significant-to-High (Class B-to C) in April 2008.

The dam breach analysis was conducted to determine the nature and extent of downstream flooding if the dam were ever to be breached. The breach analysis was performed under the following flow conditions:

<u>Flow Condition 1:</u> 100-year flood flow (~8,302 cfs)
<u>Flow Condition 2:</u> "Sunny-Day" flow (~272 cfs)

Both flow conditions were run during high and low tide downstream boundary elevations. The location of the Macallen Dam is shown in Figure 1.

Dam Breach Analysis Model

The breach analysis was completed using the U.S. Army Corps of Engineers Hydraulic Engineering Center's River Analysis System (HEC-RAS) computer program (Version 4.1). HEC-

230 Commerce Way, Suite 302
Portsmouth, NH 03801 USA
Phone 603.430.3728 | Fax 603.430.4083

RAS is computer software designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. The system is capable of performing steady and unsteady flow water surface profile calculations in addition to dam breach scenarios.

HEC-RAS Model Input Data Collection

Data for the breach analysis model was collected from the following sources:

Field Data Collection/Site Visit

Wright-Pierce staff visited the Macallen Dam site and surrounding area on April 23, 2010. Field notes recorded pertinent information relative to river widths, natural channel conditions, and dam structure geometry (including height, width, depth, material of construction, headwall/tail-wall geometry, water depth below structure, dimension of supports, channel geometry immediately upstream/downstream, and vegetative cover).

Geographical Information System (GIS)

Publicly available GIS topography data was used to produce cross sectional geometry of the river waterway beyond the immediate vicinity of the dam and was used to calculate watershed boundaries and land/water areas tributary to the dam. GIS data was also used to do the following:

- Generate elevation datum
- Generate river channel cross sections
- Determine river channel low points and channel locations
- Calculate slope of river and lowest potential elevation in river channel at each station
- Create channels in the model based on calculated elevations and field observations
- Interpolation between cross sections to provide additional information/boundary conditions for the model.

United States Geological Survey (USGS) Maps

USGS maps were used to produce watershed boundaries, cross sectional information, and to locate structures.

USGS National Water Information System (NWIS) Web interface

The NWIS Web interface was used to verify river peak flow information for the Lamprey River watershed and surrounding area. The closest available river gauge is located upstream of the dam at USGS station 01073500. Because additional runoff is directed to the Lamprey River downstream of the gauge, an area approximation method was used to approximate flow at the dam.

The 100-year flood flow data was referenced from the April 7, 2008 DES Inspection Report as 8,302 cfs. The "Sunny Day" flow of 272 cfs was determined from averaging historical NWIS data collected for the past 70 years for events recorded below the threshold limit of 2,000 cfs.

Aerial Photographs
Publically available aerial photographs (Orthophotometry, etc) were used as a modeling aid during the construction of the HEC-RAS model. Information such as approximate river width and length of reach were validated. Other information including structures and vegetative cover were obtained using the imagery.

Site Photographs
Photographs were taken of the site and surrounding area (bridges, structures, dam, and the river channel). Photographs collected during the site visit were referenced during the development of the model and included factors such as channel geometry and vegetative cover. In addition, model results for high flow conditions were compared with photographs collected during the spring storms of 2008 and 2010.

NHDOT Information
Upstream bridge geometry was obtained from the original construction drawings for the Route 108 river crossing. The information was obtained directly from the NHDOT archived records.

Fish Ladder Plans
While the Fish Ladder plans provided by the NHDES were not available until after the model analysis was complete, field collected geometry of the fish ladder was incorporated in the model cross sections. If additional model runs are required, the NHDES fish ladder plans will be used to verify the current model geometry,.

HEC-RAS Model Development

The HEC-RAS model for the dam breach analyses was developed using a combination of the aforementioned data. Figure 1 shows the HEC-RAS model cross section locations. Model input parameters and geometry of specific physical features are summarized below:

The Macallen Dam
Information regarding the dam and spillway location, dimensions and depth/geometry were obtained from field measurements. Construction drawings were not available during generation of the model.

Bridges
The upstream bridge was measured in the field and cross referenced with the original NHDOT drawing set. The following bridge data was used in the HEC-RAS model:

- Low cord elevations: measurement from the bottom of the channel to the underside (lowest section) of the bridge.
- High cord elevations: obtained by adding the thickness of the bridge to the low cord elevation.
- Width of the deck: measured at widest distance.
- Length of the deck: measured at longest distance.
- Abutment shape/design: when applicable, measurements of the abutments were obtained using the smallest potential flow opening as observed from upstream.

- Weir Coefficients: HEC-RAS default condition was utilized.

Roughness Coefficients (Manning's "n"):
Values for the channel geometry roughness coefficients (channel bottom, banks, and surrounding area outside of banks) were obtained from *Open-Channel Hydraulics*; Ven Te Chow, Ph.D., 1959. The selection of roughness coefficients were based on field observations and compared to literature values. The value of river roughness coefficients are affected by bottom geometry, lining, and slope.

Model Limits
The upstream limit of the model is located 750 feet upstream of the dam (500 feet upstream of the Route 108 Bridge). The downstream limit of the model is located 1,200 feet downstream of the dam (near the boat launch).

River Flow Rate
As discussed earlier, the "Sunny Day" flow and 100-year flood flow rates through the dam were determined to be 272 cfs and 8,302 cfs, respectively. These flows were used in the model for the breach analyses.

Boundary Conditions
Upstream and downstream boundary conditions were provided for the model. The upstream boundary condition was set as the river flow rate and the downstream boundary condition was set as the low or high tide water surface elevation (8.5 feet or 15.61 feet, respectively).

Breach Parameters
Timeline and final geometry data related to full breach conditions was obtained from *Chapter II, Appendix II-A, Table 1, Suggested Breach Parameters, Selecting and Accommodating Inflow Design Floods for Dams, 1993,* Federal Energy Regulatory Commission. Excerpts from this document are attached in Appendix A and the entire document can be found at the following website:

http://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide/chap2.pdf

The following breach parameters were used in the model and represent a typical breach scenario for the Macallen Dam.

- Crest Length (W): total length across the dam, from bank to bank, was obtained from the HEC-RAS inline structure editor (varied for each condition).
- Width of Breach (BR): equivalent to one half (0.5) the crest length (varied for each condition).
- Horizontal Component of Side Slope of Breach (Z): Zero (i.e. vertical side slope) for a concrete/stone built structure.
- Time to Failure (TFH): Two tenths of an hour (0.2 hour or 12 minutes) for complete failure.
- Breach Progression: A curve linear S-function was created to produce a maximum breach rate (i.e. steeper slope) at the midway point during the breach.

Submerged Dam Upstream

During the April field visit, a submerged dam was noted roughly 40 feet upstream of the Macallen Dam. This dam was modeled in HEC-RAS. Because of the potential for this dam to retain water if the Macallen Dam were to breach, sensitivity analyses were run to determine if a breach of the submerged dam would impact the breach wave inundation. As a result of the sensitivity analysis, it was decided to breach the submerged dam shortly after the breach of the Macallen dam to allow the entire stored volume behind the Macallen dam to contribute to the breach wave inundation.

Macallen Dam HEC-RAS Model Breach Analysis Results

Table 1 outlines the dam breach analyses results for the Macallen Dam.

TABLE 1
DAM BREACH ANALYSIS RESULTS

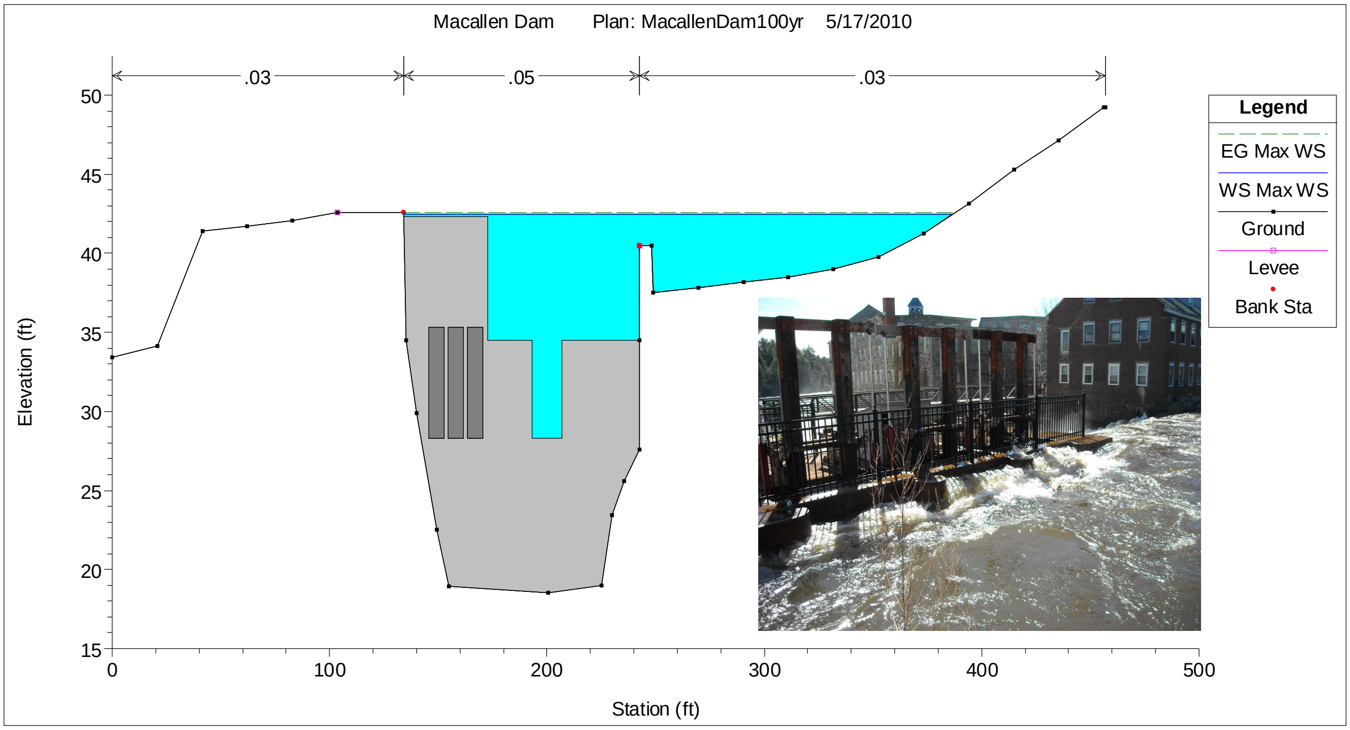
Run No.	Downstream Condition	Flow Rate	Water Surface Elevation (WSE,ft) Upstream of Dam[1]	WSE (ft) Downstream at Lowest Habitable Mill Structure[2] (STA. 0)	Downstream Structure Inundation[3] (Yes/No)	Water Overtopping Bridges (Yes/No)	Breach Surge Timing [4] (min)
1	Low Tide	Sunny Day	35.81	10.84	NO	No	25
2		100-Year	42.54	14.08	NO	No	10
3	High Tide	Sunny Day	35.81	15.84	NO	No	17
4		100-Year	42.54	16.10	NO	No	20

Notes:
1. Water surface elevations upstream of the dam depict conditions with NO GATE OPERATIONS.
2. Lowest habitable structure in the Mill Building immediately downstream of the dam is at elevation 22.61'
3. Structure inundation downstream of mill buildings was identified by visual inspection of the flood inundation maps. It is recommended that additional information be collected to confirm the potential of flooding for these locations.
4. Breach surge timing is time for breach wave to reach boat launch downstream of the dam.

Inundation mapping was developed for each of the four breach analysis model runs. These maps depict the predicted downstream flood water boundaries for each dam breach scenario. The inundation maps were created by exporting the HEC-RAS model data into GIS and are included as Figures 2 through 5. A HEC-RAS plan view, model cross sections, and tabular output data for the 100-year flood flow with high tide model run are included in Appendix B.

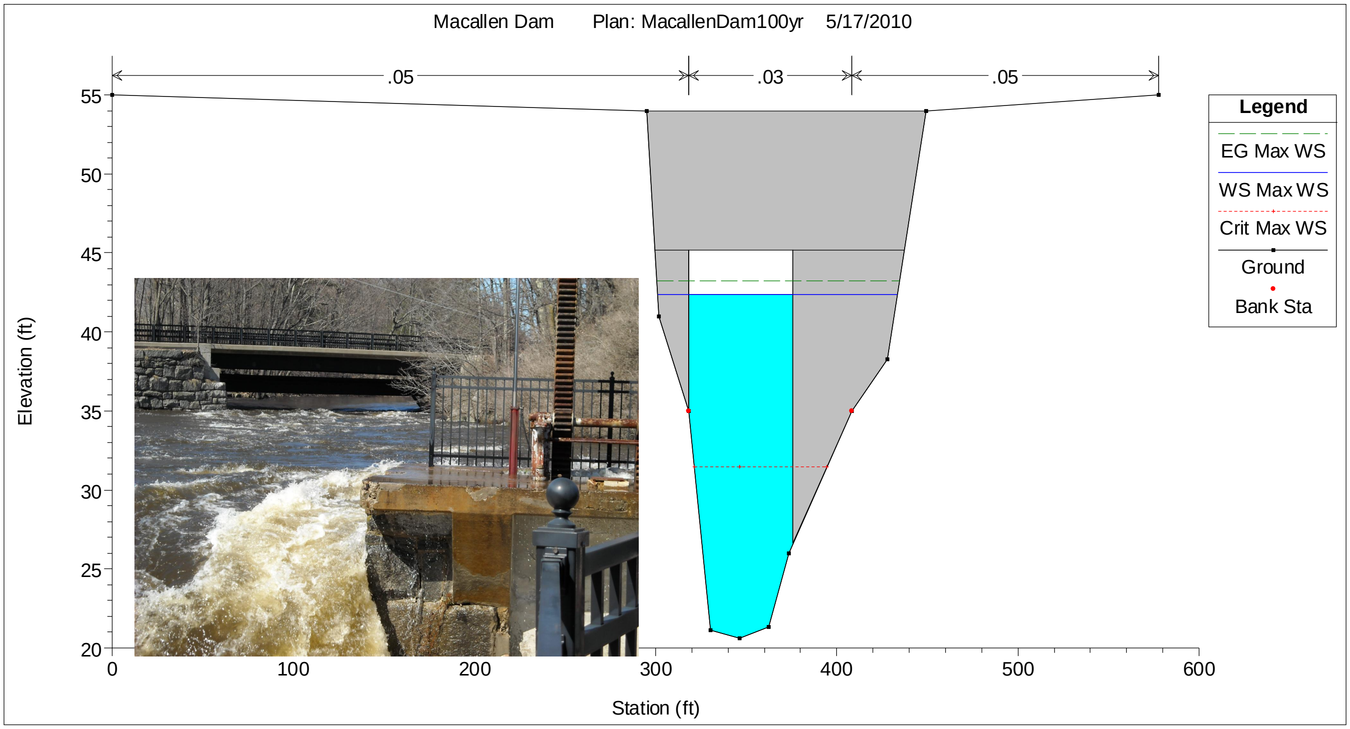
Model results at several locations for the 100-year flood flow runs were compared to photographs collected during the spring storms of 2008 and 2010. For example, the predicted water surface elevations upstream of the dam and at the Route 108 Bridge (without a breach) were compared to a photos taken upstream of the dam and at the Route 108 Bridge during flow conditions very similar to the 100-year flood flow as shown in Figures 6 and 7.

FIGURE 6
MODEL VERSUS OBSERVED WATER SURFACE UPSTREAM OF MACALLEN DAM



As shown in Figure 6, the model is predicting that the left embankment of the dam will be slightly overtopped during the 100-year flood flow. The photograph shows that the left embankment is not overtopped during a similar flood flow. This difference is explained by the fact that the model was run assuming the gates are closed (worst case breach scenario). The gates were open when the photo was taken and it is assumed that the flood flow experienced that day was slightly less then the 100-year flood flow. This accounts for difference between the model and photographed water surface elevations.

FIGURE 7
MODEL VERSUS OBSERVED WATER SURFACE AT ROUTE 108 BRIDGE



In Figure 7, the model is predicting that the water surface elevation at the Route 108 Bridge is roughly 3 feet below the bottom of the bridge. The photograph taken during similar flood flow conditions shows the water surface elevation at the bridge to be between 3 and 3.5 feet below the bottom of the bridge. Overall, the model appears to match photographed water surface elevations throughout the model limits.

It should be noted that boat docking facilities exist downstream of the dam. During sunny day and 100-year flood flow conditions without a dam breach at high tide, the model predicts river velocities between 0.4 and 5.5 feet per second. During breach conditions at high tide, the model predicts that river velocities for sunny day and 100-year flood flow will range from 1.8 to 5.5 feet per second. This minor increase in velocity during a breach should not result in damage to the docking structures or docked boats.

Macallen Dam Breach Analyses Summaries

<u>Classification of Dam</u>
Currently, the Macallen Dam is classified as a High Hazard (Class C) dam. From the NHDES "Classification of Dams in New Hampshire" fact sheet:

> "High Hazard means a dam that has a high hazard potential because it is in a location and of a size that failure or misoperation of the dam would result in probable loss of human life as a result of:

- Water levels and velocities causing the structural failure of a foundation of a habitable residential structure or commercial or industrial structure, which is occupied under normal conditions.
- Water levels rising above the first floor elevation of a habitable residential structure or a commercial or industrial structure, which is occupied under normal conditions when the rise due to dam failure is greater than one foot.
- Structural damage to an interstate highway, which could render the roadway impassable or otherwise interrupt public safety services.
- The release of a quantity and concentration of material, which qualify as “hazardous waste” as defined by RSA 471-A:2 VI.
- Any other circumstance that would more likely than not cause one or more deaths.

Based on the results of the HEC-RAS analyses, it appears that the dam can be reclassified as a Significant Hazard (Class B) dam. From the NHDES "Classification of Dams in New Hampshire" fact sheet:

> "Significant Hazard structure means a dam that has a significant hazard potential because it is in a location and of a size that failure or misoperation of the dam would result in any of the following:
> - No probable loss of lives.
> - Major economic loss to structures or property.
> - Structural damage to a Class I or Class II road that could render the road impassable or otherwise interrupt public safety services.
> - Major environmental or public health losses, including one or more of the following:
>   - Damage to a public water system, as defined by RSA 485:1-a, XV, which will take longer than 48 hours to repair.
>   - The release of liquid industrial, agricultural, or commercial wastes, septage, sewage, or contaminated sediments if the storage capacity is 2 acre-feet or more.
>   - Damage to an environmentally-sensitive site that does not meet the definition of reversible environmental losses.

The current HEC-RAS analysis shows that each of the conditions for a Significant Hazard classification are met. In addition, water levels during a breach would not rise above the first floor elevation of a habitable residential structure or a commercial or industrial structure. It is recommended that the Town consider petitioning the NHDES to reclassify the dam from a High Hazard structure to a Significant Hazard structure.

If the Town is successful in obtaining a reclassification of the dam from High Hazard to Significant Hazard, the inspection interval required by NHDES would be decreased from once every year to once every two years.

Emergency Action Plan Update

The 100-year flood flow with high tide conditions model run results in the highest water surface elevations both upstream and downstream of the dam during a breach. Therefore, it is recommended to use the inundation mapping presented in Figure 5 to update the Emergency Action Plan.

Spillway Capacity

As shown in Figure 6, the model is predicting that the dam will be slightly overtopped during 100-year flood flow conditions with the gates closed. With the gates open and flow conditions approaching the 100-year flood, the photo shows that the dam is operating with between two and four inches of freeboard.

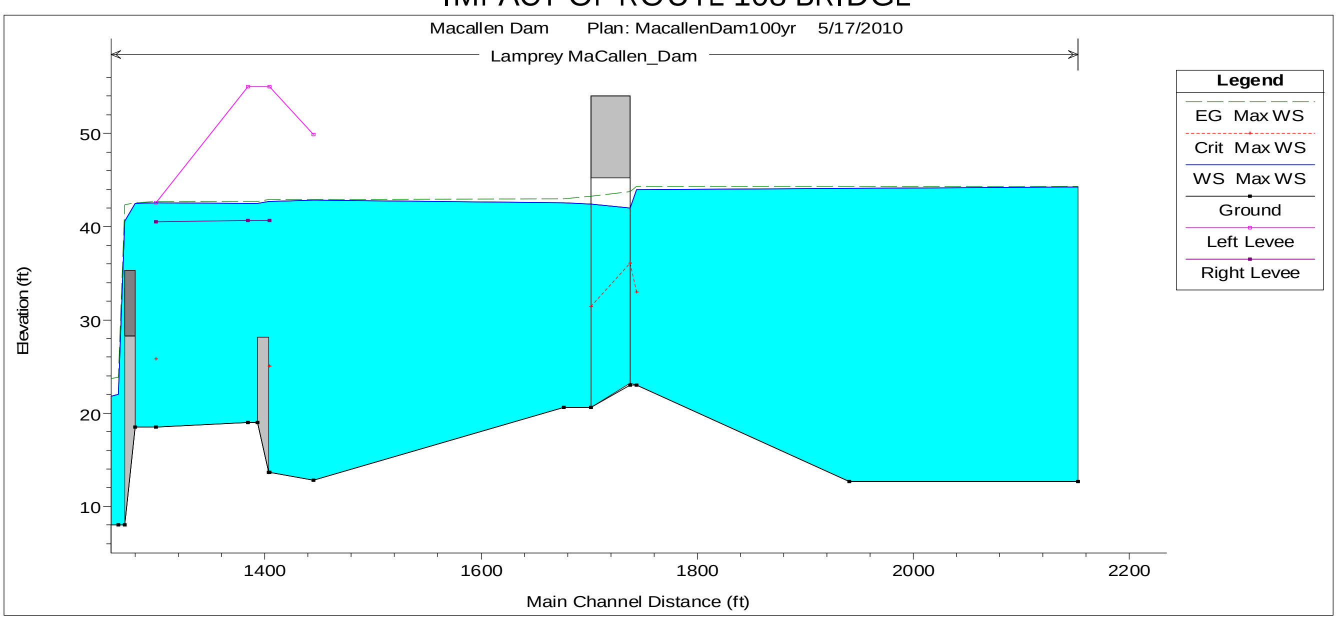
The NHDES requires that High Hazard dams be able to pass 250% of the 100-year flood flow, or at the owner's option, the site specific inflow design flood with at least one foot of freeboard. For Significant Hazard dams, the NHDES requires the dam to be able to pass the 100-year flood flow, or at the owner's option, the site specific inflow design flood.

If the dam continued to be classified as a High Hazard dam, significant structural improvements would be required in order for the dam to be able to pass 250% of the 100-year flood flow with one foot of freeboard. If the dam were to be reclassified as a Significant Hazard dam, structural improvements would be required to be able to pass the 100-year flood flow with one foot of freeboard. It is anticipated that the improvements required to accommodate the 100-year flood flow would be minor when compared to the improvements required to pass 250% of the 100-year flood flow.

Impact of Route 108 Bridge

Figure 8 shows the modeled 100-year flood flow water surface elevation profile from the Macallen Dam (left) to the Route 108 Bridge (center). Based on the model results, the bridge acts as a minor constriction point during the 100-year flood flow. The profile shows between a 1 and 1.5 foot drop in water surface elevation through the bridge structure. If the bridge were not a constriction, zero feet of drop in water surface elevation would occur through the dam. It should be noted that overtopping of the bridge is not a likely scenario during a 100-year flood flow.

FIGURE 8
IMPACT OF ROUTE 108 BRIDGE



Recommendations

Based on the findings of the breach analyses, Wright-Pierce recommends the following actions:

- The Town should present the information presented in this letter to the NHDES to begin discussions regarding reclassification of the dam. Wright-Pierce would be happy to assist the Town with NHDES coordination.

- While the model results appear to mimic conditions observed in the field, it should be noted that a formal field survey was not completed. It is recommended that survey elevations be collected in the field to verify the model geometry.

We appreciate this opportunity to assist the Town of Newmarket with these analyses. We look forward to meeting with you at your convenience to review this draft report and to discuss our recommendations with you. Please feel free to contact us at 430-3728 with any questions or comments you may have.

Very truly yours,
WRIGHT-PIERCE

Ryan T. Wingard, P.E.
Senior Project Manager

Richard N. Davee, P.E.
Vice President

Attachments

cc: Julie Glover, Newmarket Project Coordinator (w/attachments)
Rick Malasky, Newmarket Director of Public Work (w/attachments)

Town of Newmarket
Macallen Dam Breach Analysis

Site Map

Newmarket, New Hampshire

PROJ NO: 12018A	DATE: May 2010	FIGURE: 1

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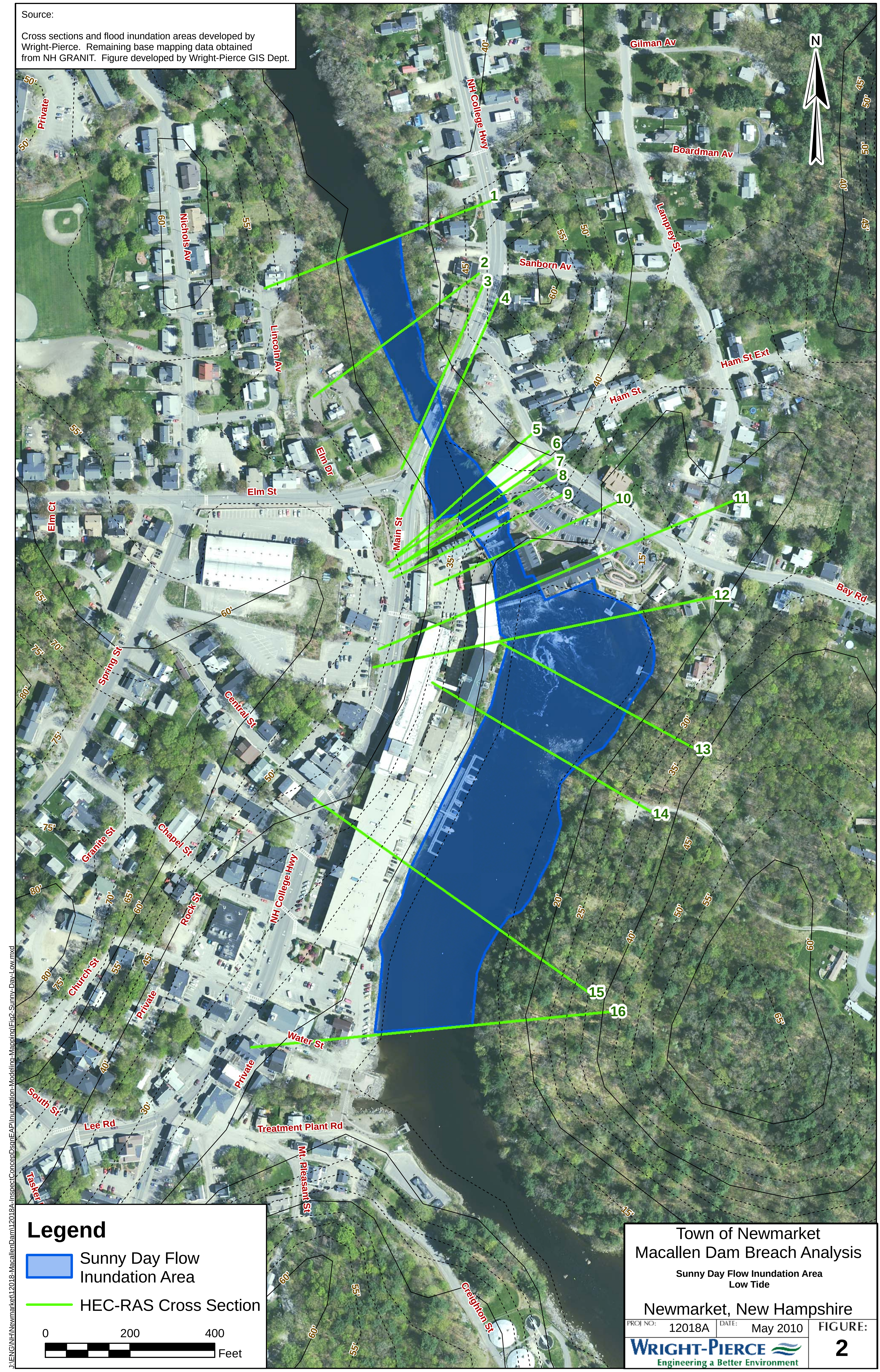
Source:
Cross sections and flood inundation areas developed by Wright-Pierce. Remaining base mapping data obtained from NH GRANIT. Figure developed by Wright-Pierce GIS Dept.
N
Gilman Av
Boardman Av
NH College Hwy
Lamprey St
Sanborn Av
Nichols Av
Lincoln Av
Private
Ham St Ext
Ham St
Elm Dr
Elm St
Elm Ct
Main St
Bay Rd
Spring St
Central St
Granite St
Chapel St
Rock St
Church St
Water St
South St
Lee Rd
Treatment Plant Rd
Mt. Pleasant St
Tasker
Creighton St
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Legend
Sunny Day Flow Inundation Area
HEC-RAS Cross Section
0
200
400
Feet
Town of Newmarket
Macallen Dam Breach Analysis
Sunny Day Flow Inundation Area
Low Tide
Newmarket, New Hampshire
PROJ NO: 12018A
DATE: May 2010
FIGURE: 2
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Engineering a Better Environment
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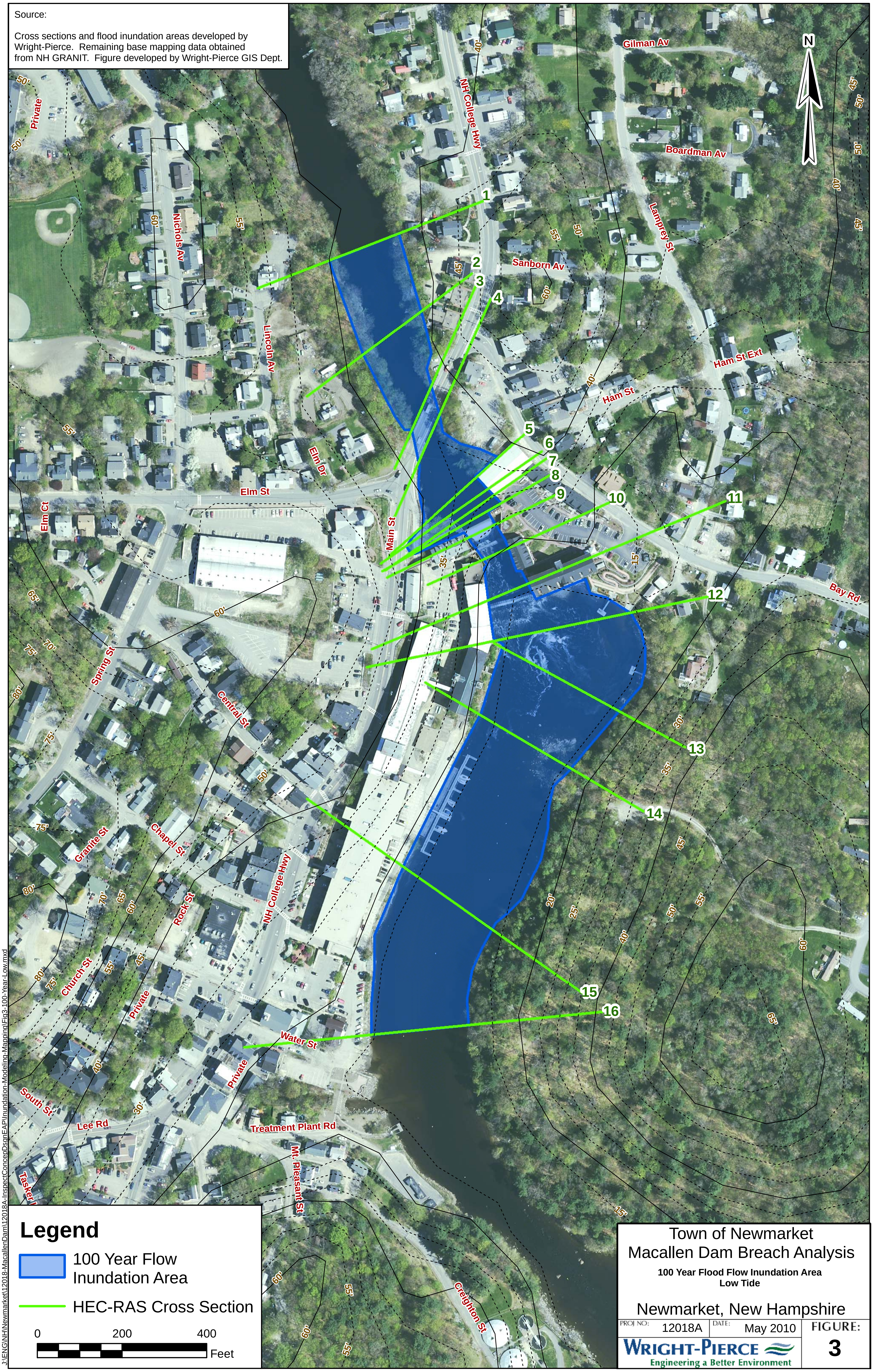
Source:

Cross sections and flood inundation areas developed by Wright-Pierce. Remaining base mapping data obtained from NH GRANIT. Figure developed by Wright-Pierce GIS Dept.



Legend

100 Year Flow Inundation Area

HEC-RAS Cross Section

0 200 400 Feet

Town of Newmarket
Macallen Dam Breach Analysis

100 Year Flood Flow Inundation Area
Low Tide

Newmarket, New Hampshire

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WRIGHT-PIERCE
Engineering a Better Environment

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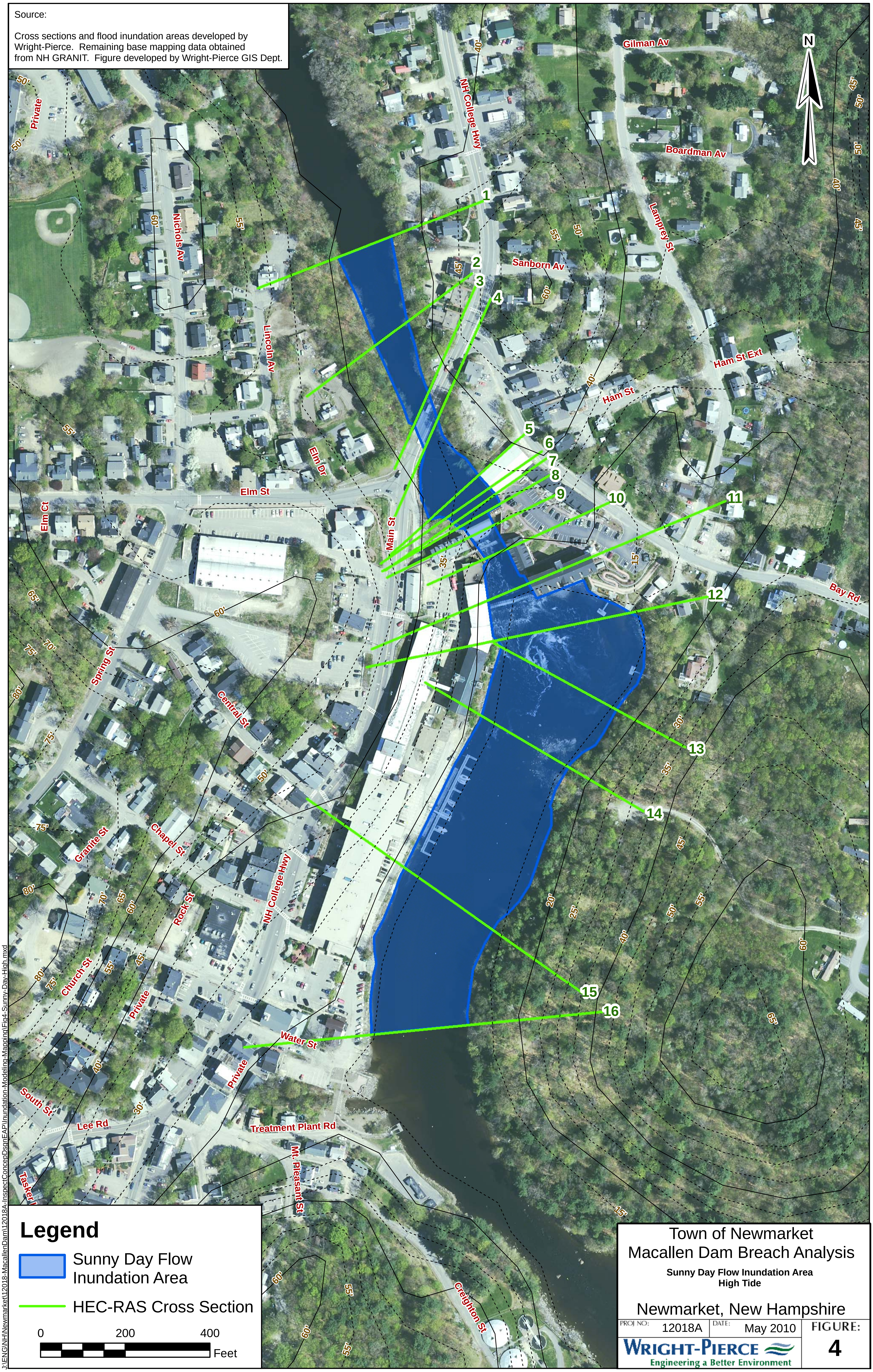

Source:

Cross sections and flood inundation areas developed by Wright-Pierce. Remaining base mapping data obtained from NH GRANIT. Figure developed by Wright-Pierce GIS Dept.

Legend

Sunny Day Flow Inundation Area

HEC-RAS Cross Section

0 200 400 Feet

Town of Newmarket
Macallen Dam Breach Analysis

Sunny Day Flow Inundation Area
High Tide

Newmarket, New Hampshire

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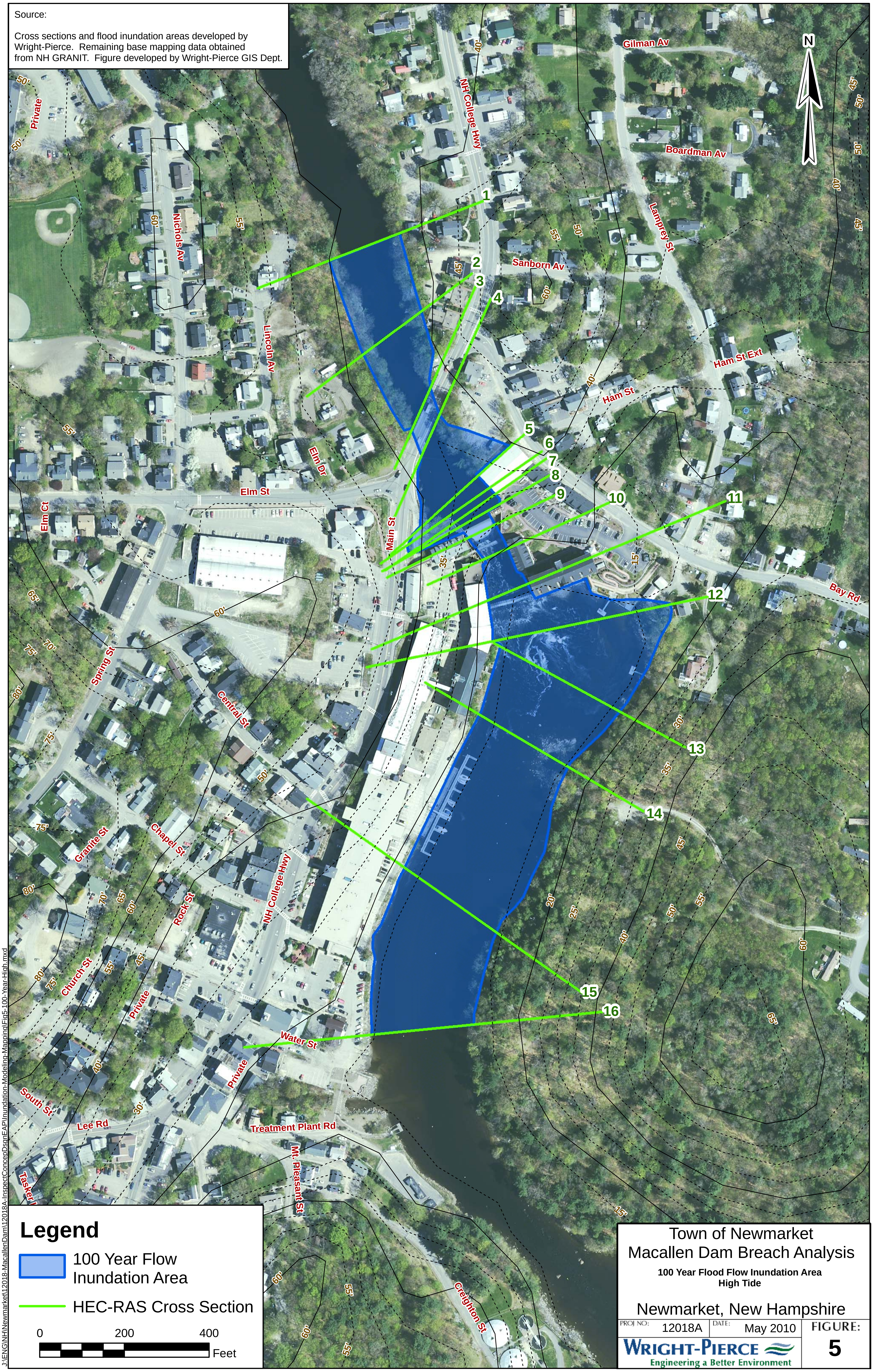

Town of Newmarket
Macallen Dam Breach Analysis

100 Year Flood Flow Inundation Area
High Tide

Newmarket, New Hampshire

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Engineering a Better Environment

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Appendix A
NHDES Dambreak Studies

Appendix II-A

DAMBREAK STUDIES

October 1993

APPENDIX II-A

Dambreak Studies

The evaluation of the downstream consequences in the event of a dam failure is a main element in determining hazard potential and formulating emergency action plans for hydroelectric projects. The solution requires knowledge of the lateral and longitudinal geometry of the stream, its frictional resistance, a discharge-elevation relationship at one boundary, and the time-varying flow or elevation at the opposite boundary.

The current state-of-the-art is to use transient flow or hydraulic methods to predict dambreak wave formation and downstream progression. The transient flow methods solve and therefore account for the essential momentum forces involved in the rapidly changing flow caused by a dambreak. Another technique, referred to as storage routing or the hydrologic method, solves one-dimensional equations of steady flow ignoring the pressure and acceleration contributions to the total momentum force. For the same outflow hydrograph, the storage routing procedures will always yield lower water surface elevations than hydraulic or transient flow routing.

When routing a dambreak flood through the downstream reaches appropriate local inflows should be included in the routing which are consistent with the assumed storm centering.

The mode and degree of dam failure involves considerable uncertainty and cannot be predicted with acceptable engineering accuracy; therefore, conservative failure postulations are necessary. Uncertainties can be circumvented in situations where it can be shown that the complete and sudden removal of a dam (or dams) will not endanger human life or cause significant property damage.

The following provides references on dambreak analyses and criteria which may prove useful as indicators of reasonableness of the breach parameters, peak discharge, depth of flow, and travel time determined by the licensee. In addition, **Section 6-2 and Appendix VI-C of Chapter VI of these Guidelines provides additional criteria on analytical requirements for dambreak analyses.**

I. REFERENCES

Suggested acceptable references regarding dam failure studies include the following:

A. Fread, D. L. "DAMBRK - The NWS Dam-Break Flood Forecasting Model," National Weather Service, Silver Spring, Maryland, 1988 Version. This (or

the most recent version) is the preferred method for performing dambreak studies.

B. Fread, D. L. "NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction", Proceedings, Association of State Dam Safety Officials, 10th Annual Conference, Kansas City, Missouri, September 26-29, 1993. ***Since this model combines the NWS DAMBRK model and the NWS DWOPER model, it is also considered the preferred method.***

C. Westmore, Jonathan N. and Fread, Danny L., "The NWS Simplified Dam-Break Flood Forecasting Model," National Weather Service, Silver Spring, Maryland, 1981. (Copy previously furnished to each Regional Office with a detailed example).

D. Fread, D. L., 1977: The development and testing of a dam-break flood forecasting model, "Proceedings, Dam-Break Flood Modeling Workshop," U.S. Water Resources Council, Washington, D.C., 1977, pp. 164-197.

E. Hydrologic Engineering Center, "Flood Hydrograph Package (HEC-1) Users Manual for Dam Safety Investigations," September, 1990.

F. Gandlach, D. L. and Thomas, W. A., "Guidelines for Calculating and Routing a Dam-Break Flood," Research Note No. 5, U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1977.

G. Cecilio, C. B. and Strassburger, A. G., "Downstream Hydrograph from Dam Failure," Engineering Foundation Conference on Evaluation of Dam Safety, 1976.

H. Soil Conservation Service, "Simplified Dam-Breach Routing Procedure," March 1979. (To be used only for flood routing technique, not dambreak discharge).

I. Chow, V. T., <u>Open Channel Hydraulics</u>, McGraw-Hill Book Company, Inc., New York, 1959, Chapter 20.

J. Henderson, F. M., <u>Open Channel Flow</u>, McMillan Company, New York, 1966, Chapters 8 and 9.

K. Hydrologic Engineering Center, "Flood Emergency Plans, Guidelines for Corps Dam," June 1980. (Forwarded to all Regional Engineers by memorandum dated February 11, 1981).

L. Hydrologic Engineering Center, "UNET, One-Dimensional Unsteady Flow Through a Full Network of Open Channels", September 1992.

II. CRITERIA

The following criteria may prove useful as an indicator of the reasonableness of a dambreak study:

A. If the dambreak analysis has been performed by an acceptable method (**References A and B are the preferred methods**), then generally only the breach parameters, peak discharge, and flood wave travel time should be <u>verified</u> as an indicator of the licensee's correct application of the method selected. Downstream routing parameters (i.e., Manning's "n") should be reviewed for acceptability and inundation maps should be reviewed for clarity and completeness of information (i.e., travel times). The following criteria are considered to be adequate and appropriate for verifying the selected breach parameters and peak discharge:

1. Breach Parameters - Most serious dam failures result in a situation resembling weir conditions. Breach width selection is judgmental and should be made based on the channel or valley width with failure occurring at the deepest section. The bottom of the breach should generally be assumed to be at the foundation elevation of the dam. Pages 2-A-8 through 2-A-11 of this appendix contain suggested breach parameters and should be used when verifying the selected breach parameters. For worst case scenarios, the breach width should be in the upper range while the time of failure should be in the lower range. However a sensitivity analysis is recommended to determine the reasonableness of the assumptions.

2. Peak Discharge - The peak discharge may be verified by use of equations (11) and (13) of Reference No. 1. Although the equations assume a rectangular-shaped breach, a trapezoidal breach may be analyzed by specifying a rectangular breach width that is equal to the average width of the trapezoidal breach.

<u>Equation 11</u>:

$$C = \frac{23.4 A_s}{\overline{BR}}$$

Where: C = constant
A_s = reservoir surface area, in acres
$\overline{BR}$ = average breach width, in feet

Equation 13:

$$Q_{bmax} = 3.1\overline{BR}\left(\frac{C}{\left(t_f + \frac{C}{\sqrt{H}}\right)}\right)^3$$

Where: Q_{bmax} = maximum breach outflow, in cfs
t_f = time of failure, in hours
H = maximum head over the weir, in feet

This equation for Q_{bmax} has been found to give results within +5% of the Q_{peak} from the full DAMBRK model.

In a rare case where a dam impounding a small storage volume has a large time of failure, the equations above will predict a much higher flow than actually occurs.

At a National Weather Service Dam-Break Model Symposium held in Tulsa, Oklahoma, June 27-30, 1983, Dr. Danny Fread presented an update to his simplified method. Equation 13 has been modified as follows to include additional outflow not attributed to breach outflow:

$$Q_{bmax} = Q_o + 3.1\overline{BR}\left(\frac{C}{\left(t_f + \frac{C}{\sqrt{H}}\right)}\right)^3$$

Where: Q_o = Additional (non-breach) outflow (cfs) at time t_f (i.e., spillway flow and/or crest overflow) (optional data value, may be set to 0).

This equation has also been modified to address instantaneous failure, because in some situations where a dam fails very rapidly, the negative wave that forms in the reservoir may significantly affect the outflow from the dam.

3. Flood Wave Travel Time - Reasonableness of the flood wave travel time may be determined by use of the following "rule-of-thumb" approximation for average wave speed:

(a) Assume an equivalent rectangular channel section for the selected irregular channel section.

(b) Assume a constant average channel slope.

(c) Compute depth of flow from the following adjusted Manning's equation.

$$d=\left(\frac{Qn}{1.46B(S)^{0.5}}\right)^{0.6}$$

Where: d = depth of flow for assumed rectangular section, ft.
Q = peak discharge, cfs
B = average width (rectangular), ft.
S = average slope, ft./ft.
n = Manning's roughness coefficient

(d) Compute average velocity from Manning's Equation:

$$V=\frac{1.49(S)^{0.5}(d)^{0.67}}{n}$$

Where: V = average velocity, fps

(e) Compute wave speed, C (Kinematic velocity):

$$C=\frac{5}{3}V(0.68)$$

Where: C = wave speed (mph)

Note: *1 fps = 0.68 mph*

(f) Determination travel time, TT

$$TT = \frac{X}{C}$$

Where: TT = travel time, hr.
X = distance from dam, mi.

Note: *If the slope is flat, the following "rule-of-thumb" provides a very rough estimate of the wave speed:*

$$C = 2(S)^{0.5}$$

Where: C = wave speed, mph
S = average slope, ft./mi.

In addition, as a "rule-of-thumb", the dynamic routing (NWS) method should be used whenever severe backwater conditions at downstream areas occur and/or the slope is less than 20 ft/mi. When these restrictions are not present normal hydrologic routing (HEC-1) may provide reasonable results. It is recommended that HEC-2 be used to determine the resulting water surface elevations when HEC-1 is used for the dambreak study.

<u>The HEC-I Manual (Reference E) states that when "a higher order of accuracy is needed, then an unsteady flow model, such as the National Weather Service's DAMBRK should be used."</u> Experience demonstrates that the higher order of accuracy is usually required. Therefore, the NWS DAMBRK model and the more recent NWS FLOODWAV model are the preferred methods and recommended for all situations requiring dambreak studies.

B. If a dambreak analysis has been performed by a method other than one of the suggested acceptable methods, the selected breach parameters, peak discharge, depth of flow and travel time of the flood wave shall be <u>verified</u> by one of the two methods:

1. **Unsteady Flow - Dynamic Routing Method (Recommended)**

The NWS "DAMBRK" Model (Reference A) and the NWS "FLOODWAV" Model (Reference B) are the recommended methods. Each FERC Regional Office has received the software using the NWS DAMBRK program and should use this program, as necessary, to verify dambreak studies. As the flood wave travels downstream, the peak discharge and wave velocity generally, but not always, decrease. This attenuation in the flood wave is primarily due to energy dissipation when it is near the dam and to valley storage as it progresses in an unsteady flow downstream. It is important that the NWS model be calibrated to historical floods, if at all possible.

2. **Steady Flow Method (Provides a rough estimate)**

If this method is selected, the breach parameters and peak discharge shall be verified as in part "A" above. The method described below should be utilized only for preliminary assessments and the obtained values may be far from the actually expected results. Sound judgement and extensive numerical experience is necessary when evaluating the results.

For a rough estimate of the travel time and flood wave, it is recommended that one of the following two steady state methods be used for verification of the licensee's values:

a. When steam gage data are available, the depth of flow and travel time can be estimated as follows (This method will indirectly take valley storage into consideration):

(1) Identify existing stream gages located downstream of the dam.

(2) Obtain the stage-discharge curve for each gage.

(3) Assuming Q_{peak} remains constant, extrapolate the curves to the Q_{peak} value of the flood wave and determine the corresponding water surface elevation.

(4) Using the continuity equation to determine the velocity, estimate the travel time between each cross-section.

b. When stream gage data is not available, the depth of flow and travel time can be estimated based on the following steady-state method:

(1) Assume the area downstream of the dam is a channel. This will neglect valley storage.

(2) Identify on topographic maps all abrupt changes in channel width and/or slope. Using this as a basis, select and plot channel cross-sections.

(3) Assume Q_{bmax} remains constant throughout the entire stream length under consideration.

(4) Selecting a fairly rough Manning's n value, determine the depth of flow by applying Manning's equation to each cross-section. Assume the energy slope is equal to the slope of the channel.

(5) Using the continuity equation to determine the velocity, estimate the travel time between each cross-section.

C. The above criteria for breach parameters, peak discharge, depth of flow, and travel time should provide the necessary "ballpark figures" needed for comparison with licensee's estimates. When large discrepancies in compared values exist, or questions arise about assumptions to be made, or it appears that an extensive review will be necessary, the Regional Director should contact the Washington Office, D2SI for guidance. The methodology used by the licensee should be a part of the study and should be requested if not included.

TABLE 1

SUGGESTED BREACH PARAMETERS

(Definition Sketch Shown in Figure 1)

Parameter	Value	Type of Dam
Average width of Breach (BR) (See Comment No. 1)*	$\overline{BR}$ = Crest Length	Arch
	$\overline{BR}$ = Multiple Slabs	Buttress
	$\overline{BR}$ = Width of 1 or more Monoliths, Usually $\overline{BR} \leq 0.5$ W	Masonry, Gravity
	HD $\leq \overline{BR} \leq$ 5HD . . . . (usually between . . . . 2HD & 4HD)	Earthen, Rockfill, Timber Crib
	$\overline{BR} \geq$ 0.8 x Crest Length	Slag, Refuse
Horizontal Component of Side Slope of Breach (Z) (See Comment No. 2)*	$0 \leq Z \leq$ slope of valley walls . . .	Arch
	Z = O	Masonry, Gravity Timber Crib, Buttress
	$¼ \leq Z \leq 1$	Earthen (Engineered, Compacted)
	$1 \leq Z \leq 2$	Slag, Refuse (Non-Engineered)
Time to Failure (TFH) (in hours) (See Comment No. 3)*	TFH $\leq$ 0.1	Arch
	0.1 $\leq$ TFH $\leq$ 0.3	Masonry, Gravity, Buttress
	0.1 $\leq$ TFH $\leq$ 1.0	Earthen (Engineered, Compacted) Timber Crib
	0.1 $\leq$ TFH $\leq$ 0.5	Earthen (Non Engineered Poor Construction)
	0.1 $\leq$ TFH $\leq$ 0.3	Slag, Refuse

Definition:

HD	-	Height of Dam
Z	-	Horizontal Component of Side Slope of Breach
BR	-	Average Width of Breach
TFH	-	Time to Fully Form the Breach
W	-	Crest Length

Note: See Page 2-A-12 for definition Sketch

**Comments: See Page 2-A-10 - 2-A-11*

Comments:

1. $\overline{BR}$ is the <u>average</u> breach width, which is not necessarily the bottom width. $\overline{BR}$ is the bottom width for a rectangle, but $\overline{BR}$ is not the bottom width for a trapezoid.

2. Whether the shape is rectangular, trapezoidal, or triangular is not generally critical if the average breach width for each shape is the same. What is critical is the assumed average width of the breach.

3. Time to failure is a function of height of dam and location of breach. Therefore, the longer the time to failure, the wider the breach should be. Also, the greater the height of the dam and the storage volume, the greater the time to failure and average breach width will probably be. Time to failure is the time from the start of the breach formation until the complete breach is formed. It does not include the time leading up to the start of the breach formation. For example, the time to erode away the downstream slope of an earth dam is not included. In this situation, the time to failure commences after sufficient erosion of the downstream slope has occurred and actual formation of the breach (the lowering of the crest) has begun.

4. The bottom of the breach should be at the foundation elevation.

5. Breach width assumptions should be based on the type of dam, the height of dam, the volume of the reservoir, and the type of failure (e.g. piping, sustained overtopping, etc.). Slab and buttress dams require sensitivity analyses that vary the number of slabs assumed to fail.

6. For a worst-case scenario, the average breach width should be in the upper portion of the recommended range, the time to failure should be in the lower portion of the range, and the Manning's "n" value should be in the upper portion of the recommended range. In order to fully evaluate the impacts of a failure on downstream areas, a sensitivity analysis is required to estimate the confidence and relative differences resulting from varying assumptions.

    a. To compare relative differences in peak elevation based on variations in breach widths, the sensitivity analysis should be based on the following assumptions:

        1. Assume a probable (reasonable) maximum breach width, a probable minimum time to failure, and a probable maximum Manning's "n" value. Manning's "n" values for sections immediately below the dam and up to several thousand feet or more downstream of the dam should be assumed to be larger than the maximum value suggested by field investigations in

order to account for uncertainties of high energy losses, velocities, turbulence, etc., resulting from the initial failure.

2. Assume a probable minimum breach width, a probable maximum time to failure, and a probable minimum Manning's "n" value.

Plot the resulting water surface elevation at selected locations downstream from the dam for each run on the same graph. Compare the differences in elevation with respect to distance downstream from the dam for the two cases.

b. To compare differences in travel time of the flood wave, the sensitivity analysis should be based on the following assumptions:

1. Use criteria in a. 1.

2. Assume a probable maximum breach width, a probable minimum time to failure, and a probable minimum Manning's "n" value.

Plot the results (elevation-distance downstream) of both runs on the same graph to compare the changes in travel time with respect to distance downstream from the dam.

c. To compare differences in elevation between natural flood conditions and natural flood conditions plus dambreak, the sensitivity analysis should be based on the following assumptions:

1. Route natural flood without dambreak assuming maximum probable Manning's "n" value.

2. Use criteria in a. 1.

Plot the results (elevation-distance downstream) of both runs on the same graph to compare the changes in elevation with respect to distance downstream from the dam.

7. When dams are assumed to fail from overtopping, wider breach widths than those suggested in Table 1 should be considered if overtopping is sustained for a long period of time.

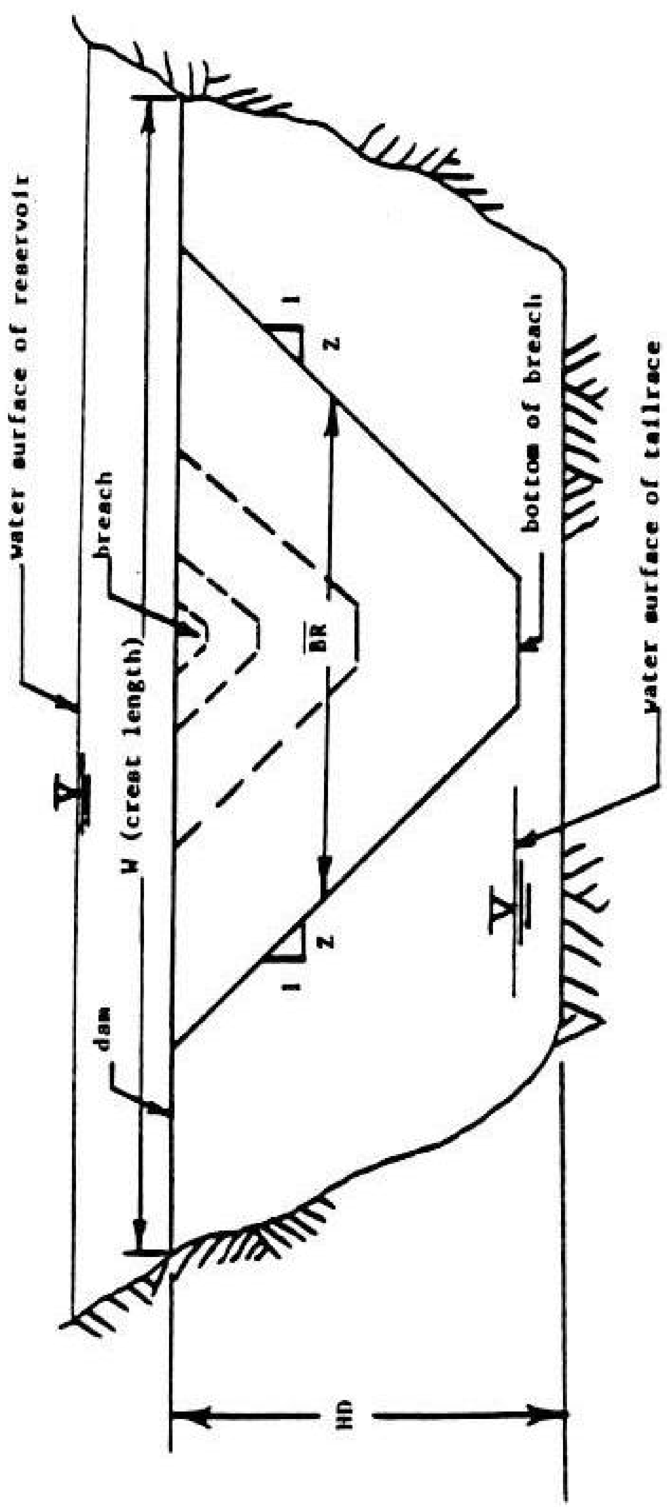
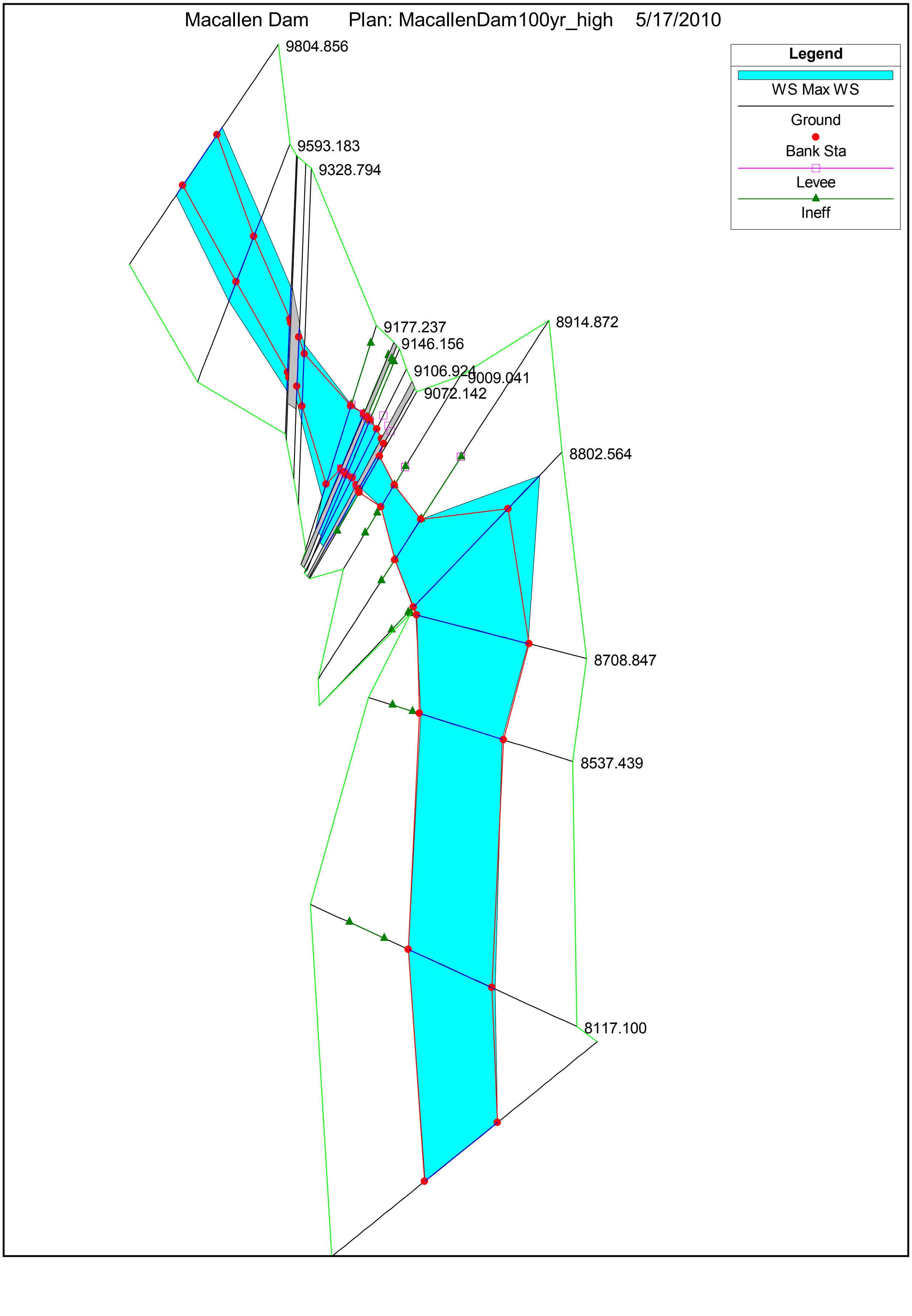
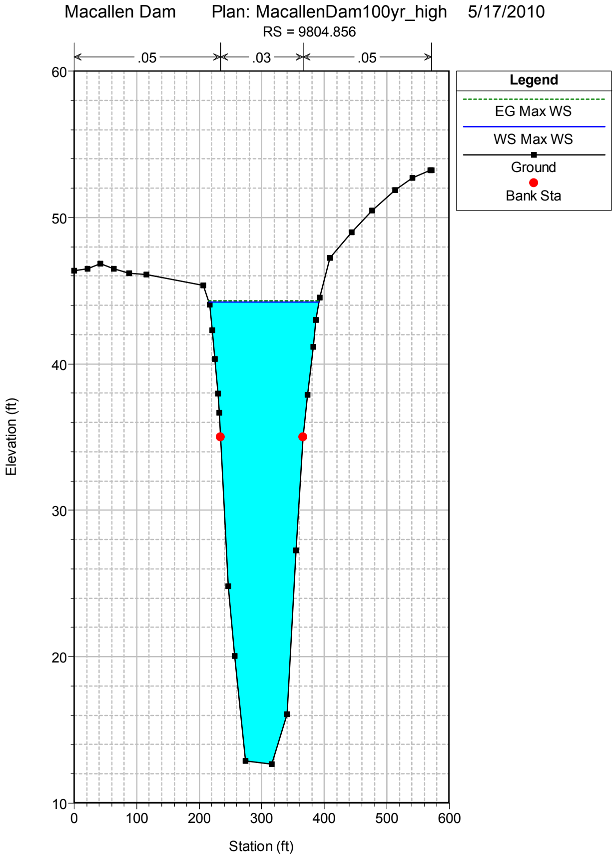


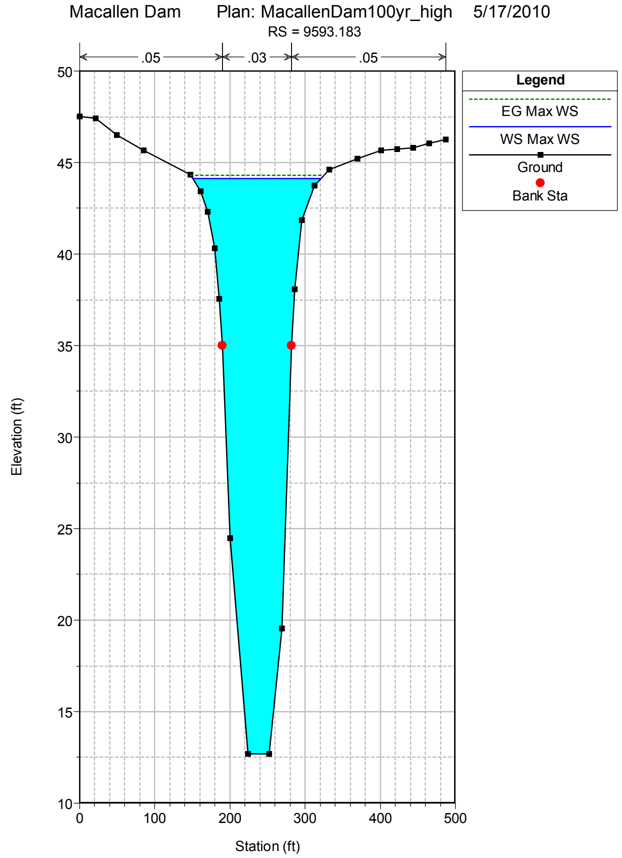
FIGURE 1. DEFINITION SKETCH OF BREACH PARAMETERS

Appendix B
HEC-RAS Output

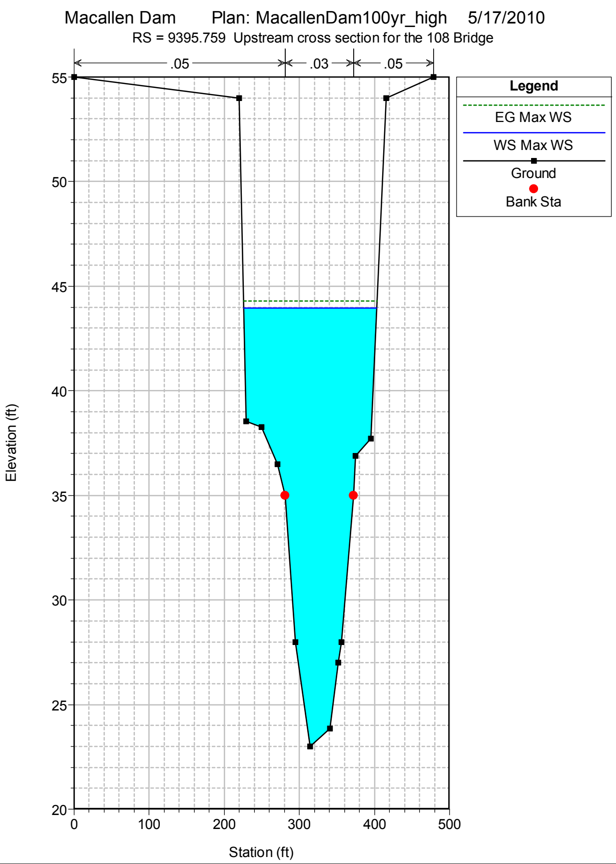

Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
Legend
WS Max WS
Ground
Bank Sta
Levee
Ineff
9804.856
9593.183
9328.794
9177.237
9146.156
9106.924
9072.142
9009.041
8914.872
8802.564
8708.847
8537.439
8117.100



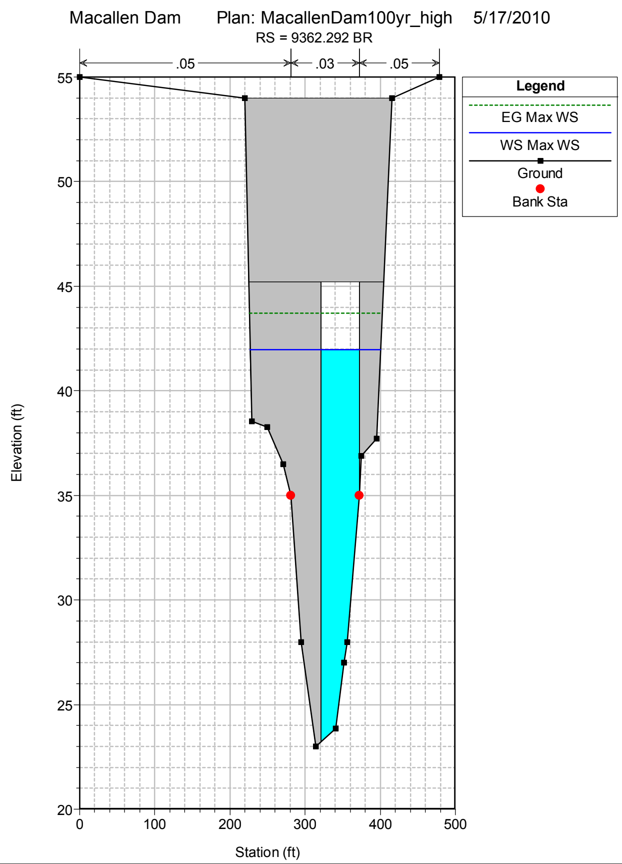
Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 9804.856
.05
.03
.05
Legend
EG Max WS
WS Max WS
Ground
Bank Sta
Elevation (ft)
Station (ft)



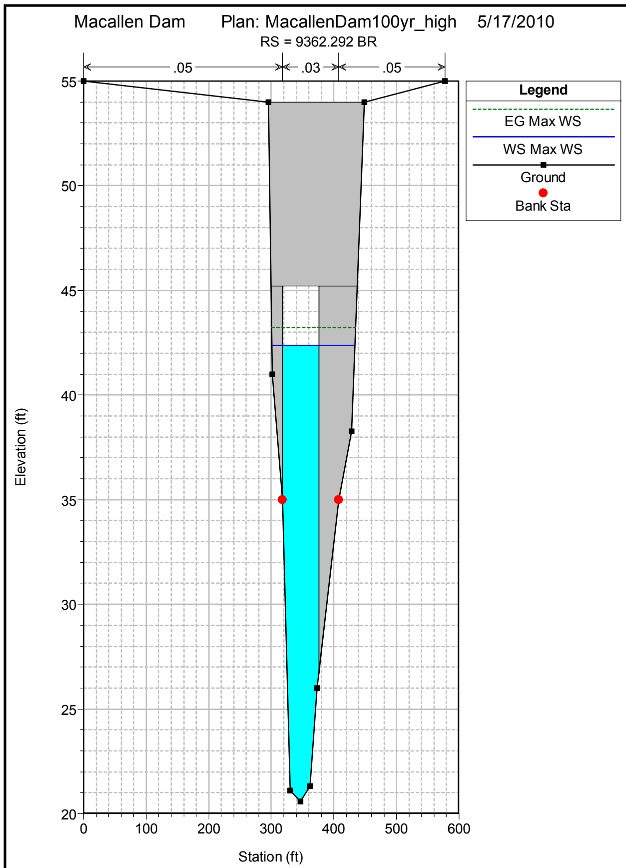
Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 9593.183
.05
.03
.05
Legend
EG Max WS
WS Max WS
Ground
Bank Sta
Elevation (ft)
Station (ft)

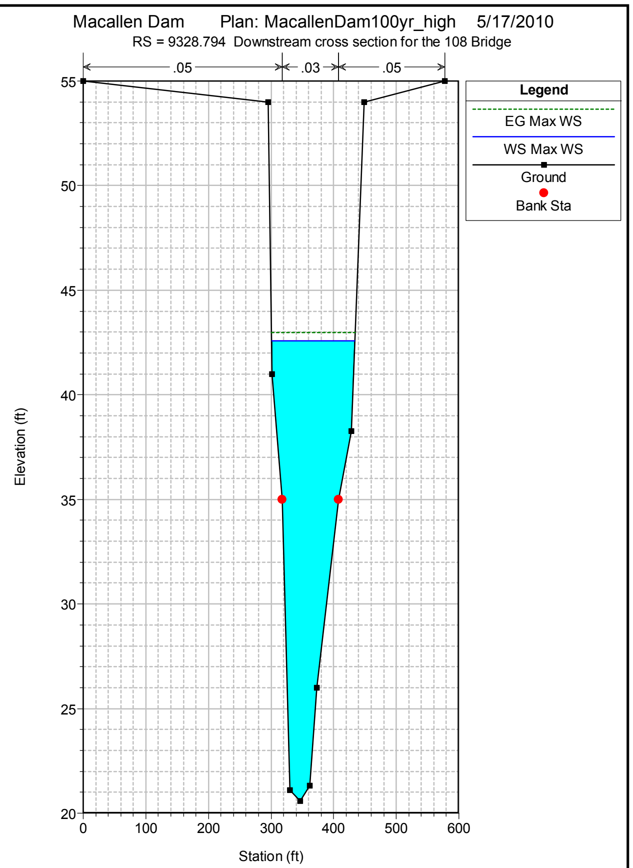


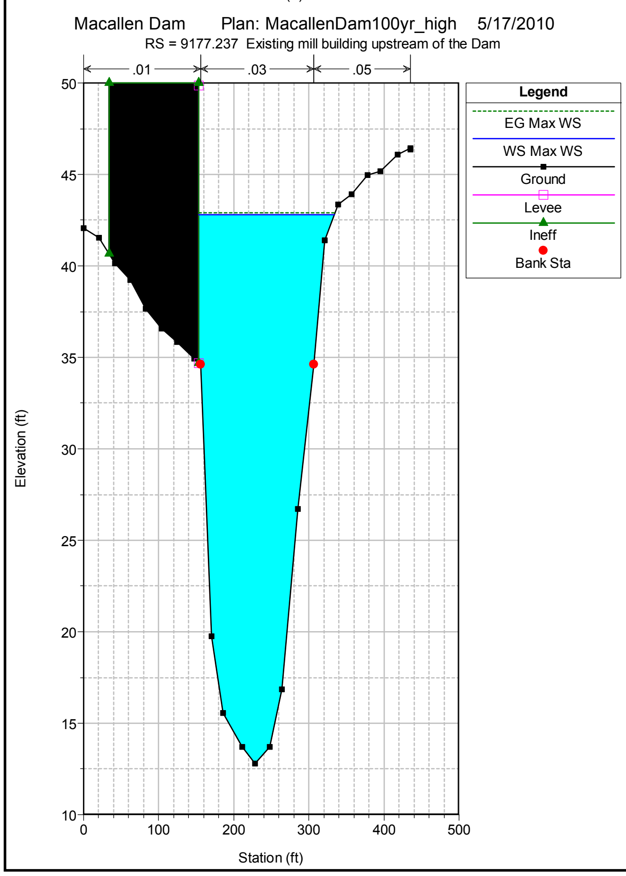
Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 9395.759 Upstream cross section for the 108 Bridge
.05
.03
.05
Legend
EG Max WS
WS Max WS
Ground
Bank Sta
Elevation (ft)
Station (ft)

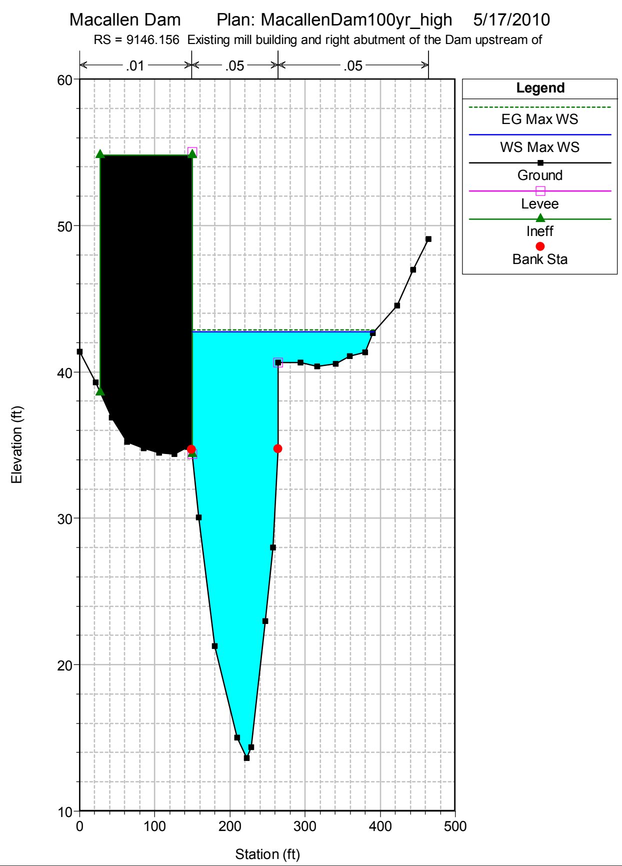


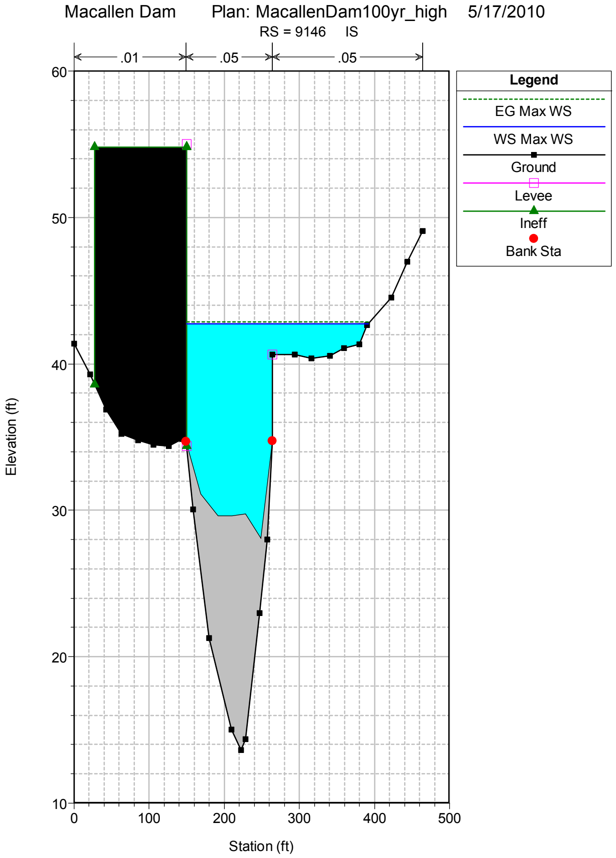
Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 9362.292 BR
.05
.03
.05
Legend
EG Max WS
WS Max WS
Ground
Bank Sta
Elevation (ft)
Station (ft)

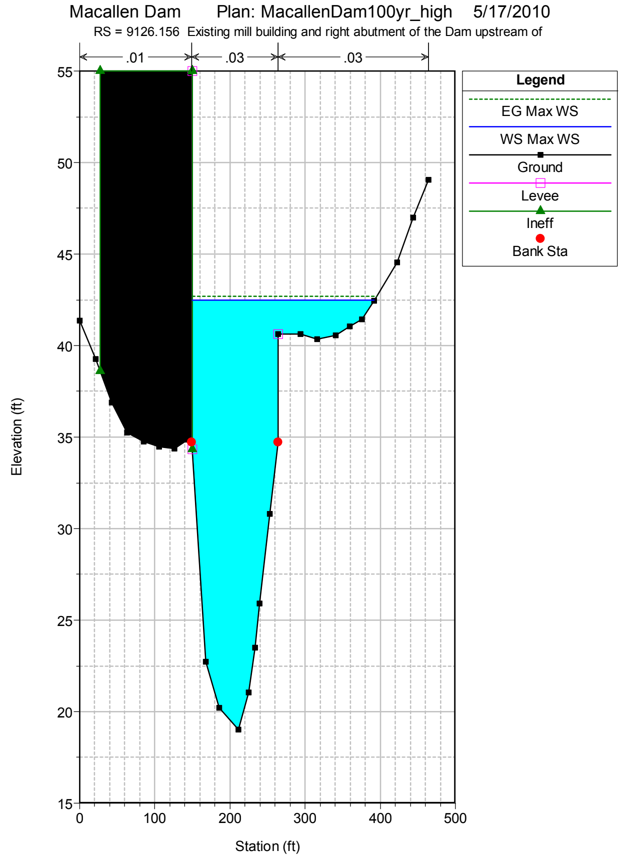

Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 9362.292 BR
.05
.03
.05
Legend
EG Max WS
WS Max WS
Ground
Bank Sta
Elevation (ft)
Station (ft)
55
50
45
40
35
30
25
20
0
100
200
300
400
500
600

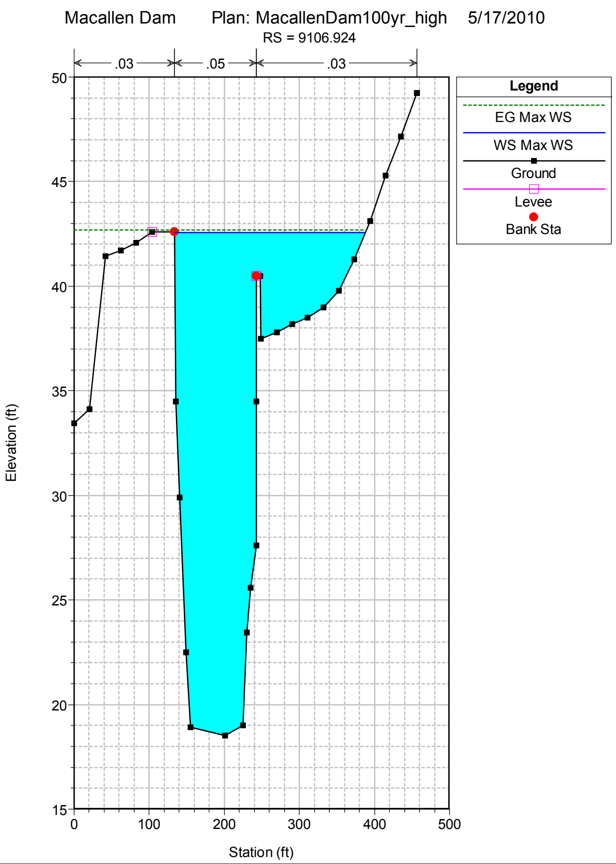

Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 9328.794 Downstream cross section for the 108 Bridge
.05
.03
.05
Legend
EG Max WS
WS Max WS
Ground
Bank Sta
Elevation (ft)
Station (ft)
55
50
45
40
35
30
25
20
0
100
200
300
400
500
600

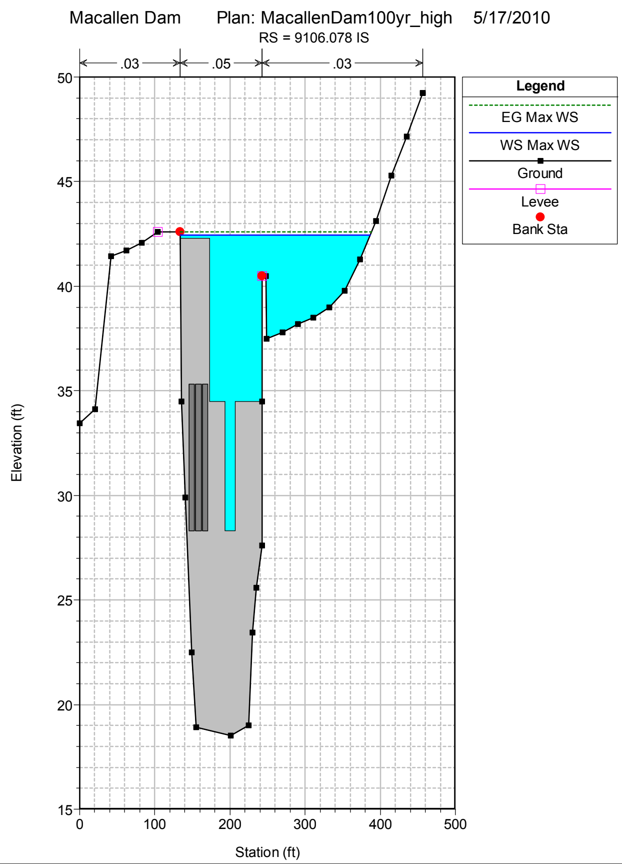

Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 9177.237 Existing mill building upstream of the Dam
.01
.03
.05
Legend
EG Max WS
WS Max WS
Ground
Levee
Ineff
Bank Sta
Elevation (ft)
Station (ft)
50
45
40
35
30
25
20
15
10
0
100
200
300
400
500

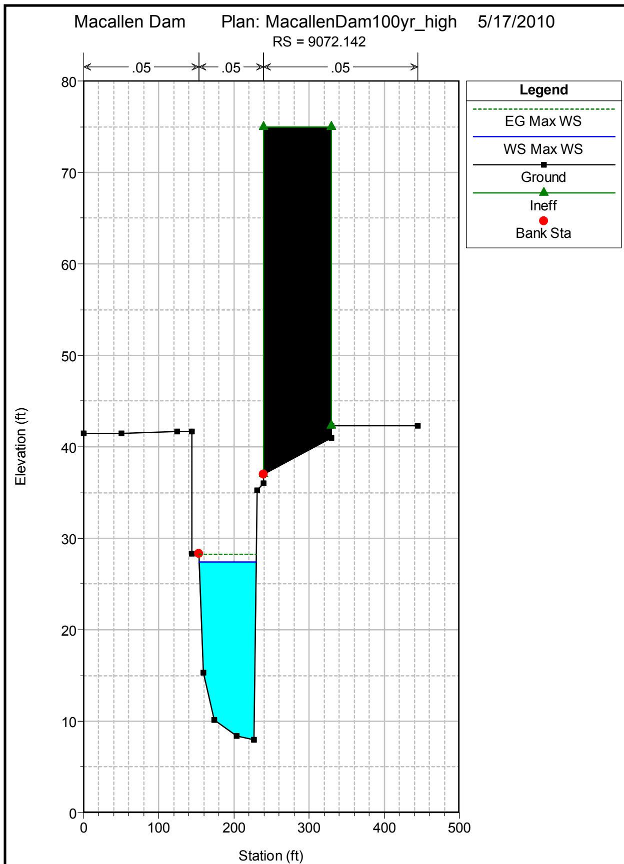

Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 9146.156 Existing mill building and right abutment of the Dam upstream of
.01
.05
.05
Legend
EG Max WS
WS Max WS
Ground
Levee
Ineff
Bank Sta
Elevation (ft)
Station (ft)
60
50
40
30
20
10
0
100
200
300
400
500


Macallen Dam
Plan: MacallenDam100yr_high
5/17/2010
RS = 9146 IS
.01
.05
.05
Legend
EG Max WS
WS Max WS
Ground
Levee
Ineff
Bank Sta
Elevation (ft)
Station (ft)


Macallen Dam
Plan: MacallenDam100yr_high
5/17/2010
RS = 9126.156 Existing mill building and right abutment of the Dam upstream of
.01
.03
.03
Legend
EG Max WS
WS Max WS
Ground
Levee
Ineff
Bank Sta
Elevation (ft)
Station (ft)


Macallen Dam
Plan: MacallenDam100yr_high
5/17/2010
RS = 9106.924
.03
.05
.03
Legend
EG Max WS
WS Max WS
Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)


Macallen Dam
Plan: MacallenDam100yr_high
5/17/2010
RS = 9106.078 IS
.03
.05
.03
Legend
EG Max WS
WS Max WS
Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)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RS = 9072.142
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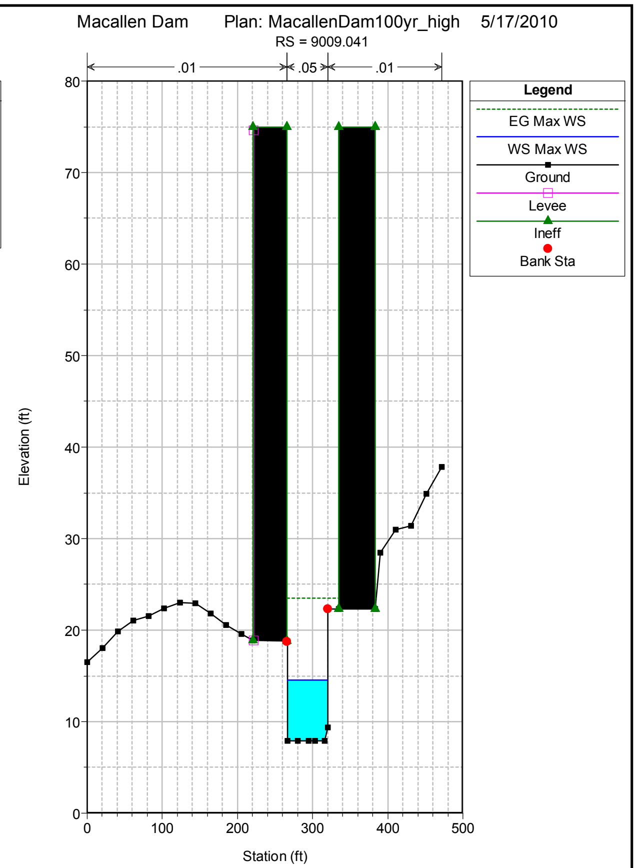
.05
.05
Legend
EG Max WS
WS Max WS
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)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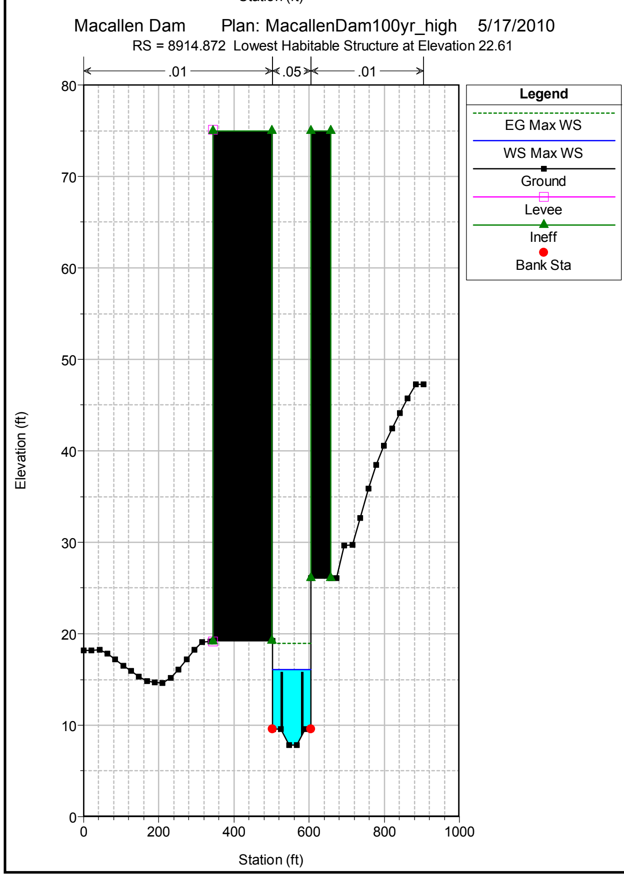
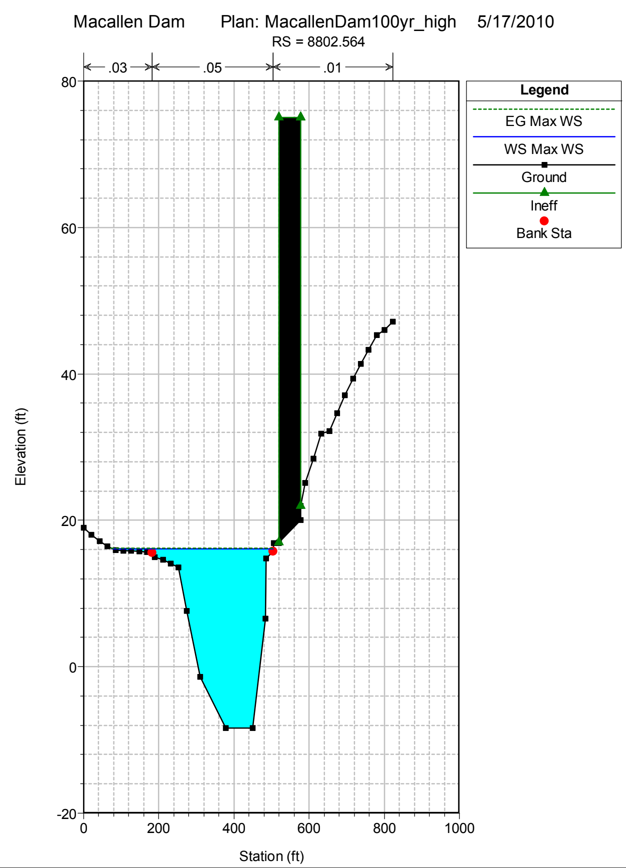
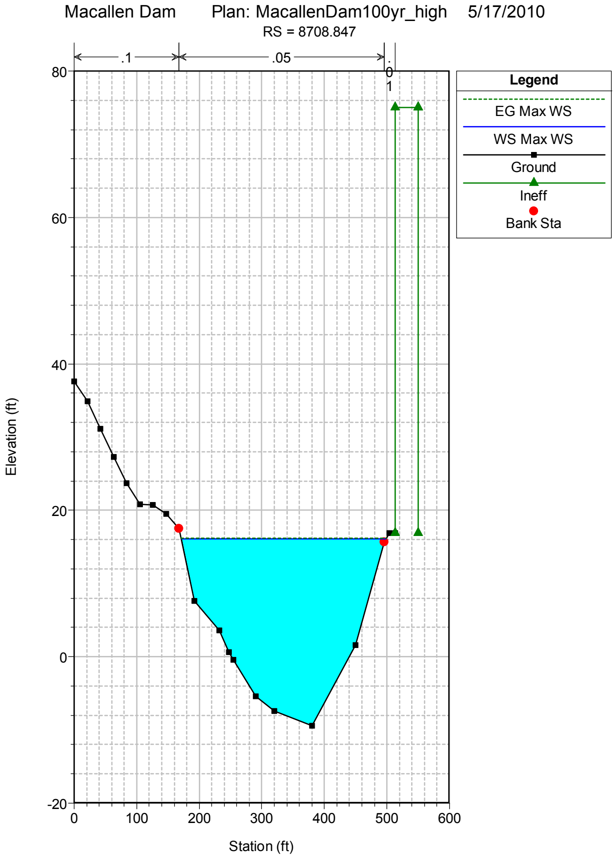
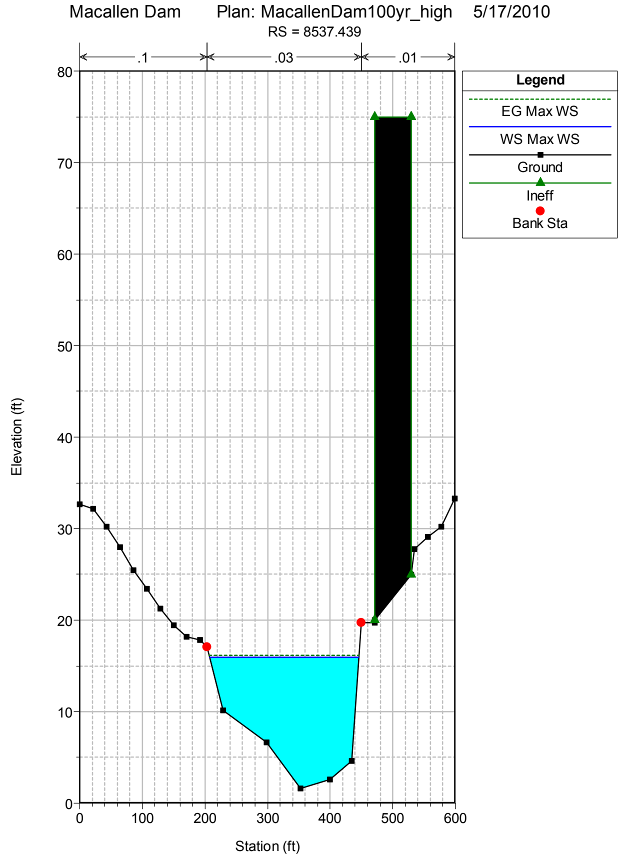
RS = 9009.041
.01
.05
.01
Legend
EG Max WS
WS Max WS
Ground
Levee
Ineff
Bank Sta
Elevation (ft)
Station (ft)


Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 8914.872 Lowest Habitable Structure at Elevation 22.61
.01
.05
.01
Legend
EG Max WS
WS Max WS
Ground
Levee
Ineff
Bank Sta
Elevation (ft)
Station (ft)


Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 8802.564
.03
.05
.01
Legend
EG Max WS
WS Max WS
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 8708.847
.1
.05
Legend
EG Max WS
WS Max WS
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)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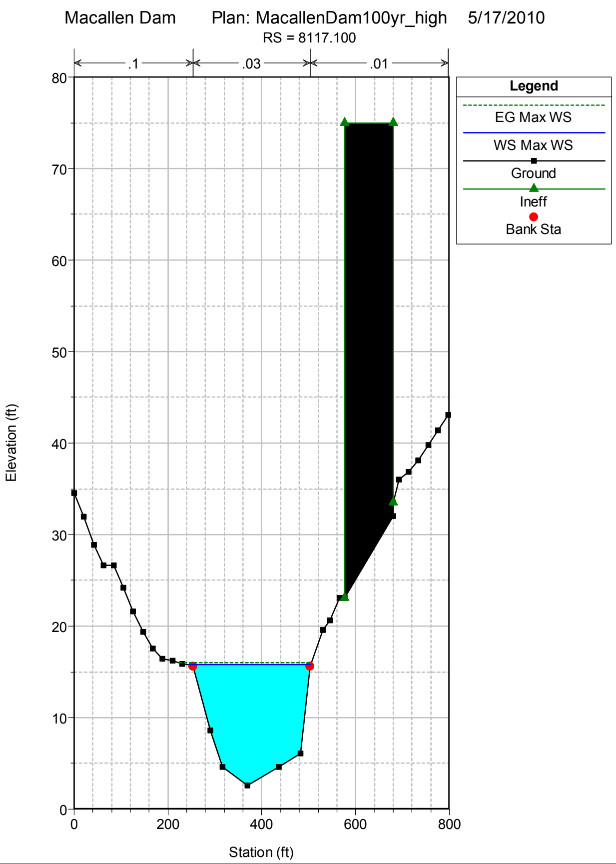
.1
.03
.01
Legend
EG Max WS
WS Max WS
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)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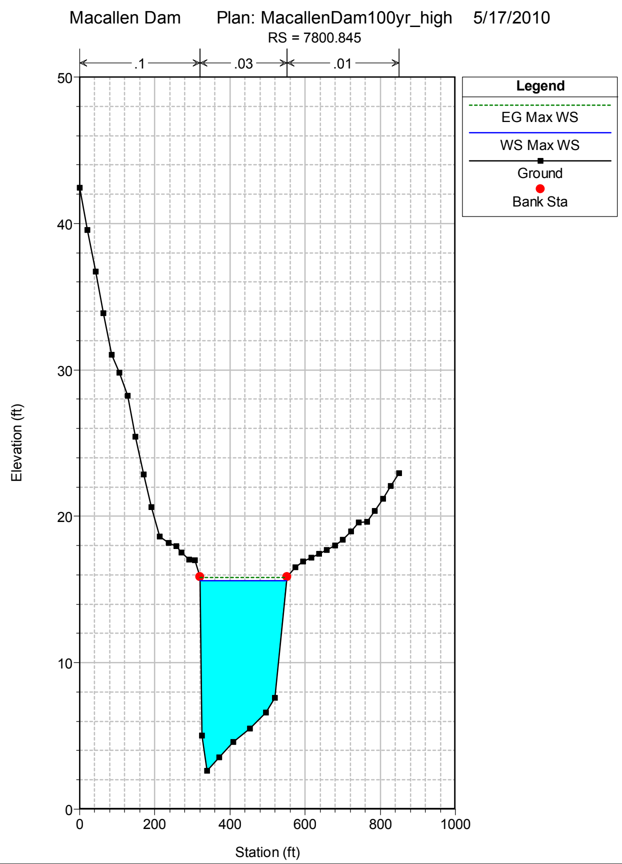
.1
.03
.01
Legend
EG Max WS
WS Max WS
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


Macallen Dam Plan: MacallenDam100yr_high 5/17/2010
RS = 7800.845
.1
.03
.01
Legend
EG Max WS
WS Max WS
Ground
Bank Sta
Elevation (ft)
Station (ft)

HEC-RAS Plan: MacallenDam100 River: Lamprey Reach: MaCallen_Dam Profile: Max WS

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
MaCallen_Dam	9804.856	Max WS	8302.00	12.68	44.22		44.31	0.000035	2.42	3579.42	175.81	0.08
MaCallen_Dam	9593.183	Max WS	8256.39	12.69	44.12		44.30	0.000082	3.49	2539.64	170.66	0.12
MaCallen_Dam	9395.759	Max WS	8219.45	23.00	43.96	32.99	44.29	0.000230	4.81	2078.89	177.53	0.21
MaCallen_Dam	9362.292		Bridge									
MaCallen_Dam	9328.794	Max WS	8219.45	20.59	42.59		43.01	0.000301	5.28	1706.32	132.55	0.23
MaCallen_Dam	9177.237	Max WS	8206.62	12.81	42.82		42.90	0.000034	2.28	3670.88	181.03	0.08
MaCallen_Dam	9146.156	Max WS	8205.00	13.64	42.73	25.02	42.90	0.000253	3.30	2677.46	241.89	0.13
MaCallen_Dam	9146		Inl Struct									
MaCallen_Dam	9126.156	Max WS	8006.86	19.03	42.49		42.70	0.000138	3.74	2307.89	242.34	0.15
MaCallen_Dam	9106.924	Max WS	7879.33	18.51	42.57	25.79	42.70	0.000252	3.03	2812.70	253.78	0.12
MaCallen_Dam	9106.078		Inl Struct									
MaCallen_Dam	9072.142	Max WS	9325.51	8.00	22.01		23.81	0.006474	10.77	865.52	73.12	0.55
MaCallen_Dam	9009.041	Max WS	9141.36	7.90	19.92		23.11	0.013549	14.33	638.01	54.23	0.74
MaCallen_Dam	8914.872	Max WS	9326.50	7.80	14.08	15.60	19.79	0.058644	19.17	486.63	92.40	1.47
MaCallen_Dam	8802.564	Max WS	9097.94	-8.39	13.13		13.23	0.000185	2.48	3668.67	230.17	0.11
MaCallen_Dam	8708.847	Max WS	9089.68	-9.39	13.15		13.22	0.000137	2.04	4459.51	309.15	0.09
MaCallen_Dam	8537.439	Max WS	8976.26	1.61	12.75		13.22	0.000883	5.49	1635.98	223.13	0.36
MaCallen_Dam	8117.100	Max WS	9003.04	2.61	12.36		12.85	0.000945	5.60	1606.90	225.53	0.37
MaCallen_Dam	7800.845	Max WS	8424.06	2.61	8.50	8.80	10.75	0.011217	12.03	700.16	199.09	1.13