Boundary Hydroelectric Project (FERC No. 2144)

Study No. 2
Analysis of Peak Flood Flow
Conditions above Metaline Falls
Final Report

Prepared for
Seattle City Light

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March 2009

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Study No. 2: Analysis of Peak Flood Flow Conditions above Metaline Falls Final Report Boundary Hydroelectric Project (FERC No. 2144)

1 INTRODUCTION

Study No. 2, the Analysis of Peak Flood Flow Conditions above Metaline Falls, was conducted in support of the relicensing of the Boundary Hydroelectric Project (Project), Federal Energy Regulatory Commission (FERC) No. 2144, as identified in the Revised Study Plan (RSP; SCL 2007) submitted by Seattle City Light (SCL) on February 14, 2007, and approved by FERC in its Study Plan Determination letter dated March 15, 2007. This final report describes the study field efforts, analyses, and the determination of Project effects.

During the July 19, 2006, FERC scoping meeting, Mr. Karl McKenzie, owner of Riverview Trailer Court & RV Park in Metaline, Washington, indicated that Project operations might exacerbate flooding in the Upper Reservoir Reach of the Boundary Reservoir (above Metaline Falls), thereby potentially contributing to the flooding of his property. However, FERC noted in its study request (FERC 2006a) that information in the Pre-Application Document (PAD) (SCL 2006) suggested that the natural constriction formed by the Canyon Reach and Metaline Falls prevents the Project from significantly affecting flood water surface elevations in the Upper Reservoir Reach. After FERC reviewed the information presented in the PAD and the comments provided during the scoping meeting, FERC identified a gap between existing information and the information needed to evaluate whether Project operations are influencing the duration and water surface elevation of floods upstream of Metaline Falls (FERC 2006a).

The study used recent topography and bathymetry to develop a hydraulic routing model to characterize reservoir conditions upstream of Metaline Falls relative to Project operations during periods of high flows in 1972, 1974, and 1997—the years Mr. McKenzie indicated, in the FERC scoping meeting, when flooding occurred. During the 1972, 1974, and 1997 events, the peak inflow from Box Canyon Reservoir into Boundary Reservoir, as recorded at USGS gage 12396500, which is located just below Box Canyon Dam, was 136,000, 133,000, and 134,000 cubic feet per second (cfs), respectively. A comparison of the annual peak flows recorded at USGS gage 12396500 is summarized in Table 1.0-1; this comparison indicates that the inflows to the Boundary Reservoir recorded during the high flow events occurring in 1972, 1974, and 1997 were the maximum observed since completion of the Project in 1967.

The hydraulic routing model developed to support the analysis necessary in assessing high flow conditions above Metaline Falls is referred herein as the Peak Flow Model. A separate model was constructed to assess habitat conditions in support of Study 7, Mainstem Aquatic Habitat Modeling Study Final Report (SCL 2009a). The Study 7 model, referred to as the Boundary Reservoir Hydraulic Routing Model (HRM), was used as the starting point for the development of the Peak Flow Model as discussed in further detail in this study.

Table 1.0-1. Summary of annual peak inflow to Boundary Reservoir, as recorded at USGS gage 12396500 since the completion of the Project (1967).

Water Year	Inflow (cfs)	Water Year	Inflow (cfs)
1972	136,000	2003	80,500
1997	134,000	1986	79,200
1974	133,000	1995	77,100
1971	114,000	1969	76,100
1981	106,000	1984	73,100
1982	105,000	1978	67,600
1991	104,000	1968	65,500
1970	103,000	1989	63,600
1975	103,000	1985	62,300
1996	100,000	1988	58,000
2002	100,000	1993	56,800
2006	98,400	2007	55,400
1980	98,300	2000	54,000
2008	96,000[1]	2004	51,300
1976	88,400	1987	44,300
1979	87,100	1994	33,600
2005	86,300	1991	32,400
1998	84,600	2001	32,100
1983	84,000	1976	29,000
1990	83,700	1973	27,500
1999	82,700		

<u>Note</u>:
1 Provisional based on data provided by USGS.

This study evaluated how Project operations appear to influence flood conditions on the McKenzie property in the town of Metaline and on other improved properties above Metaline Falls within the reservoir floodplain. For the purpose of this study, flood conditions were defined as the point at which normal functioning of the community is impacted as a result of elevated water levels. In the case of the town of Metaline, this point occurs when flood waters begin to flow overbank in areas adjacent to the riverfront along the high bank at an elevation of approximately 2,013 feet NAVD 88 (2,008.97 feet NGVD 29)[1]. This occurs when inflows from Box Canyon to Boundary are at or above approximately 120,000 cfs. Historically, Project

[1] SCL is in the process of converting all Project information from an older elevation datum (National Geodetic Vertical Datum of 1929 [NGVD 29]) to a more recent elevation datum (North American Vertical Datum of 1988 [NAVD 88]). As such, elevations are provided relative to both data throughout this document. The conversion factor between the old and new data is approximately 4 feet (e.g., the crest of the dam is 2,000 feet NGVD 29 and 2,004 feet NAVD 88). Although some other relicensing studies may round the conversion to 4 feet, the Project forebay elevations are monitored with precision of 0.01 foot and the hydraulic routing model provides output to the same level of precision — rounding of output, if appropriate, will be performed after application of the actual conversion factor of 4.03 feet.

forebay elevations during such peak flows have generally been approximately equal to 1,989 feet NAVD 88 (1,984.97 feet NGVD 29).

Analysis conducted as part of this study primarily addressed the estimated influence of Project operation on flood conditions in terms of flood elevations and areas inundated. Throughout this report, modeled water surface elevations are reported to the precision of 0.01 foot provided in the model output. Readers should be aware that the model is not considered to simulate water surface elevations to this level of accuracy. The results of the calibration process for a variety of conditions produced root mean square errors ranging from 0.12 foot to 0.46 foot. The difference in the maximum water surface modeled compared to the those recorded for four flood events used for calibration varied from -0.22 foot to +0.18 foot (details of the calibration process are provided in Section 4.2).[2] These ranges indicate a typical accuracy of modeled water surface elevations at peak flows of approximately 0.2 foot

2 STUDY OBJECTIVES

The goals of the study were to: 1) evaluate how Project operations may influence the duration and water surface elevation of flood conditions on the McKenzie property and on other improved properties above Metaline Falls within the reservoir floodplain and 2) identify any procedures that may attenuate the potential conditions and the cost of implementing such measures. The objectives of the study were as follows:

- Select transects to measure and utilize in a hydraulic routing model for the Upper Reservoir Reach adjacent to and in the vicinity of the town of Metaline.
- Develop an unsteady-flow hydraulic routing model that estimates water surface elevations and storage along modeled transects on an hourly basis for historical 1972, 1974, and 1997 high flows and Project operational conditions.
- Document the flow conditions when flooding of the Metaline area occurred in 1972, 1974, and 1997, and document the reservoir water surface elevations and surrounding land elevations in the Metaline area during these floods, if applicable.
- Determine the Project operational feasibility, effects on generation, and cost of implementing any procedures that might attenuate flooding conditions attributable to Project operations.

3 STUDY AREA

The overall study area for this effort encompassed the Pend Oreille River from Box Canyon Dam at Project river mile (PRM) 34.5 to Boundary Dam at PRM 17.0, as presented in Figure 3.0-1 (SCL 2007). The Project reservoir occupies three main reaches. From downstream to upstream, these reaches are defined as follows:

- *Forebay Reach*—this reach is relatively wide and extends upstream from Boundary Dam (PRM 17.0) to approximately PRM 19.4.

[2] All model-simulated flood elevations will be reported to the nearest hundredth of a foot. The reader should be cautioned that this does not imply the model has been calibrated to that level of accuracy. The development, calibration, and accuracy of the Peak Flow Model are discussed in detail within Section 4.

- *Canyon Reach*—this reach occupies a narrow gorge extending from the downstream end of Z Canyon (PRM 19.4) upstream to Metaline Falls (PRM 26.8).
- *Upper Reservoir Reach*—this reach is wider and shallower than the two downstream reaches, and extends from Metaline Falls (PRM 26.8) up to Box Canyon Dam (PRM 34.5).

The area of focus for this study was within the Upper Reservoir Reach. The study area encompassed land adjacent to the reservoir sufficient to address the question of the potential relationship of Project operations to flood conditions on the McKenzie property in the town of Metaline (PRM 29.0 to 27.8). It further encompassed other improved properties determined to be within the reservoir floodplain in the Upper Reservoir Reach based on the results of the Peak Flow Model.

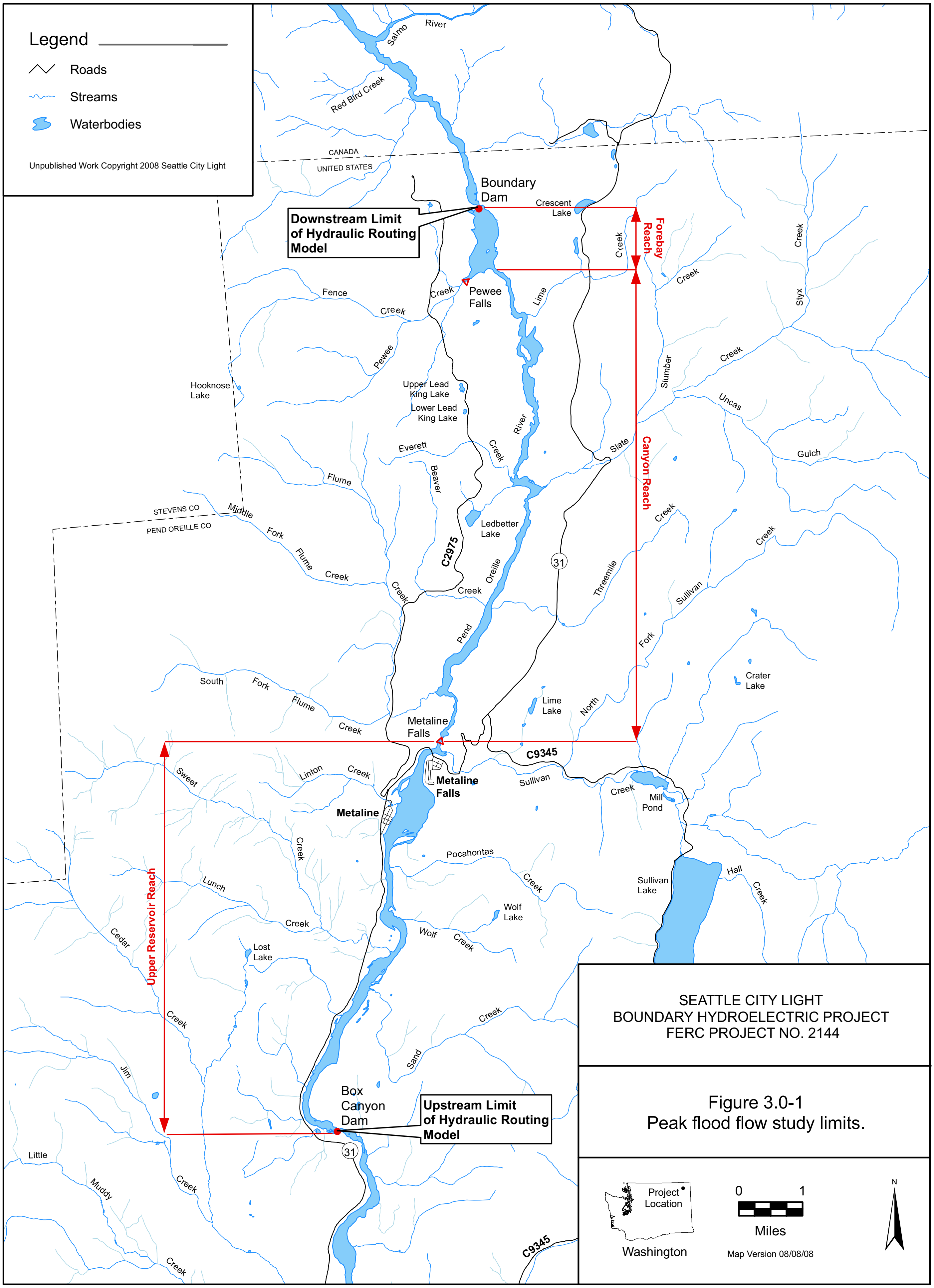

Figure 3.0-1
Peak flood flow study limits.

4 METHODS

The following sections describe the methods used to construct and calibrate the Peak Flow Model used to assess the potential effects of Project operations during periods of high flow. The RSP (SCL 2007) identified that a one-dimensional, unsteady-flow hydraulic routing model be used. The Peak Flow Model was developed using the U.S. Army Corps of Engineers (USACE) HEC-RAS software (USACE 2002a, 2002b, 2006). The construction of the Peak Flow Model was based on initial efforts from the Study 7 Boundary Reservoir HRM. The Study 7 Boundary Reservoir HRM was constructed to evaluate habitat during flows less than 80,000 cfs. The Peak Flow Model was developed to address hydraulic conditions above this threshold of 80,000 cfs. The calibrated Peak Flow Model was used to estimate the influence of Project operations on water surface elevations and duration during flooding conditions above Metaline Falls.

4.1. Hydraulic Routing Model Construction

This section will present various aspects of the development of the Peak Flow Model, including:

- Bathymetry and topography used to develop the hydraulic routing model
- Cross section location and development of cross section geometry
- Boundary conditions for the hydraulic model
- Data and information specifically used for calibration

The Peak Flow Model was developed using Version 4.0 of the USACE HEC-RAS model, along with Version 4.1.1 of the USACE HEC-GeoRAS software. The HEC-RAS executable code and documentation are public domain software that was developed by the Hydrologic Engineering Center (HEC) for the USACE (USACE 2006). HEC-RAS is designed to perform one-dimensional hydraulic calculations for a dendritic network of natural and constructed channels. HEC-RAS computes the propagation of a floodwave with respect to the distance along the channel through the solution of the complex one-dimensional Saint-Venant equations of unsteady flow. The principles of conservation of mass and conservation of momentum form the basis of these equations. User input to HEC-RAS is comprised of a series of cross sections spaced at intervals along the length of the study area; information that characterizes each of the cross section's resistance to flow; and definition of the boundary conditions at the upstream and downstream end of the modeling reach.

HEC-GeoRAS is an ArcGIS extension that provides the user with a set of procedures, tools, and utilities for the preparation of Geographic Information System (GIS)-based data for import into HEC-RAS and generation of GIS-based representation of HEC-RAS output. HEC-GeoRAS was used primarily in the creation of a geo-referenced cross section location database and for pre-processing of the geometric data for input into HEC-RAS and the post-processing to develop inundation maps. Version 4.1.1 of the HEC-GeoRAS was used and is compatible with ArcGIS Version 9.1.

The basic data and information necessary for the development of the hydraulic routing models are topography, upstream and downstream boundary conditions, and local inflows. Topographic data were used to develop the series of cross sections oriented perpendicular to the flow that represent the geometry of the river and reservoir system. The boundary condition at the upstream boundary consists of a flow hydrograph of hourly inflows from Box Canyon Dam into Boundary reservoir. The boundary condition at the downstream end of the study consists of stage hydrograph of hourly water surface elevation versus time and represents Project forebay elevations.

4.1.1. Bathymetry and Topography

A multibeam sonar bathymetric survey was conducted within Boundary Reservoir by Global Remote Sensing, LLC (GRS) in 2006. The data from this survey were supplemented and checked, within eleven priority areas, with a high-resolution multibeam bathymetry and scanning survey by Tetra Tech in June and July 2007. GRS partially resurveyed the reservoir with a high-resolution multibeam bathymetry system in October 2007. Tetra Tech conducted a concurrent shoreline scanning laser survey to provide full coverage of the shoreline below Metaline Falls.

The bathymetric and scanning laser data were combined with topographic surveys conducted using Light Detection and Ranging (LiDAR) technology. The LiDAR data were collected from aerial flights in August 2005 by Terrapoint (2005). The bathymetric data and the LiDAR data were merged together to form a continuous digital terrain model (DTM) in the form of a triangulated irregular network (TIN) for Boundary Reservoir. The DTM is a digital representation of the ground surface topography. Figure 4.1-1 shows an example portion of the terrain model in the vicinity of Metaline Falls. Included in this figure are the hydraulic model cross sections (discussed in Section 4.1.2) that are located through this region.

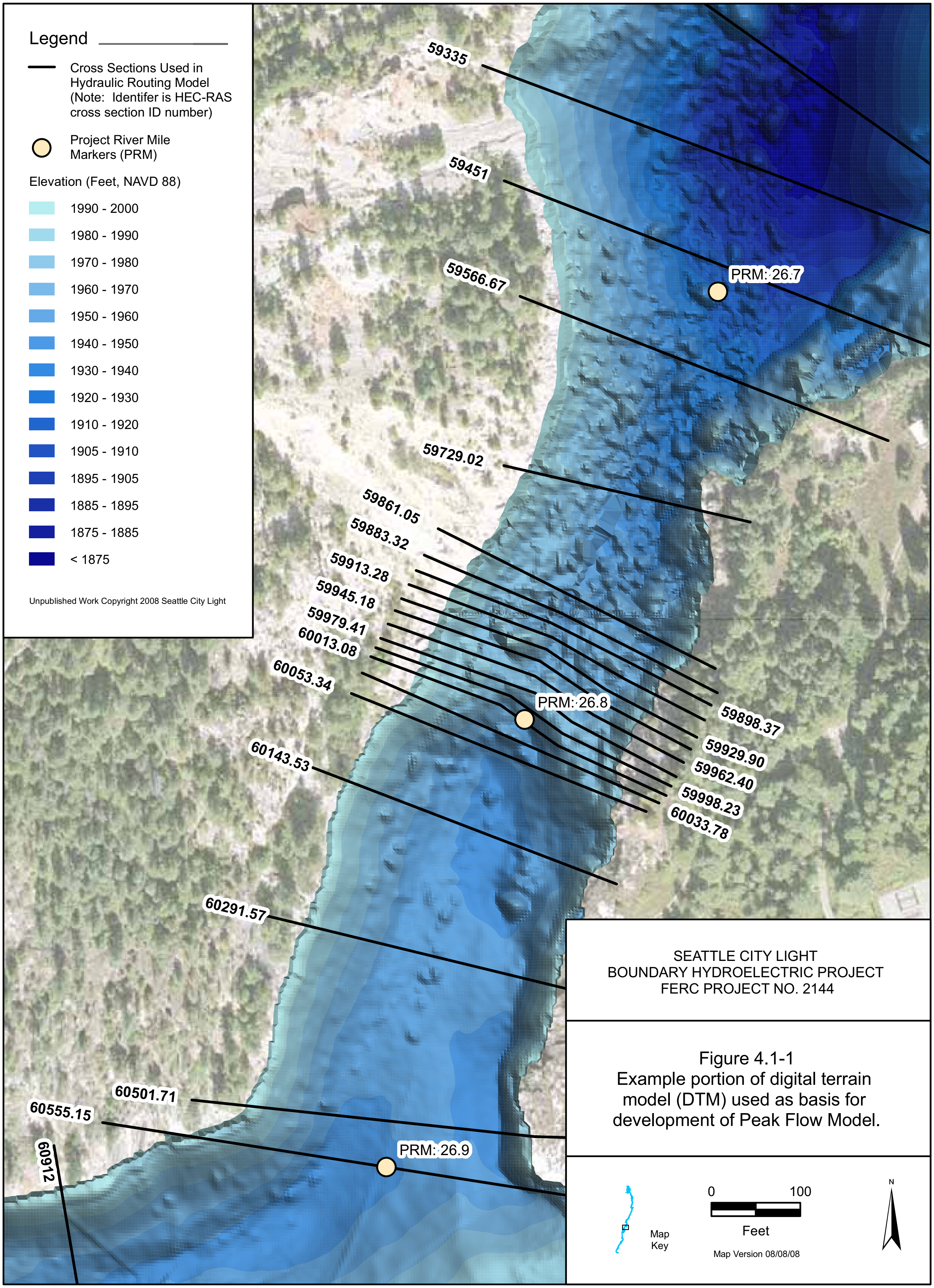

Figure 4.1-1
Example portion of digital terrain model (DTM) used as basis for development of Peak Flow Model.

4.1.2. Cross Section Locations

As previously mentioned, the initial selection of cross sections for the Peak Flow Model was based on efforts conducted as part of Study 7, Mainstem Aquatic Habitat Modeling Study Final Report (SCL 2009a). A total of 225 cross sections were imported for use in the Peak Flow Model from the Study 7 Boundary Reservoir HRM. Each cross section was used to characterize the conveyance capacity of the river at a point in space, but is also used in the hydraulic computations to represent the channel and floodplain geometry to the next downstream cross section. Cross section locations from the Study 7 Boundary Reservoir HRM were identified at locations along the river where changes in channel slope and channel shape were observed to occur, and at locations where changes in the channel roughness conditions were observed during site visits. In locations where abrupt changes in geometry occur, such as at Metaline Falls, cross section spacing was intensified.

For the Peak Flow Model, additional cross sections were also located at specific points where it was anticipated that hydraulic information would be required. This included locations where high water marks from historical events were available. A total of 230 cross sections were ultimately included in the Peak Flow Model. Figure 4.1-1 is an example figure showing the cross section locations for a portion of the model. A complete set of figures showing all cross section locations is included in Appendix 1 of this report. The average spacing between the cross sections was approximately 400 feet, although this value is skewed due to the closer spacing of the cross sections through Metaline Falls. The average spacing between cross sections through Metaline Falls is approximately 20 feet.

Once the locations were identified, the cross sections were cut through the DTM using the HEC-GeoRAS software and were imported into the HEC-RAS model. All cross sections were cut through the DTM with a downstream orientation (i.e., how the cross sections would look to an observer standing upstream looking downstream). An example of a cross section from the Peak Flow Model is shown in Figure 4.1-2.

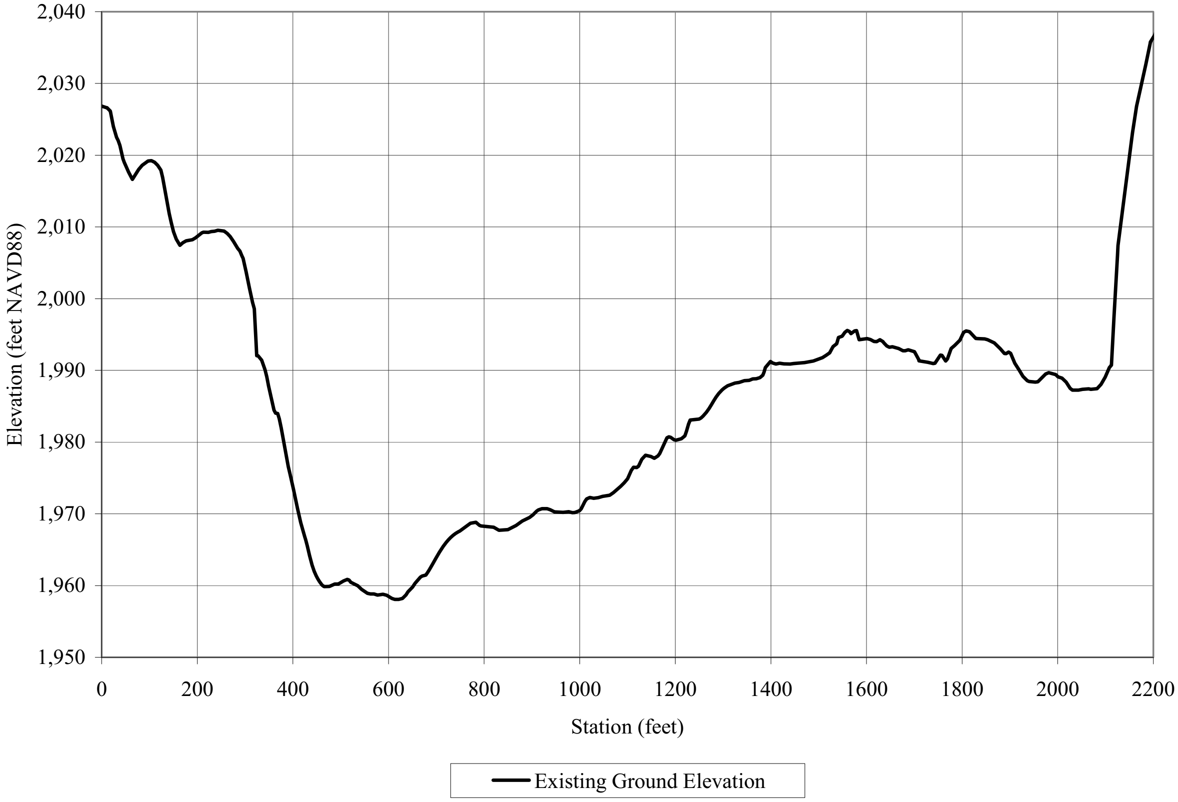

Figure 4.1-2. Example cross section from the Peak Flow Model.

4.1.3. Initial n-Values and Loss Coefficients

Initial estimates of the Manning's roughness coefficients and the expansion and contraction coefficients were derived from field observations, aerial photography, and guidance from USACE (2006), Barnes (1967), and Arcement and Schneider (1989). These coefficients are used to assess the energy losses at each cross section within the hydraulic routing model.

Significant inundation of the overbank areas was not expected to occur during the development of the Boundary Reservoir HRM, and the use of three Manning's roughness coefficients (left overbank, channel, and right overbank) was justified. However, during peak flows, inundation of the overbank areas does occur, requiring the use of horizontal variation of Manning's n-value. Horizontal adjustment of overbank Manning's roughness allows for the variability of land cover within the floodplain to be modeled during the periods of high flow.

Delineation of land cover types above Metaline Falls to Box Canyon Dam (PRM 26.8 to 34.5) were based on the 2005 aerial photographs using GIS software. Arcement and Schneider (1989) and Chow (1959) was used to estimate initial overbank Manning's roughness coefficients based on the general land cover classifications identified during site visits. Initial overbank Manning's roughness coefficients selected for a given land cover are presented in Table 4.1-1. An example of the land cover delineation is presented in Figure 4.1-3.

Table 4.1-1. Initial estimates of overbank Manning's roughness coefficient for a given land cover type.

Land Cover	Calculated Overbank Manning's n-Value		
	Lower Limit	Initial Value Selected	Upper Limit
Dense Forest	0.090	0.106	0.160
Light Forest	0.070	0.086	0.130
Grass/Open Space	0.040	0.058	0.110
Urban	0.035	0.043	0.090

Channel Manning's roughness coefficients were based on the calibrated values developed for the Boundary Reservoir HRM (SCL 2009a). The calibrated coefficients were used as the initial starting point of the Peak Flow Model. Table 4.1-2 summarizes the initial estimated Manning's roughness coefficients and expansion and contraction coefficients used for the Peak Flow Model. Values reported within Table 4.1-1 were used to develop the conveyance weighted, average overbank Manning's roughness coefficient presented in Table 4.1-2.

Table 4.1-2. Initial estimates of model calibration parameters for Peak Flow Model.

Upstream HEC-RAS Cross Section[1]	Downstream HEC-RAS Cross Section[1]	Upstream Project River Mile[2]	Downstream Project River Mile[2]	Manning's Roughness			Contraction Coefficient	Expansion Coefficient
				Left Overbank[3]	Channel	Right Overbank[3]		
102198	99871	34.39	33.98	0.046	0.051	0.048	0.4	0.6
99871	98093	33.98	33.64	0.052	0.029	0.031	0.3	0.5
98093	95152	33.64	33.11	0.089	0.036	0.048	0.3	0.5
95152	91365	33.11	32.44	0.047	0.031	0.045	0.3	0.5
91365	84450	32.44	31.18	0.042	0.028	0.042	0.3	0.5
84450	72815	31.18	29.09	0.051	0.032	0.046	0.3	0.5
72815	60912	29.09	26.98	0.039	0.032	0.041	0.3	0.5
60912	60053.34	26.98	26.81	0.098	0.098	0.098	0.6	0.7
60053.34	59566.67	26.81	26.72	0.122	0.122	0.122	0.6	0.8
59566.67	12256	26.72	18.03	0.092	0.092	0.092	0.3	0.5
12256	5428	18.03	17.02	0.028	0.028	0.028	0.1	0.3

Notes:
1 Refer to figures in Appendix 1 for HEC-RAS cross section locations.
2 Project river miles were based on linear interpolation between Project river mile identifiers at 0.1 mile increments.
3 Represented conveyance weighted, average overbank Manning's roughness coefficient.

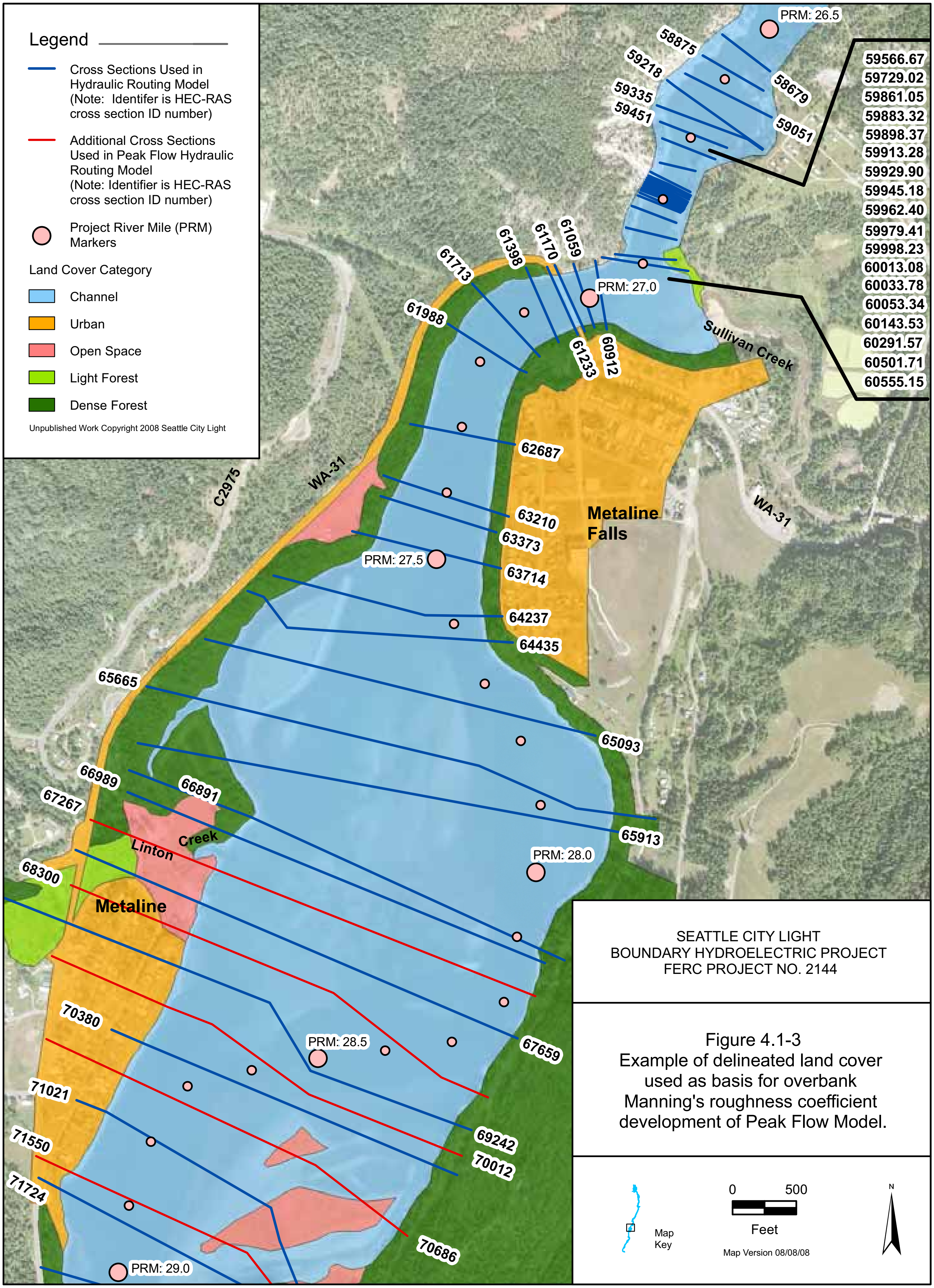

Figure 4.1-3
Example of delineated land cover used as basis for overbank Manning's roughness coefficient development of Peak Flow Model.

For the portion of the Pend Oreille River downstream of Metaline Falls, there is no discernable overbank area. This reach is essentially a deep, narrow canyon with nearly vertical walls. In addition, there is very little change in the portion of the channel wetted under the range of flow conditions modeled. Therefore, while right and left bank stations were defined in HEC-RAS for the purposes of executing the model, they were not defined with the intent of delineating a change in roughness coefficient between the main channel and the overbank areas. As presented in Table 4.1-1, a single Manning's roughness coefficient value was used as a cross section averaged value for those cross sections downstream of Metaline Falls.

Estimates of the contraction and expansion loss coefficients were based on guidance presented in the HEC-RAS User's Manual (USACE 2006), review of the bathymetry and aerial photographs, and observations of site conditions. Some of these site conditions observed included the presence of flow separation and eddies throughout the study area. One such location was within the Canyon Reach where large eddies and vortices were observed at PRM 21.78 where the canyon walls constrict flow through a small opening. According to USACE (2006), typical values of the empirical contraction and expansion coefficients where the change in the effective cross sectional area is small and gradual are 0.1 and 0.3, respectively. Calibration efforts of the Study 7 Boundary Reservoir HRM, however, suggest that these values are higher. The initial contraction and expansion coefficients selected for the Peak Flow Model were 0.3 and 0.5, respectively. This was applied to all reaches, with the exception of the Canyon Reach downstream of Metaline Falls. In this reach, higher initial values for the contraction and expansion coefficients were assigned (0.6 and 0.8, respectively) based on the observations that the flow is repeatedly expanding and contracting through narrow bedrock outcroppings and through the irregularly shaped walls of the canyon.

4.1.4. Boundary Conditions

Boundary conditions are required at upstream and downstream limits of the study area to permit modeling of hydraulics. For the Peak Flow Model, the water surface elevation in the forebay is used to establish the downstream boundary condition, and the inflow from Box Canyon is used to establish the upstream boundary condition. In addition to these boundary conditions, the model also requires tributary inflows if these are to be accounted for. Since the modeling involves unsteady flow, the boundary conditions must be provided for the entire time period being modeled. The following data were available for use as boundary conditions within the Peak Flow Model:

- Flow data from the USGS for the Pend Oreille River below Box Canyon Dam (USGS gage 12396500)
- Project forebay water surface elevations provided by SCL
- Synthesized flow records for the tributaries to Boundary Reservoir between Box Canyon Dam and Boundary Dam provided by R2 Resource Consultants, Inc. (2008)

Time series of Project forebay water surface elevations and coincidental flows measured at the USGS gage 12396500 (USGS 2006) were used as the downstream and upstream boundary conditions, respectively. Total inflow consists of flows released by Box Canyon Dam plus the sum of all inflows from the tributaries between Boundary Dam and Box Canyon Dam. Flows released by Box Canyon Dam are represented by the flow rate measured at USGS gage

12396500. To determine the inflows from the tributaries to Boundary Reservoir, it was necessary to synthesize the inflow time series for each of the 15 named tributaries and for the combined 13 unnamed tributaries and the local hill slope drainage area. Synthesis of the tributary inflow time series was necessary because the only gaged tributary within the study area is Sullivan Creek (USGS gage 12398000). The methodology used to synthesize these time series is described in R2 Resource Consultants, Inc. (2008). The data used to establish the boundary conditions for the Peak Flow Model are shown in detail in Appendix 2.

4.1.5. Information for Calibration

Calibration of the Peak Flow Model was conducted for the three historic periods of high flow that occurred in 1972, 1974, and 1997. An additional event was included within the calibration matrix that occurred in 2008 and was selected because of the relatively large peak flow that was recorded this spring (14th highest peak flow since completion of the Project in 1967) and the availability of calibration data. A summary of the calibration events is presented in Table 4.1-3.

Table 4.1-3. Summary of calibration events for the Peak Flow Model.

		USGS Gage 12396500				Project Forebay Elevation Fluctuation during the Peak Discharge Recorded at USGS Gage 12396500	
		Range of Discharge during Event Time Period (cfs)[2]	Peak Discharge (cfs)	Peak Water Surface Elevation at Primary Gage			
Year	Time Period			(ft NGVD 29)	(ft NAVD 88)	(ft NGVD 29)	(ft NAVD 88)
2008	5/19 – 6/12	60,600 – 96,000[1]	96,000[1]	2,008.25	2,012.28	1,985.73 – 1,988.72	1,989.76 – 1,992.75
1997	5/15 – 6/23	88,800 – 134,000	134,000	2,015.65	2,019.68	1,983.50 – 1,985.63	1,987.53 – 1,989.66
1974	6/7 – 7/7	72,600 – 133,000	133,000	2,014.87	2,018.90	1,983.05 – 1,983.19	1,987.08 – 1,987.22
1972	5/25 – 6/24	81,500 – 136,000	136,000	2,015.54	2,019.57	1,983.61 – 1,983.87	1,987.64 – 1,987.90

Note:
1 Provisional based on data provided by USGS.
2 Represents mean daily discharge reported at USGS gage 12396500

The available data that were used to calibrate the Peak Flow Model include the following:

- Stage data from the USGS for the Pend Oreille River below Box Canyon Dam (gage 12396500) at the primary and auxiliary gage location.
- Water surface elevation data (15-minute readings) during high flow conditions that occurred in 2008. Water surface elevation data are available at pressure transducers deployed in the Pend Oreille River at the following locations:
    - Just downstream of Box Canyon Dam (PRM 34.5), in the Upper Reservoir Reach.
    - Just upstream from Metaline Falls (PRM 27.0), in the Upper Reservoir Reach.
    - Just downstream from Metaline Falls (PRM 26.7), in the Canyon Reach.

- At the downstream end of the Canyon Reach (PRM 18.0).
- In the Project forebay (PRM 17.0), in the Forebay Reach.

- High-water marks as observed by Mr. McKenzie within the town of Metaline for the 1997 and 2008 event.
- Flow and stage data from SCL for total flow release from Boundary Reservoir (energy generation plus spill). Four turbines were operating in 1972 and 1974, and six turbines were operating in 1997.[3]

Calibration to recorded water surface elevations, high-water marks, and pertinent anecdotal information was accomplished for each flood event as discussed in further detail in Section 4.2. The raw water surface elevation data from the USGS gage 12396500 were available in 15-minute time increments for the 1997 and 2008 event. This USGS station actually comprises two recording stations: the primary station (PRM 34.3), which is located 1,000 feet downstream of Box Canyon Dam, and the auxiliary station (PRM 33.1), which is located 1.2 miles downstream of Box Canyon Dam. The provisional raw data from both stations, as provided by the USGS, were used for calibration to the 2008 event. Water surface elevations for the 1972 and 1974 events at USGS gage 12396500 were based on synthesized records developed by R2 Resource Consultants, Inc. (2008).

Continuously recorded 15-minute water surface elevation data were obtained from pressure transducers deployed in September 2006 at five locations in Boundary Reservoir. Each installation is comprised of a set of two identical Solinst Levelogger® (model M10/F30) that provide redundancy in the event one of the transducers malfunctions. Table 4.1-4 summarizes the coordinate location of each pressure transducer installation in the reservoir as well as the abbreviated naming convention assigned to each pressure transducer installation. The location of each installation as well as the USGS gage 12396500 is shown in Figure 4.1-4.

Table 4.1-4. Boundary Reservoir pressure transducer installation locations and naming conventions.

Pressure Transducer Installation Name	Description of Pressure Transducer Installation Location	Northing[1] (feet)	Easting[1] (feet)
BOX_TR	Box Canyon Tailrace	667,964.68	2,464,743.08
US_MET	Upstream of Metaline Falls—transducer mounted on one of the piers of the Highway 31 bridge	698,985.74	2,473,103.68
DS_MET	Downstream of Metaline Falls—transducer mounted on old powerhouse on east bank	700,302.83	2,474,187.03
CANYON	Mouth of "Z" Canyon—transducer mounted on canyon wall on east bank	738,667.89	2,478,253.01
BND_LK	Boundary Dam forebay	743,748.62	2,476,857.27

Note:
1 Northing and easting coordinates are relative to the Washington State Plane North Zone (4601) coordinate system and the NAD 1983 horizontal datum.

[3] The Project began commercial operation in 1967, with turbine-generator Units 51 through 54. In accordance with a 1982 license amendment approving expansion of the Project, Units 55 and 56 were constructed in two previously excavated bays in the machine hall and went on line in 1986.

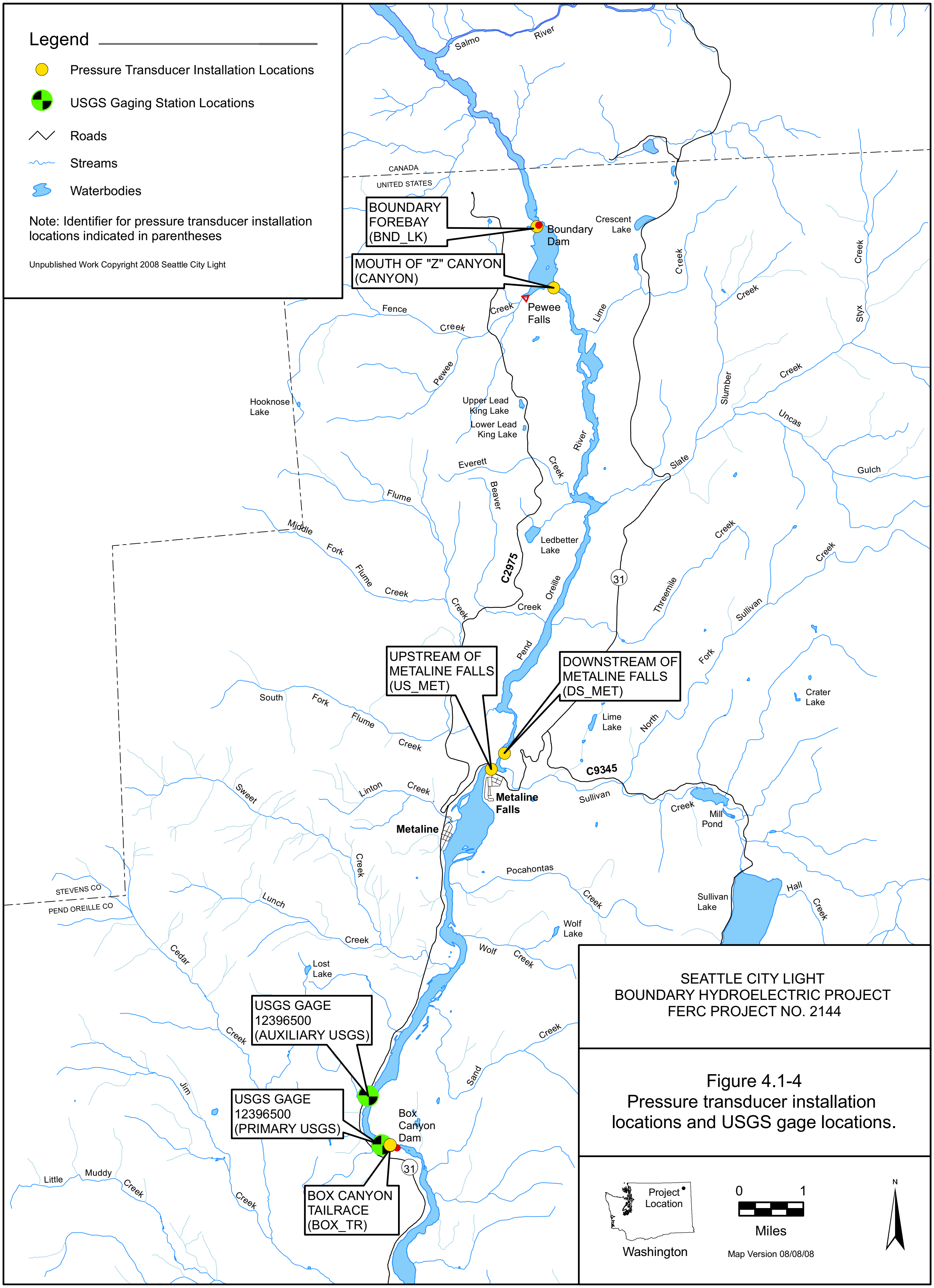

Figure 4.1-4
Pressure transducer installation locations and USGS gage locations.

Each pressure transducer provides continuous recording of the combined water pressure and barometric pressure above the transducer at 15-minute time intervals. The raw data collected by the pressure transducers were downloaded approximately every 3 months and were post-processed in Microsoft Excel®. The time series of depth data were converted to a time series of water surface elevation data relative to the NAVD 88 datum. This conversion included subtracting out the influence of barometric pressure on the depth readings at each time step. The data were reviewed to identify and eliminate erroneous instantaneous values. Since each installation included two pressure transducers, the review included identifying instances where the differential between two transducers was greater than a nominal value of 0.5 foot. The number of such instances was minimal, thus providing additional confidence in the reliability of the data. The final step was to average the two post-processed water surface elevations at each time step so as to generate a continuous time series at each pressure transducer location. The data collected at the BND_LK pressure transducer were used to represent the downstream boundary condition during the 2008 event. These data were not available for the 1972, 1974, and 1997 events.

High water marks observed by Mr. McKenzie were used to supplement the calibration of the Peak Flow Model. An interview was conducted on April 30, 2008, with Mr. McKenzie regarding his recollections of the river conditions experienced during the calibration events. Inundation extents and high water marks recalled by Mr. McKenzie were collected during the interview. The inundation extent experienced during the 1997 event at McKenzie's house is shown in Figure 4.1-5.

Figure 4.1-5. Oblique photograph of McKenzie's house during the 1997 high water event (date unknown).

A subsequent interview on July 19, 2008, was conducted to establish the inundation extents observed by McKenzie during the period of high water in 2008. Table 4.1-5 summarizes the high water surface elevations identified by McKenzie.

Table 4.1-5. Water surface elevation surveyed at the high water locations identified by Mr. McKenzie.

Year	Date	Observed Water Surface Elevation (feet NGVD 29)	Observed Water Surface Elevation (feet NAVD 88)
2008	June 7	2,005.23	2,009.26
1997	June 7	2,012.49	2,016.52
1997	June 10	2,011.80	2,015.83

Prior to the peak of the 2008 period of high water, a survey was conducted to determine the inundation extents within Metaline Park. Inundation extents at Metaline Park were mapped on May 30, 2008 by Tetra Tech staff, and water surface elevations were recorded. Flows reported by USGS gage 12396500 were approximately 89,000 cfs during the time of the survey. The level of inundation observed at Metaline Park during the survey is shown in Figure 4.1-6. Table 4.1-6 summarizes the observed water surface elevations recorded at the time of the survey and the locations are displayed in Figure 4.1-7.

Figure 4.1-6. Observed high water at Metaline Park on May 30, 2008.

Table 4.1-6. Observed water surface elevation recorded at Metaline Park on May 30, 2008.

Location	Northing[1] (feet)	Easting[1] (feet)	Observed Water Surface Elevation	
			(feet NGVD 29)	(feet NAVD 88)
Large Gazebo	694,352.81	2,470,046.49	2,003.75	2,007.78
Electrical Junction Box	694,299.84	2,470,001.31	2,003.82	2,007.85
Large Tree	694,268.42	2,469,956.22	2,003.79	2,007.82
Roadway	694,241.34	2,469,934.97	2,003.90	2,007.93

Note:
1 Northing and easting coordinates are relative to the Washington State Plane North Zone (4601) coordinate system and the NAD 1983 horizontal datum.

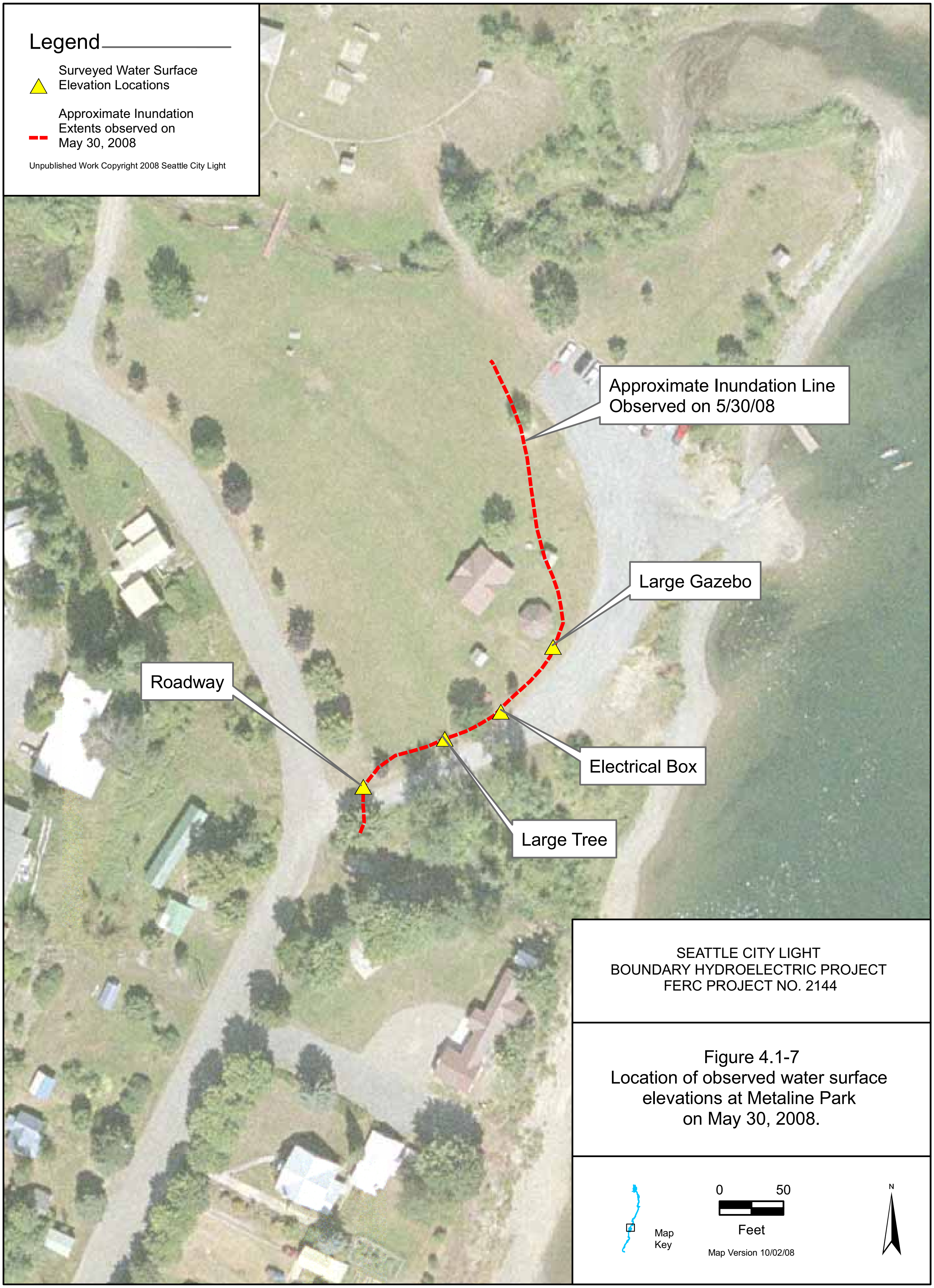

Figure 4.1-7
Location of observed water surface elevations at Metaline Park on May 30, 2008.

4.2. Hydraulic Routing Model Calibration

The calibration of the Peak Flow Model was conducted in a two-phase approach. The first phase consisted of the initial calibration of the 1972, 1974, and 1997 historical high flow events. The second phase calibration included a verification of the preliminary calibrated Peak Flow Model against the recorded water surface elevations occurring during the 2008 period of high flow. Review of the verification simulation highlighted the need for additional calibration to match the observed stage records recorded in 2008 above and below Metaline Falls (PRM 26.8). No information was available within this area during the preliminary calibration. The 2008 period of high flow was incorporated into the calibration matrix to generate the final calibrated Peak Flow Model. The following sections contain information regarding the calibration process of the Peak Flow Model.

4.2.1. Phase I—Calibration of Peak Flow Model

The Peak Flow Model was calibrated to observed water surface elevation hydrographs developed from measurements recorded at USGS gage 12396500. Iterative adjustments to Manning's roughness coefficients and the expansion and contraction coefficients were made until model-simulated water surface elevations were within an acceptable tolerance, as defined below. The arithmetic difference between the model-simulated and observed was computed for each 15-minute time increment. The maximum difference was then computed at each location for each calibration period. This provided quantitative feedback as to specific points in time within a given calibration period where the most significant deviation from observed conditions occurred. To provide a quantitative measure of the deviation from observed conditions at each calibration location for each event, the root mean square error (RMSE) was evaluated as follows:

$$\text{Root Mean Square Error (RMSE)} = \left[\sqrt{\frac{\sum_{i=1}^{n}\left(WSEL_{OBSi} - WSEL_{SIMi}\right)^2}{n}}\right]$$

Where:

$WSEL_{OBSi}$ = observed water surface elevation at time interval i
$WSEL_{SIMi}$ = model-simulated water surface elevation at time interval i
n = number of time intervals in model simulation

At the onset of the calibration, criteria were established that were used to guide the calibration and to determine when a successful calibration had been attained. For each calibration location within each calibration period, the magnitudes of the calibration parameters were iteratively varied, within physically acceptable ranges, until the following criteria were met:

- Maximum absolute difference between the observed water surface elevation hydrograph and the model-simulated water surface elevation hydrograph of less than 1.0 foot for each calibration location within each calibration event
- RMSE between observed and model-simulated less than 0.5 foot for each calibration location within each calibration event

Increases in Manning's roughness coefficients were necessary to match model-simulated water surface elevations with observed. Iterative adjustments were made starting at the upstream cross section at Metaline Falls progressing upstream towards Box Canyon Dam. A table summarizing the absolute difference in water surface elevation between the model-simulated and observed for the Phase I calibration is presented in Table 4.2-1. The RMSE between the model-simulated and observed for the Phase I calibration is presented in Table 4.2-2.

Table 4.2-1. Preliminary calibrated magnitude of maximum difference between observed and model-simulated water surface elevation for each calibration event.

High Flow Event	USGS 1239600 Primary Gage (HEC-RAS Station 101240[1])		USGS 12396500 Auxiliary Gage (HEC-RAS Station 96759[1])	
	Maximum Positive (feet)	Maximum Negative (feet)	Maximum Positive (feet)	Maximum Negative (feet)
1997	0.53	-0.45	0.32	-0.60
1974	0.36	-0.14	0.09	-0.42
1972	0.51	-0.98	0.28	-1.22

Note:
1 Table includes identifiers for HEC-RAS model cross sections associated with each calibration location.

Table 4.2-2. Preliminary RMSE at each calibration location for each calibration event.

High Flow Event	USGS 1239600 Primary Gage (HEC-RAS Station 101240[1]) RMSE (feet)	USGS 12396500 Auxiliary Gage (HEC-RAS Station 96759[1]) RMSE (feet)
1997	0.20	0.21
1974	0.14	0.22
1972	0.46	0.58

Note:
1 Table includes identifiers for HEC-RAS model cross sections associated with each calibration location.

Tables 4.2-1 and 4.2-2 identified a discrepancy in the model-simulations with the observed water surface elevations for the 1972 event. Review of the water surface elevation hydrograph for the 1972 event suggests that the rising limb of the inflow flood hydrograph is inconsistent with the concurrent water surface elevations reported at the two USGS gage locations. The synthesis was based on mean daily flows reported by USGS supplemented with discrete observations also provided by USGS. The resulting synthesized 15-minute flow hydrograph is smoothed and does not contain any rapid fluctuations. The resulting stage hydrograph appears to be underestimating the magnitude of inflow experienced from Box Canyon Dam during the rising limb of the flood hydrograph. The peak stage estimated, based on model simulations by the Peak Flow Model, however, does agree with the observed maximum water surface elevation and was within +0.12 foot and -0.17 foot for the primary and auxiliary gage, respectively.

4.2.2. Phase II—Verification and Final Calibration of Peak Flow Model

On June 4, 2008, the Pend Oreille River crested at a peak inflow of approximately 96,000 cfs[4]. During this period of high water, the pressure transducers installed throughout the study area were recording 15-minute water surface elevations. Prior to this event, there was limited data available at and below Metaline Falls for calibration purposes. Both the Canyon Reach and Metaline Falls greatly influences the hydraulic conditions above the falls. A verification simulation using the Phase I calibrated model suggested that additional calibration of the hydraulic routing model was necessary at Metaline Falls. The 2008 high flow period was incorporated into the calibration matrix of events and used to generate a final calibrated parameter set.

The verification simulation indicated that adjustments upwards of the Manning's roughness coefficients were necessary within the Canyon Reach of the Project. Water surface elevations within the Canyon Reach provide the tail water elevation of Metaline Falls. The increase in energy losses through the canyon generated a reduction in conveyance capacity through the falls. Manning's roughness coefficients at the falls were overestimating energy losses and were subsequently reduced to match observed water surface elevations at the US_MET pressure transducer. Further adjustment of upstream Manning's roughness coefficients was necessary to generate a model capable of producing results within the specified tolerances discussed earlier in Section 4.2.1.

Adjustments to the channel and overbank Manning's roughness coefficients were made until the model simulations reproduced the observed water surface elevations within the specified tolerance. Table 4.2-3 presents a summary of the absolute error during the calibration events. Absolute differences between model-simulated and observed water surface elevations for the final calibration matrix shown in Table 4.2-3 were within the pre-established tolerances except for the 1972 and 2008 events.

[4] At the time of this report, the peak discharge that occurred during the 2008 water year is provisional and is based on data supplied by the USGS.

Table 4.2-3. Final calibrated magnitude of maximum difference between observed and model-simulated water surface elevation for each calibration event.

High Flow Event		Box Canyon Tailrace Pressure Transducer (HEC-RAS Station 102198[1])	USGS 1239600 Primary Gage (HEC-RAS Station 101240[1])	USGS 12396500 Auxiliary Gage (HEC-RAS Station 96759[1])	Upstream of Metaline Falls Pressure Transducer (HEC-RAS Station 61170[1])	Downstream of Metaline Falls Pressure Transducer (HEC-RAS Station 59451[1])	Canyon Pressure Transducer (HEC-RAS Station 12445[1])
2008	**Maximum Positive (feet)**	+0.98	+0.86	+1.06	+0.92	+0.58	+0.08
	Maximum Negative (feet)	-0.77	-0.34	-0.25	-0.36	-0.58	-0.09
1997	**Maximum Positive (feet)**	N/A	+0.28	+0.22	N/A	N/A	N/A
	Maximum Negative (feet)	N/A	-0.83	-0.83	N/A	N/A	N/A
1974	**Maximum Positive (feet)**	N/A	+0.50	+0.43	N/A	N/A	N/A
	Maximum Negative (feet)	N/A	-0.12	-0.21	N/A	N/A	N/A
1972	**Maximum Positive (feet)**	N/A	+0.22	+0.17	N/A	N/A	N/A
	Maximum Negative (feet)	N/A	-1.06	-1.11	N/A	N/A	N/A

Notes:
1 Table includes identifiers for HEC-RAS model cross sections associated with each calibration location.
N/A – not applicable; transducers not installed until 2006

RMSE between the model-simulated and observed is shown in Table 4.2-4. The RMSE for the final calibration matrix was within pre-established tolerances of 0.5 foot for all peak flow events.

Table 4.2-4. Final RMSE at each calibration location for each calibration event.

High Flow Event	Box Canyon Tailrace Pressure Transducer (HEC-RAS Station 102198[1]) RMSE (feet)	USGS 1239600 Primary Gage (HEC-RAS Station 101240[1]) RMSE (feet)	USGS 12396500 Auxiliary Gage (HEC-RAS Station 96759[1]) RMSE (feet)	Upstream of Metaline Falls Pressure Transducer (HEC-RAS Station 61170[1]) RMSE (feet)	Downstream of Metaline Falls Pressure Transducer (HEC-RAS Station 59451[1]) RMSE (feet)	Canyon Pressure Transducer (HEC-RAS Station 12445[1]) RMSE (feet)
2008	0.31	0.35	0.46	0.35	0.21	0.03
1997	N/A	0.30	0.35	N/A	N/A	N/A
1974	N/A	0.13	0.12	N/A	N/A	N/A
1972	N/A	0.44	0.45	N/A	N/A	N/A

Notes:
1 Table includes identifiers for HEC-RAS model cross sections associated with each calibration location.
N/A – not applicable

As previously discussed during the Phase I calibration, discrepancies found between observed water surface elevations and flow hydrographs during the rising limb of the 1972 flood are likely the cause for the exceedance in tolerance of the absolute difference calculated for this event. Elevated absolute difference measurements during the 2008 calibration event occurred during flows below the lower bounds of the model's applicable flow range (approximately 80,000 cfs); however, as flows increased towards flood conditions, the absolute error decreased. Table 4.2-5 summarizes the distribution of differences between model-simulated and observed water surface elevations during the 2008 period of high water. As displayed within Table 4.2-5, the difference in model-simulated water surface elevations at the USGS gage station 12396500 auxiliary gage during the period of flows between 80,000 and 100,000 cfs are within 1.0 foot of observed.

Table 4.2-5. Absolute error distribution during the 2008 period of high water at USGS gage station 12396500 (auxiliary).

	Percent of Time Records Occur for a Given Absolute Difference							
Absolute Error (feet)	**0.00**	**0.01**	**0.05**	**0.10**	**0.25**	**0.50**	**0.75**	**1.00**
Flow Range (cfs)								
60,000 to 70,000	0	0	0	0	0	31	69	0
70,000 to 80,000	0	0	1	12	10	22	39	16
80,000 to 90,000	2	7	12	33	27	11	8	0
90,000 to 100,000	1	2	4	29	54	11	0	0

Comparison of observed high water marks with model-simulated water surface elevations is presented in Table 4.2-6. The estimated water surface elevations, based on model simulations, were within 0.4 foot of the observed high water marks.

Table 4.2-6. Difference in model-simulated versus observed high water marks for a given calibration event.

High Flow Event	Location	Date	Observed Water Surface Elevation (feet NAVD 88)	Simulated Water Surface Elevation (feet NAVD 88)	Difference (feet)
2008	Metaline Park	5/30/2008	2,007.78	2,007.55	-0.23
2008	McKenzie's House	6/7/2008	2,009.62	2,009.26	-0.36
1997	McKenzie's House	6/7/1997	2,016.52	2,016.19	-0.33
1997	McKenzie's House	6/10/1997	2,015.83	2,016.03	+0.20

The purpose of the Peak Flow Model is to simulate the maximum inundation extents during these highest flows. Peak water surface elevations reported by the Peak Flow Model were in general agreement with the observed records for all events. Table 4.2-7 summarizes the model-simulated water surface elevation and the observed values. The Peak Flow Model was able to reproduce the peak water surface elevations within 0.25 foot of observed and represents a well-calibrated model. An example at one location during the 2008 high flow event at the US_MET pressure transducer located at the Highway 31 Bridge is shown in Figure 4.2-1. Hydrograph comparison plots at each of the calibration locations for each of the calibration events are available in Appendix 3.

Table 4.2-7. Difference in model-simulated versus observed maximum water surface elevation for a given calibration event.

High Flow Event	Box Canyon Tailrace Pressure Transducer (HEC-RAS Station 102198[1]) Difference in Peak Water Surface Elevation (feet)	USGS 1239600 Primary Gage (HEC-RAS Station 101240[1]) Difference in Peak Water Surface Elevation (feet)	USGS 12396500 Auxiliary Gage (HEC-RAS Station 96759[1]) Difference in Peak Water Surface Elevation (feet)	Upstream of Metaline Falls Pressure Transducer (HEC-RAS Station 61170[1]) Difference in Peak Water Surface Elevation (feet)	Downstream of Metaline Falls Pressure Transducer (HEC-RAS Station 59451[1]) Difference in Peak Water Surface Elevation (feet)	Canyon Pressure Transducer (HEC-RAS Station 12445[1]) Difference in Peak Water Surface Elevation (feet)
2008	-0.21	+0.16	+0.18	+0.04	+0.01	+0.04
1997	N/A	-0.17	-0.22	N/A	N/A	N/A
1974	N/A	+0.04	-0.05	N/A	N/A	N/A
1972	N/A	-0.07	-0.17	N/A	N/A	N/A

Notes:
1 Table includes identifiers for HEC-RAS model cross sections associated with each calibration location.
N/A – not applicable

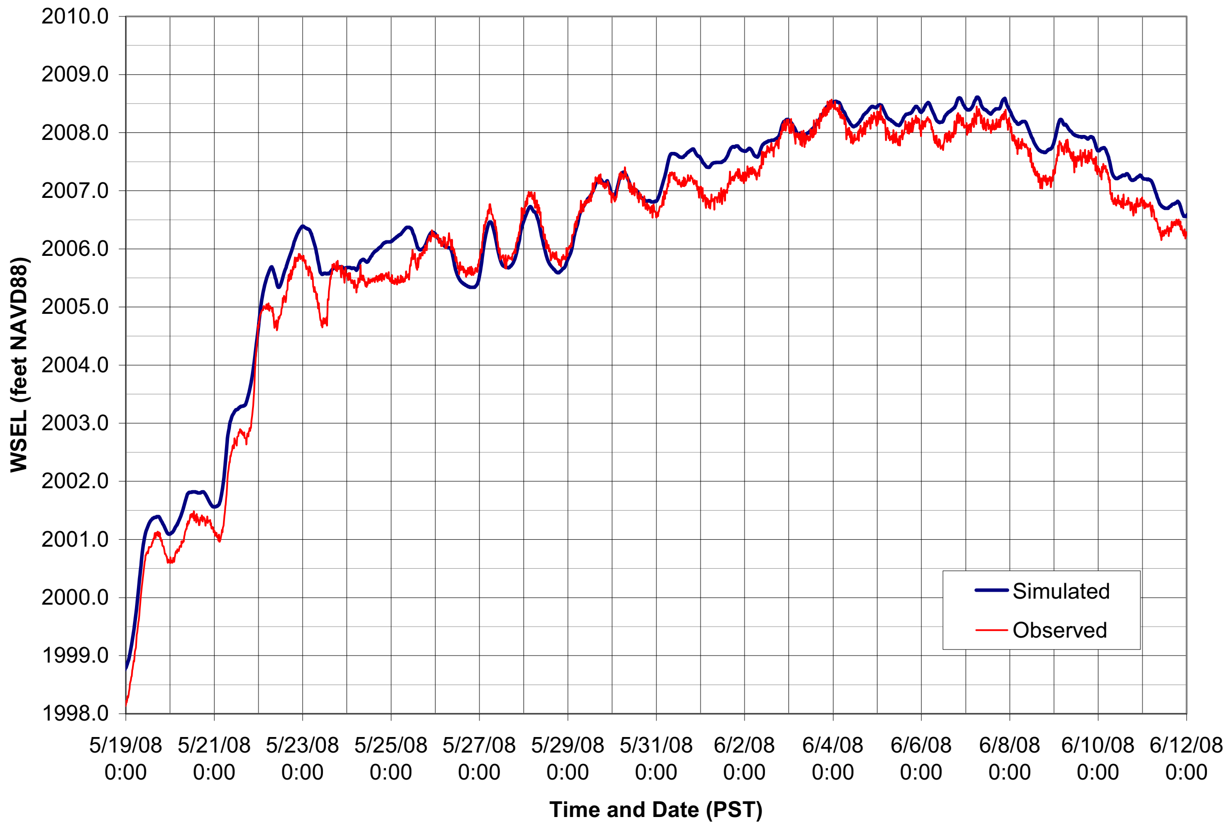

Figure 4.2-1. Model-simulated versus observed water surface elevation hydrograph at the US_MET pressure transducer during the 2008 calibration event.

The final calibrated parameter set used to generate the results tabulated above is presented in Table 4.2-8. Manning's roughness coefficients below Metaline Falls increased slightly from the initial values. Values within the town of Metaline Falls increased while Manning's roughness coefficients near Box Canyon Dam decreased. Contraction and expansion coefficients did not require adjustment.

Table 4.2-8. Final model calibration parameters used for Peak Flow Model.

Upstream HEC-RAS Cross Section[1]	Downstream HEC-RAS Cross Section[1]	Upstream Project River Mile[2]	Downstream Project River Mile[2]	Manning's Roughness			Contraction Coefficient	Expansion Coefficient
				Left Overbank[3]	Channel	Right Overbank[3]		
102198	99871	34.39	33.98	0.041	0.041	0.042	0.3	0.5
99871	98093	33.98	33.64	0.052	0.026	0.095	0.3	0.5
98093	95152	33.64	33.11	0.081	0.040	0.076	0.3	0.5
95152	91365	33.11	32.44	0.075	0.040	0.105	0.3	0.5
91365	84450	32.44	31.18	0.058	0.036	0.102	0.3	0.5
84450	72815	31.18	29.09	0.049	0.041	0.043	0.3	0.5
72815	60912	29.09	26.98	0.059	0.040	0.076	0.3	0.5
60912	60053.34	26.98	26.81	0.093	0.093	0.093	0.6	0.7
60053.34	59566.67	26.81	26.72	0.122	0.119	0.122	0.6	0.7
59566.67	12256	26.72	18.03	0.096	0.096	0.096	0.3	0.5
12256	5428	18.03	17.02	0.029	0.029	0.029	0.1	0.3

Notes:
1 Refer to figures in Appendix 1 for HEC-RAS cross section locations.
2 Project river miles were based on linear interpolation between Project river mile identifiers at 0.1-mile increments.
3 Represented conveyance weighted, average overbank Manning's roughness coefficient.

4.3. Development of Inundation Mapping

Inundation maps of the 1972, 1974, and 1997 periods of high flow were generated from modeling results from the calibrated Peak Flow Model. The generation of inundation maps was facilitated by using HEC-GeoRAS extension within GIS (USACE 2005). HEC-GeoRAS provides pre-processing and post-processing tools for the development and transfer of geospatial data into and out of HEC-RAS. Using HEC-GeoRAS, the water surface elevation data estimated, based on the model simulations, at each cross section for each of the historical events was exported from the Peak Flow Model into GIS. The maximum inundation extents were displayed using tools within HEC-GeoRAS.

As described in Section 5.1.2, additional inundation maps were generated to show the maximum estimated inundation extents without the potential hydraulic effects of Project operations for the inflow hydrographs of the 1972, 1974, and 1997 historical high flow events. A comparison of the two inundation mapping sets was performed to assess the estimated Project effects above Metaline Falls.

4.4. Model Documentation and Executable Model

An executable hydraulic routing model was constructed, and supporting documentation has been included within this report. The model was used to assess hydraulic conditions experienced during the 1972, 1974, and 1997 high flow events. The model was calibrated with observed flows ranging from approximately 60,000 to 136,000 cfs and observed Project forebay elevations varying from

1,983.70 to 1,993.17 feet NAVD 88 (1,979.67 to 1,989.14 feet NGVD 29). The model is referred to as the Peak Flow Model. It is also applied in other studies, primarily Study 7, to determine water surface elevations for flows above 80,000 cfs. For flows below 80,000 cfs, mainstem water surface elevations will be modeled using the Study 7 Boundary Reservoir HRM (SCL 2009a).

The calibrated model was used to assess the hydraulic influence of Project operations above Metaline Falls. This was accomplished by modeling hydraulic conditions with and without the influence of Project operations during the three high flow events. The removal of the hydraulic influences from Project operations was determined by iterative adjustments to the Project forebay elevations used in the Peak Flow Model. Adjustments were made until the model-simulated change in water surface elevations above Metaline Falls associated with Project operations was less than 0.1 foot. Comparisons of the resulting model-simulated water surface profiles for with and without the influence of Project operations for each of the historic high flow events were made to determine the effect of the Project above Metaline Falls.

Model-simulated water surface elevations from the Peak Flow Model were exported from the hydraulic routing model using HEC-GeoRAS into GIS maps. Maps were generated to illustrate areas inundated for the 1972, 1974, and 1997 events above Metaline Falls.

5 RESULTS

This section summarizes the development of inundation mapping for the three highest historical flow events since completion of the Project in 1967, and assessment of the estimated effects of Project operations on the extents and duration of inundation. Model-simulated results from the calibrated Peak Flow Model were used in combination with HEC-GeoRAS to generate inundation maps of the 1972, 1974, and 1997 high flow events. These events correspond to the periods of high water as indicated during the FERC scoping meeting (FERC 2006b), when inundation at Mr. McKenzie's house and within the town of Metaline occurred. The model was also used to estimate the potential hydraulic effect of existing Project operations above Metaline Falls.

5.1. Inundation during High Flow Periods

The following sections contain the results of the inundation mapping above Metaline Falls using model-simulated results from the calibrated Peak Flow Model. Inundation maps have been generated to show the maximum simulated inundation extent during the 1972, 1974, and 1997 periods of high water. Additional maps have been generated from the model to show the estimated maximum inundation extent if the hydraulic influence of Project operations is removed.

5.1.1. Inundation Extent With Hydraulic Influence of Project Operations

An example of the maximum model-simulated inundation extent during the 1997 period of high water within the town of Metaline is shown in Figure 5.1-1. Additional figures showing the simulated extents of inundation in the entire Upper Reservoir Reach for the 1972, 1974, and 1997 high flow periods can be found in Appendix 4.

The cross section located at Mr. McKenzie's house (PRM 28.9, HEC-RAS Station 71550) was used for the basis of comparing model-simulated water surface elevations for conditions representing the influence, with versus without, of Project operations. Table 5.1-1 summarizes the model-simulated water surface elevation within the town of Metaline for the 1972, 1974, and 1997 periods of high water. Table 5.1-1 also includes the conditions experienced at the Project forebay coincidental to the peak model-simulated water surface elevation at the town of Metaline.

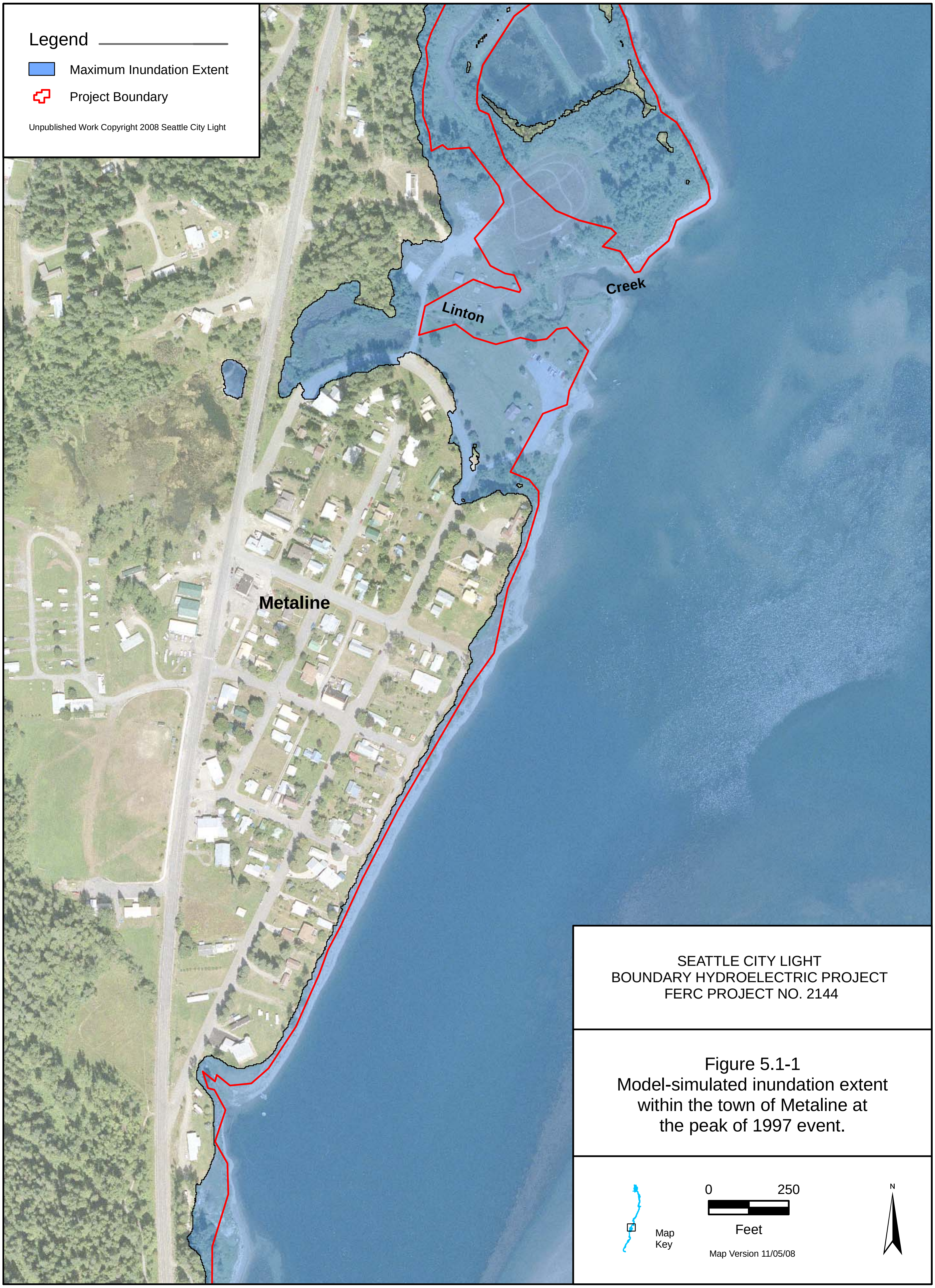

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Figure 5.1-1
Model-simulated inundation extent within the town of Metaline at the peak of 1997 event.

Table 5.1-1. Summary information of the model-simulated maximum water surface elevation within the town of Metaline during the historical high flow events.

Event	Peak Discharge (cfs)	Forebay Elevation Fluctuation during Peak[1] (feet NAVD 88)	Forebay Elevation at Peak Water Surface Elevation[2] (feet NAVD 88)	Maximum Model-simulated Water Surface Elevation at the Town of Metaline (PRM 28.9, HEC-RAS Station 71550[3]) (feet NAVD 88)
1997	134,000	1,987.56 – 1,989.65	1,987.76	2,016.19
1974	133,000	1,987.07 – 1,987.22	1,987.17	2,015.54
1972	136,000	1,987.65 – 1,987.74	1,987.70	2,016.12

Notes:
1 Forebay elevation range ±1 day of peak water surface elevation at McKenzie's house.
2 Peak water surface elevation modeled at McKenzie's house (PRM 28.9, HEC-RAS Station 71550)
3 Refer to figures in Appendix 1 for HEC-RAS cross section locations.

5.1.2. Inundation Extent Without Hydraulic Influence of the Project Operations

The development of the simulated inundation mapping with the hydraulic influence of Project operations removed was generated by first using the model to estimate the conditions (i.e., forebay elevations) under which Project operations no longer influence water surface elevation above Metaline Falls. Hydraulic influences from Project operations were simulated for the peak flows analyzed in this study by iterative adjustments to the Project forebay elevations used in the model until the model-simulated change in water surface elevations above Metaline Falls associated with Project operations was less than 0.1 foot. The value of 0.1 foot selected as the threshold for Project influence is consistent with the level of accuracy associated with model calibration. The model was calibrated to an accuracy of approximately ±0.2 foot at peak flows with minimum error on the order of 0.1 foot. Calibration of the model over a range of flows produced root mean error ranging from 0.12 foot to 0.46 foot (see Section 4.2 for details on model calibration). Selecting a threshold value near the highest level of accuracy expected from the model (e.g., 0.1 foot) is conservative in that it results in a greater lowering of the forebay level to define the limit of Project influence than if a larger value (e.g., 0.5 foot) were chosen. On the other hand, selecting a value of the threshold well beyond the potential accuracy of the model would be unreasonable because it would imply a false precision in the results.

When inflow from Box Canyon to Boundary reservoir is 140,000 cfs, this iterative simulation suggests that a forebay elevation of approximately 1,957.03 feet NAVD 88 (1,953 feet NGVD 29) would be the elevation at which the hydraulic influence from Project operations would be less than 0.1 foot within the Upper Reservoir Reach. An inflow of 140,000 cfs was selected because it represented the approximate maximum observed peak flow rate for the three peak flow years analyzed in this study, which were the three highest flows since the completion of the Project in 1967. At lower inflow rates to the reservoir, the Project has less effect on water surface elevations above Metaline Falls at 1,957.03 feet NAVD 88 (1,953 feet NGVD 29). Subsequently, the point where the hydraulic influence of Project operations is less than 0.1 foot at lower flows (less than 140,000 cfs) is at a higher forebay elevation. It should be noted that the actual forebay elevations during the events used to calibrate the Peak Flow Model were much higher than 1,957.03 feet NAVD 88 (1,953 feet NGVD 29) (see Table 4.1-3). Additionally, it should be noted that the turbines in the existing configuration require a minimum operating elevation of 1,964.03 feet NAVD 88 (1,960 feet NGVD 29). Power generation at a forebay elevation of 1,957.03 feet NAVD 88 (1,953 feet NGVD 29) would result in air entrainment into the draft tubes and cavitation of the turbines.

Based on the conditions identified above, inundation mapping of the model-simulated water surface elevations above Metaline Falls without the influence of Project operations was then conducted. Based on the 0.1-foot criteria established above, at flows reaching approximately 140,000 cfs, a forebay elevation of 1,957.03 feet NAVD 88 (1,953 feet NGVD 29) would produce inundation extents above Metaline Falls reflecting conditions without the influence of Project operations. Because this elevation was near the maximum reservoir drawdown level as authorized under the current license (an elevation of 1,954.03 feet NAVD 88 [1,950 feet NGVD 29]), the lower limit in the license was used for the comparative modeling and mapping exercise.

An example of the model-simulated inundation extent estimated to occur during the 1997 period of high water without the hydraulic influences of Project operations at the town of Metaline is shown in Figure 5.1-2. The model-simulated inundation extents resulting from the inflow hydrographs for the 1972, 1974, and 1997 peak flow events without the influence of Project operation can be found in Appendix 5. Table 5.1-2 summarizes the model-simulated water surface elevations at the town of Metaline without the influence of Project operations.

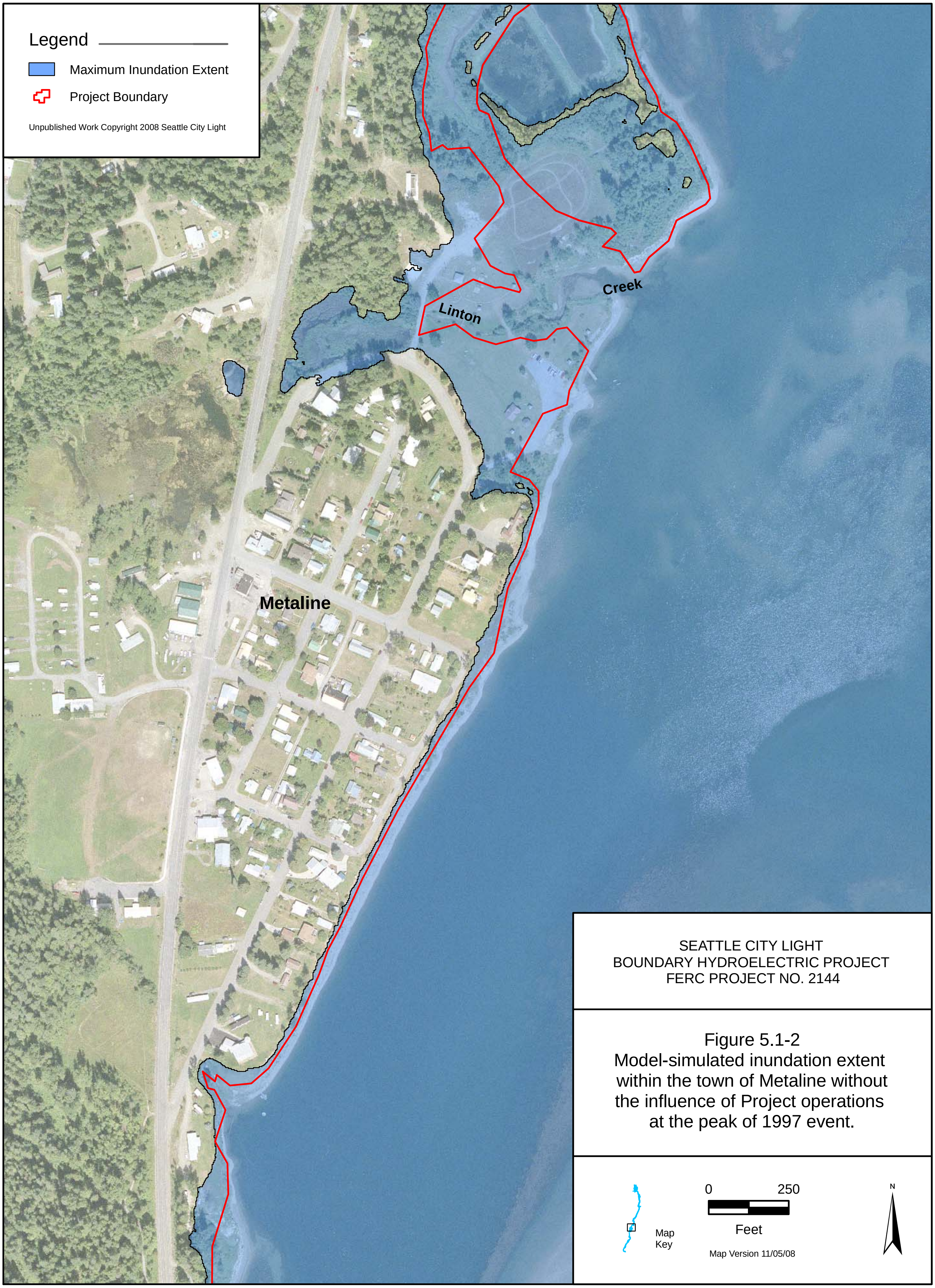

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Figure 5.1-2
Model-simulated inundation extent within the town of Metaline without the influence of Project operations at the peak of 1997 event.

Map Version 11/05/08

Table 5.1-2. Summary information of the model-simulated maximum water surface elevation within the town of Metaline during the historical high flow events without the influence of Project operations.

Event	Peak Discharge (cfs)	Model-simulated Forebay Elevation at Peak Water Surface Elevation (feet NAVD 88)	Maximum Model-simulated Water Surface Elevation at the Town of Metaline (PRM 28.9, HEC-RAS Station 71550[1]) (feet NAVD 88)
1997	134,000	1,954.03	2,014.63
1974	133,000	1,954.03	2,014.20
1972	136,000	1,954.03	2,014.69

Notes:
1 Refer to figures in Appendix 1 for HEC-RAS cross section locations.

5.2. Estimated Project Effect

An assessment of the estimated Project effect on water surface elevations and duration of inundation above Metaline Falls during the three highest peak flow events since Project completion in 1967 was conducted using the Peak Flow Model and the inundation mapping developed using HEC-GeoRAS. Analyses comparing the maximum extent of inundation with and without the influence of Project operation were used to estimate the effects of the Project. Table 5.2-1 summarizes the difference in model-simulated water surface elevation within the town of Metaline during the 1972, 1974, and 1997 periods of high water, and indicates the estimated Project effect. An example of the estimated Project effect in the town of Metaline is shown in Figure 5.2-1. Additional mapping can be found in Appendix 6.

Table 5.2-1. Summary information of model-simulated maximum water surface elevation within the town of Metaline during the historical high flow events with and without the influence of Project operations.

High Water Year	Peak Discharge at USGS Gage 12396500 (cfs)	Forebay Elevation at Peak Water Surface Elevation (ft NAVD 88)	Forebay Elevation Fluctuation during Peak[1] (ft NAVD 88)	Difference in Forebay Elevation (ft)	Model-simulated Peak Water Surface Elevation at town of Metaline (PRM 28.9, HEC-RAS Station 71550)[2]		
					With Project Effects (ft NAVD 88)	Without Project Effects (ft NAVD 88)	Estimated Project Effects (Difference in ft)
1997	134,000	1,987.76	1,987.56 – 1,989.65	1.89	2,016.19	2,014.63	+1.56
1974	133,000	1,987.17	1,987.07 – 1,987.22	0.15	2,015.54	2,014.20	+1.34
1972	136,000	1,987.70	1,987.65 – 1,987.74	0.09	2,016.12	2,014.69	+1.43

Notes:

1 Forebay elevation range ±1 day of peak water surface elevation at McKenzie's house (PRM 28.9, HEC-RAS Station 71550).

2 Refer to figures in Appendix 1 for HEC-RAS cross section locations.

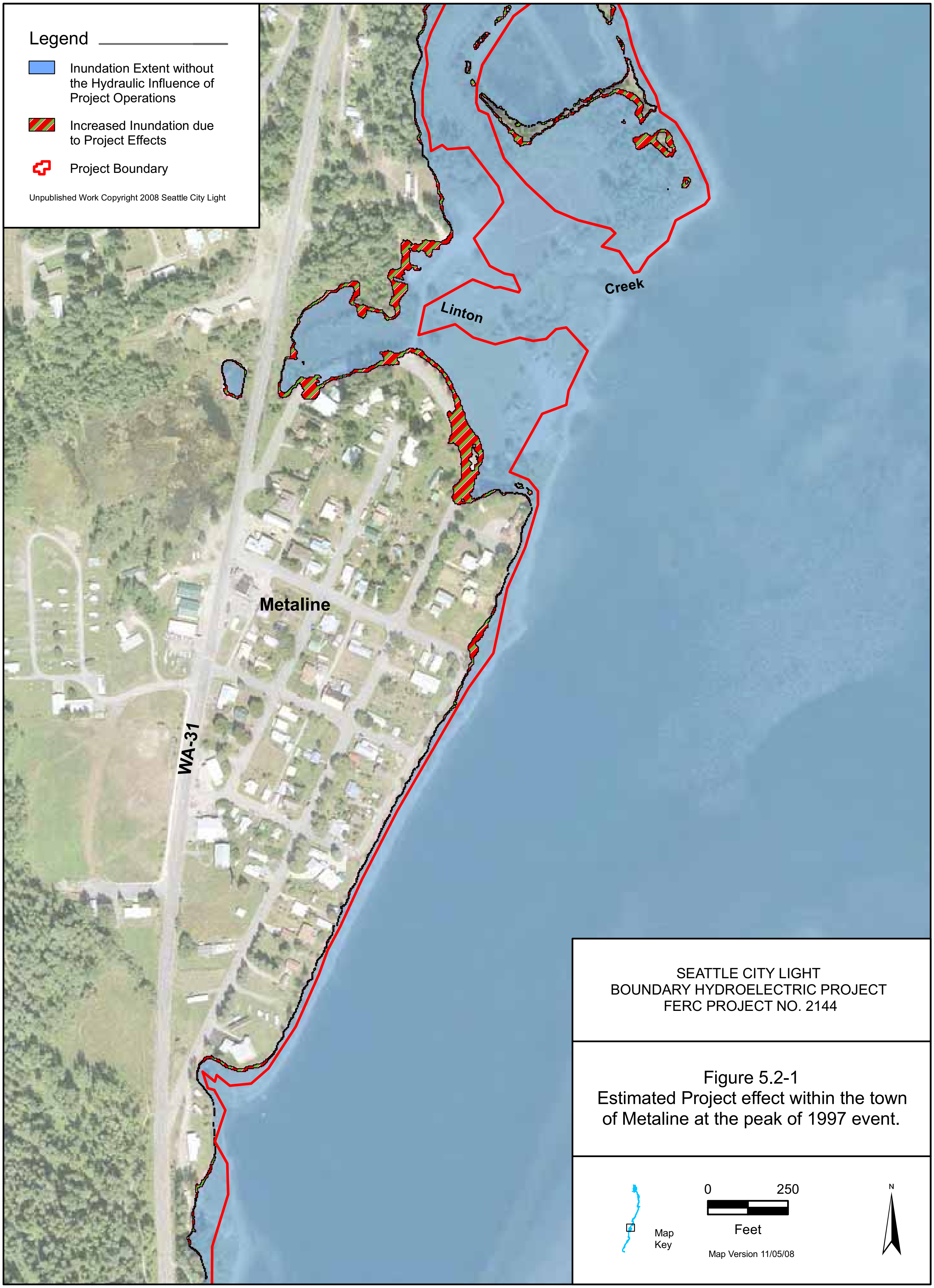

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Figure 5.2-1
Estimated Project effect within the town of Metaline at the peak of 1997 event.

The model simulation results displayed in Table 5.2-1 indicate that Project operations appeared to influence the peak water surface elevation within the town of Metaline during the historical peak flow periods. It is estimated that an additional 1.3 to 1.6 feet of inundation were experienced during the 1972, 1974, and 1997 events as the result of Project operations. During the historical events, the Project forebay was operated between 1,987.07 to 1,989.65 feet NAVD 88 (1,983.04 to 1,985.62 feet NGVD 29) at the time of the model-simulated peak stage at McKenzie's house and was below the maximum operating pool level authorized by the current license of 1,994.03 feet NAVD 88 (1,990 feet NGVD 29).

Review of the inundation maps overlaid on recent aerial photography (Terrapoint 2005) suggest that there are currently no habitable structures influenced by the increased inundation caused by the influence of Project operations that are estimated to occur during the highest historical peak flow events, i.e., those approaching 140,000 cfs. Currently, the improved properties affected by increased inundation during these peak flow events consist primarily of lawns, naturally vegetated areas, and recreational areas such as Metaline Park.

Model-simulated durations of inundation occurring above particular elevations due to the influence of Project operations for each of the three historical events are presented in Table 5.2-2. In general, the duration of inundation above specific elevations appears to be increased by the influence of Project operations. In the case of the 1997 event, the estimated maximum increase in duration of inundation is +24.2 days, occurring between the range of 2,014.5 to 2,015 feet NAVD 88 (2,010.47 to 2,010.97 feet NGVD 29). The estimated maximum increase in duration of inundation during the 1972 and 1974 high flow events is significantly less than the 1997 high flow event. The estimated increase in duration of inundation during the 1972 event and 1974 event was +6.1 and +6.7 days, respectively.

Table 5.2-2. Comparison of model-simulated duration of inundation within the town of Metaline during the historical high flow events with and without influence of Project operations.

Model-simulated Water Surface Elevation (ft NAVD 88)	Model-simulated Duration of Inundation Above Given Water Surface Elevation								
	1997 High Water			1974 High Water			1972 High Water		
	With Project Influence (Days)	Without Project Influence (Days)	Estimated Project Effect (Days)	With Project Influence (Days)	Without Project Influence (Days)	Estimated Project Effect (Days)	With Project Influence (Days)	Without Project Influence (Days)	Estimated Project Effect (Days)
2,016.5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2,016.0	1.0	N/A	+1.0	N/A	N/A	N/A	2.7	N/A	+2.7
2,015.5	10.4	N/A	+10.4	0.7	N/A	+0.7	3.9	N/A	+3.9
2,015.0	17.7	N/A	+17.7	5.4	N/A	+5.4	6.1	N/A	+6.1
2,014.5	24.8	0.6	+24.2	6.7	N/A	+6.7	7.9	2.8	+5.1
2,014.0	29.2	7.9	+21.3	8.3	3.3	+4.9	9.1	5.2	+3.9
2,013.5	31.0	13.9	+17.1	10.0	5.8	+4.2	10.8	6.2	+4.6
2,013.0	32.7	23.7	+9.1	10.9	7.3	+3.7	12.1	7.8	+4.3
2,012.5	34.5	28.8	+5.7	11.8	9.1	+2.7	13.5	8.7	+4.8
2,012.0	35.4	31.4	+4.0	12.6	10.5	+2.1	14.4	10.4	+4.0
2,011.5	36.2	33.9	+2.3	13.4	11.5	+1.9	15.2	12.1	+3.1
2,011.0	36.9	35.7	+1.3	14.1	12.4	+1.8	16.2	13.1	+3.1
2,010.5	37.8	36.7	+1.1	14.9	13.2	+1.7	17.5	13.9	+3.6
2,010.0	38.4	37.4	+1.0	16.1	13.8	+2.3	18.6	14.7	+3.9

Note:
N/A – not applicable

5.3. Potential Procedures to Attenuate Flooding Conditions Attributable to Project Operations

Results from the Peak Flow Model indicate that during the highest peak flow events on record since Project completion, an increase in flooding conditions due to the influence of Project operations occurs. Procedures that might attenuate future Project-related flooding within the town of Metaline during such peak flow events were evaluated. Review of the modeling results suggests that effects from Project operations during flows of approximately 140,000 cfs would be negligible (0.1 foot or less) if reservoir forebay elevations were lowered to 1,957.03 feet NAVD 88 (1,953 feet NGVD 29). However, to allow for power generation, a minimum operating elevation of 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) is required. Based on this constraint, the model results indicate it may be impracticable to lower the forebay sufficiently to completely attenuate Project-related contributions to flooding above Metaline Falls.

It should be noted that the estimated elevation of 1,957.03 feet NAVD 88 (1,953 NGVD 29) is based on model results assuming the forebay elevation was lowered approximately 27 feet below the actual conditions upon which the Peak Flow Model was calibrated. Considering the complexity of hydraulic conditions in the Canyon Reach and Metaline Falls, there is some uncertainty in assigning this elevation. Further testing during high flows to evaluate the effects of lower forebay elevations would be required to support conclusions regarding the efficacy of any operational procedures designed to significantly reduce the hydraulic influences from Project operations above Metaline Falls. Depending on the inflow to the Project, such testing could provide information to better calibrate the model for lower forebay elevations or to evaluate actual operations.

Because a minimum operating elevation of 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) is required for power generation, elevations below 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) were not considered in the assessment of potential procedures to attenuate Project-related contributions to flooding conditions. Options assessed include the following:

- Option 1—Maintain a forebay elevation at or below 1,983.03 feet NAVD 88 (1,979 feet NGVD 29) during flood flows (i.e., flows over 120,000 cfs) to keep the Project-related increase in peak flood elevation above Metaline Falls to less than 1 foot.
- Option 2—Maintain a forebay elevation at or below 1,975.03 feet NAVD 88 (1,971 feet NGVD 29) during flood flows to keep the increase in peak flood elevation above Metaline Falls to less than 0.5 foot.
- Option 3—Maintain a forebay elevation at or below 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) during flood flows to keep the increase in peak flood elevation above Metaline Falls to less than 0.2 foot.

Specifics regarding each of the potential procedures to attenuate Project-related contributions to flooding conditions as outlined above are described in further detail in the following sections. The operational feasibility, effects on generation, and the cost of implementing each of the procedures have been assessed. Operational and generation effects include the impacts to voluntary minimum pool restrictions, the reduction in power generations, and the impacts to recreational use of the Forebay and Canyon reaches. Impacts of the different procedures described below on other resources within the Boundary Project area (e.g., fish habitat) are not considered in this report. The impacts not considered may be revisited to evaluate the potential effect on other resources as part of actual testing that is discussed above.

5.3.1. Option 1—Maintain a Forebay Elevation of 1,983.03 Feet NAVD 88

The first option considered includes maintaining the Project forebay elevation at or below 1,983.03 feet NAVD 88 (1,979 feet NGVD 29) during flood conditions (i.e., when inflow to Boundary reservoir exceeds 120,000 cfs). It is estimated that this could attenuate Project-related contribution to flooding to less than 1 foot. It order to bring the forebay to an elevation of 1,983.03 feet NAVD 88 (1,979 feet NGVD 29), throttling (i.e., varying the gate openings) of the sluice gates would be required due to insufficient capacity through the two spillways and power plant. At this forebay elevation, a maximum discharge through the two spillways of 65,000 cfs was estimated. Conveyance through the power plant was assumed to be 53,800 cfs. The remaining inflow into Boundary Reservoir would need to pass through the sluiceway.

The sluice gates were designed to operate in either the fully open or fully closed positions. In order to provide the necessary conveyance during flood conditions while maintaining the targeted forebay elevation, two sluice gates would need to be modified to allow for throttling. Previous tests of the sluice gates at partial openings demonstrated problems with flow interactions with slot heater covers and large sprays that are disruptive and may be causing damage to concrete near the gate seals.

5.3.1.1. Cost of Sluice Gate Modifications

The cost associated with modifying a single gate is based on an engineer's cost estimate conducted as part of Study 3, Evaluation of Total Dissolved Gas and Potential Abatement Measures Study Final Report (SCL 2009b). The cost of modifying a single sluice gate to allow for throttling was estimated to be $1,297,000 in 2007 dollars. The total cost of modifying the two sluice gates for this option is $2,594,000. This estimate was based on "screening level" alternative layouts and intended for comparison purposes as presented in Study 3.

Construction for this option includes continuing with the modifications to the gate seals and heater plates and the installation of larger sealing plates on the faces of the gates. Work could be performed incrementally, spreading the associated cost over any period predetermined by SCL. This work could be performed any time after the high flows have passed for the year. Work could also be performed through the winter months because the relatively small work areas could be quite easily heated. Additional information regarding the specifics of the sluice gate modifications are presented in Study 3.

5.3.1.2. Cost to Operations

The reduction in head would impact power generation at the Project. Water surface elevations would be approximately 4 to 5 feet lower than what typically occurs during flood conditions. The overall reduction in capacity would be approximately 25 megawatts (MW) if the forebay were maintained at an elevation of 1,983.03 feet NAVD 88 (1,979 feet NGVD 29) rather than the current level of about 1,987.03 to 1,989.03 feet NAVD 88 (1,983 to 1,985 feet NGVD 29). For hydrologic conditions such as occurred in 1972, 1974, and 1997, the lost energy would equate to 6,400 megawatt hour (MWh), 5,700 MWh, and 20,900 MWh, respectively[5]. The cost associated with the loss in revenue during a single hydrologic event with the forebay restrictions during flood conditions in-place as well as the capital cost associated with this option is summarized within Table 5.3-1.

5.3.1.3. Estimated Cost for Option 1

The revenue loss estimated from forebay restrictions coinciding with hydrologic conditions experienced during the 1972, 1974, and 1997 events as well as the capital cost associated with the implementation of Option 1 is presented in 2007 dollars in Table 5.3-1.

[5] The reduction in capacity estimate is an approximation based on simplifying assumptions and should be used for comparative purposes only. As more sophisticated tools are made available, this estimate could be revisited.

Table 5.3-1. Estimated cost associated with maintaining a forebay elevation of 1,983.03 feet NAVD 88 during the 1972, 1974, and 1997 flood events.

Event	Loss of Revenue due to Reduction in Forebay Elevation for a Single Event [1,3]	Capital Cost associated with Option 1 [2,3]
1972	$303,400	$2,594,000
1974	$271,000	$2,594,000
1997	$988,400	$2,594,000

Notes:

1 Loss of revenue is calculated using an assumed average market cost during May/June timeframe of $47.24 per MWh (2007$).

2 Capital costs associated with option 1 include the modification of 2 sluice gates to allow for throttling of flows during flood conditions (inflows greater than 120,000 cfs).

3 Costs are for comparative purposes only and are based on simplifying assumptions. As more sophisticated models are made available, cost could be revisited.

5.3.2. Option 2—Maintain a Forebay Elevation of 1,975.03 Feet NAVD 88

There is insufficient conveyance capacity through the two spillways and powerhouse to pass flood flows at this forebay elevation. An additional 42,600 cfs will need to be discharged through the sluice way when inflows into Boundary Reservoir are at 140,000 cfs. A total of three sluice gates will require modifications to allow for the throttling of discharge through the gates to achieve this forebay elevation. The cost associated with modifying three gates is estimated to be $3,891,000.

Reservoir access and use at the Forebay Recreation Area would be impacted. Launching of boats is achievable; however, the existing configuration of the boat launch does not provide sufficient depth to launch. Instead, boats are launched from primitive sites within the Forebay Recreation Area. Boats with large drafts are unable to launch from either site. Also, the ability to launch life safety boats within the Forebay and Canyon reaches during flood conditions would not be feasible at this elevation. Improvements to the boat launch to facilitate the launching of boats when forebay conditions are at this water surface elevation are estimated to be $33,700.

5.3.2.1. Cost to Operations

Maintaining a forebay elevation of 1,975.03 feet NAVD 88 (1,971 feet NGVD 29) during flooding conditions could also impact operations during the Memorial Day to Labor Day period when SCL voluntarily restricts the water surface fluctuations to a 10-foot range (between elevations 1,984.03 feet and 1,994.03 feet NAVD 88 [1,980 and 1,990 feet NGVD 29]) to facilitate reservoir access and related recreational activities during daytime hours. For the remainder of the year, the water surface generally fluctuates between elevations 1,994 feet and 1,974 feet NAVD 88 (1,990 and 1,970 feet NGVD 29).

The reduction in head would impact power generation at the Project. Water surface elevations would be approximately 12 to 14 feet lower than what typically occurs during flood conditions. The overall reduction in capacity would be approximately 70 MW if the forebay were maintained at an elevation of 1,975.03 feet NAVD 88 (1,971 feet NGVD 29) rather than the current level of about 1,987.03 to 1,989.03 feet NAVD 88 (1,983 to 1,985 feet NGVD 29). For hydrologic conditions such as occurred in 1972, 1974, and 1997, the lost energy would equate to 18,500 MWh, 17,700 MWh, and 54,800 MWh, respectively. The cost associated with the loss in revenue during a single hydrologic event with the forebay restrictions during flood conditions in-place as well as the capital costs associated with this option is summarized within Table 5.3-2.

5.3.2.2. Estimated Cost for Option 2

The revenue loss estimated from forebay restrictions coinciding with hydrologic conditions experienced during the 1972, 1974, and 1997 events as well as the capital cost associated with the implementation of Option 2 is presented in 2007 dollars in Table 5.3-2.

Table 5.3-2. Estimated cost associated with maintaining a forebay elevation of 1,975.03 feet NAVD 88 during the 1972, 1974, and 1997 flood events.

Event	Loss of Revenue due to Reduction in Forebay Elevation for a Single Event [1,3]	Capital Cost associated with Option 2 [2,3]
1972	$873,700	$3,924,700
1974	$834,900	$3,924,700
1997	$2,590,300	$3,924,700

Notes:
1 Loss of revenue is calculated using an assumed average market cost during May/June timeframe of $47.24 per MWh (2007$).
2 Capital costs associated with option 2 include the modification of 3 sluice gates to allow for throttling of flows during flood conditions (inflows greater than 120,000 cfs) and the extension of the Forebay Recreational Area boat launch.
3 Costs are for comparative purposes only and are based on simplifying assumptions. As more sophisticated models are made available, cost could be revisited.

5.3.3. Option 3—Maintain a Forebay Elevation of 1,964.03 Feet NAVD 88

The last option considered includes maintaining the Project forebay elevation at 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) during flood conditions. It is estimated that this could attenuate Project-related contribution to flooding to less than 0.2 foot.

5.3.3.1. Cost of Sluice Gate Modifications and Other Capital Improvements

Similar to the other options presented above, the lowering of the forebay during flood conditions will require the use of the sluice gates to provide conveyance sufficient to achieve and maintain the desired target elevation. It is estimated that an additional 69,000 cfs will need to be discharged through the sluice way when inflows into Boundary Reservoir are at 140,000 cfs. A total of four sluice gates will require modifications to allow for the throttling of discharge through the gates to achieve this forebay elevation. The cost associated with modifying four gates is estimated to be $5,188,000.

Reservoir access and use at the Forebay Recreation Area would be impacted. Currently, the configuration of the boat launch prohibits launching of boats when forebay elevations are this low. Also, the ability to launch life safety boats within the Forebay and Canyon reaches during flood conditions would not be feasible at this elevation. Improvements to the boat launch to facilitate the launching of boats when forebay conditions are at this water surface elevation are estimated to be $67,500.

5.3.3.2. Cost to Operations

Maintaining a forebay elevation of 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) during flood conditions could also impact operations during the Memorial Day to Labor Day period when SCL voluntarily restricts the water surface fluctuations to a 10-foot range (between elevations 1,984.03 feet and 1,994.03 feet NAVD 88 [1,980-1,990 feet NGVD 29]) to facilitate reservoir access and related recreational activities during daytime hours. For the remainder of the year, the water surface generally fluctuates between elevations 1,994 feet and 1,974 feet NAVD 88 (1,990-1,970 feet NGVD 29).

The reduction in head would impact power generation at the Project. Water surface elevations would be approximately 23 to 25 feet lower than what typically occurs during flood conditions. The overall reduction in capacity would be approximately 130 MW if the forebay were maintained at an elevation of 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) rather than the current level of about 1,987.03 to 1,989.03 feet NAVD 88 (1,983 to 1,985 feet NGVD 29). For hydrologic conditions such as occurred in 1972, 1974, and 1997, the lost energy would equate to 34,700 MWh, 33,700 MWh, and 100,400 MWh, respectively. The cost associated with the loss in revenue during a single hydrologic event with the forebay restrictions during flood conditions in-place as well as the capital costs associated with this option is summarized within Table 5.3-3.

5.3.3.3. Estimated Cost for Option 3

The revenue loss estimated from forebay restrictions coinciding with hydrologic conditions experienced during the 1972, 1974, and 1997 event as well as the capital cost associated with the implementation of Option 3 is presented in 2007 dollars in Table 5.3-3.

Table 5.3-3. Estimated cost associated with maintaining a forebay elevation of 1,964.03 feet NAVD 88 during the 1972, 1974, and 1997 flood event.

Event	Loss of Revenue due to Reduction in Forebay Elevation for a Single Event [1,3]	Capital Cost associated with Option 3 [2,3]
1972	$1,639,700	$5,255,500
1974	$1,592,400	$5,255,500
1997	$4,742,100	$5,255,500

Notes:
1 Loss of revenue is calculated using an assumed average market cost during May/June timeframe of $47.24 per MWh (2007$).
2 Capital costs associated with option 3 include the modification of 4 sluice gates to allow for throttling of flows during flood conditions (inflows greater than 120,000 cfs) and the extension of the Forebay Recreational Area boat launch.
3 Costs are for comparative purposes only and are based on simplifying assumptions. As more sophisticated models are made available, cost could be revisited.

5.3.4. Total Project Cost for Each of the Options

The total cost associated with each of the three options over the potential 50-year term of a future license is presented in 2007 dollars in Table 5.3-4. Flood conditions with a magnitude of 120,000 cfs have approximately a 0.10 probability of exceedance in any one year (SCL 1997). The expected revenue loss in 2007 prices that could occur due to forebay elevation restrictions imposed during flood conditions was estimated based upon the 0.10 annual exceedance probability and the average of the lost generation during the 1972, 1974, and 1997 flood events. A discount rate of 3.12 percent was used to adjust future costs over the 50-year term of the future license to their present value at the beginning of the 50-year license term. The resulting loss of revenue estimated over the 50-year period was aggregated with the capital cost to determine the total cost associated with each option.

Table 5.3-4. Present value of estimated cost over 50-year future license associated with forebay restrictions during flood conditions.

Option	Forebay Elevation during Flood Flow Conditions (120,000 cfs or greater)		Capital Cost Associated with Option (2007 Prices)[1]	Average Loss of Revenue due to Reduction in Forebay Elevation (2007 Prices)[2]	Total Cost for the 50-Year Duration of a Future License (2007 Prices)[3,4,5]
	(ft NAVD29)	(ft NAVD 88)			
1	1,979.00	1,983.03	$2,594,000	$520,900	$3,905,500
2	1,971.00	1,975.03	$3,924,700	$1,433,000	$7,532,000
3	1,960.00	1,964.03	$5,255,500	$2,658,100	$11,946,700

Notes:

1 Capital costs associated with each option are discussed in detail in the preceding sections.

2 Loss of revenue is calculated using an assumed average market cost during a May/June timeframe of $47.24 per MWh (2007$) and is based on the mean loss of revenue estimated during similar hydrologic conditions experienced during the 1972, 1974, and 1997 flood events.

3 Total cost is based on a discount rate of 3.12 percent for a 50-year period of analysis. The potential for revenue loss was assumed to have a 0.10 probability of occurrence for any one year for a 50-year period of analysis. All capital cost were assumed to occur prior to the end of year 1.

4 Costs are for comparative purposes only and are based on simplifying assumptions. As more sophisticated models are made available, cost could be revisited.

5 Cost of inflation was not included in these calculations.

6 CONCLUSIONS

The need to establish the relationship between Project operations and inundation above Metaline Falls was identified during the FERC scoping meeting (FERC 2006b). Study 2 was developed to estimate the potential influence of the Project in regards to the extent and duration of flooding above Metaline Falls. A hydraulic routing model, more specifically the Peak Flow Model, was constructed to simulate the conditions experienced during the periods of high flows in 1972, 1974, and 1997. During the FERC scoping meeting, these flow events were indicated by a local resident to have resulted in flooding within the town of Metaline, more specifically, the McKenzie property.

The development of the Peak Flow Model has allowed potential Project influence above Metaline Falls to be estimated. Based on the model results, Project hydraulic influence for the 1972, 1974, and 1997 historical peak flow events is estimated to have increased water surface elevations within the town of Metaline from 1.3 to 1.6 feet. The maximum estimated incremental increase in duration above an elevation of 2,014.5 feet NAVD 88 (2,010.5 feet NGVD 29) during the 1972, 1974, and 1997 events were approximately 6.1, 6.7, and 24.2 days, respectively.

Review of the inundation maps suggests that there are currently no habitable structures influenced by the increased inundation associated with hydraulic influences of the Project for conditions estimated to have occurred during the historical high flow events approaching 140,000 cfs. The areas affected by increased inundation, as determined by the model simulations, consist primarily of lawns, naturally vegetated areas, and recreational areas such as Metaline Park.

Modeling results suggest that the point at which the hydraulic influences of the Project change the extent and duration of flooding above Metaline Falls is influenced by a combination of inflow and forebay elevation. When flows are approximately 140,000 cfs, a forebay elevation of approximately 1,957.03 feet NAVD 88 (1,953 feet NGVD 29) is the estimated elevation at which the hydraulic influence from the Project above Metaline Falls would be negligible. It appears that reducing the Project forebay to a sufficient elevation would essentially eliminate Project effects on inundation in terms of both duration and elevation during an event similar to those that occurred in 1972, 1974, and 1997. However, power generation at a forebay elevation of 1,957.03 feet NAVD 88 (1,953 feet NGVD 29) would result in cavitation to the turbines. To prevent cavitation, a minimum operating elevation of 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) is required. Procedures to attenuate Project-related contribution to flood conditions within the town of Metaline were based on regulating forebay elevations above this minimum operating elevation of 1,964.03 feet NAVD 88 (1,960 feet HGVD 29).

Options have been assessed to determine the operational feasibility, effects on generation, and the cost of implementing certain procedures to attenuate Project-related contribution to flood conditions above Metaline Falls. Three options were evaluated and include restriction of the maximum forebay elevation to 1,983.03 feet NAVD 88 (1,979 feet NGVD 29), 1,975.03 feet NAVD 88 (1,971 feet NGVD 29), and 1,964.03 feet NAVD 88 (1,960 feet NGVD 29) to attenuate Project-related contribution to flooding to less than 1.0 foot, 0.5 foot, and 0.2 foot, respectively. Total costs for the three options are summarized in Table 6.0-1. In addition, SCL may consider non-operational procedures to mitigate for, or lessen the effect of, Project operations on flooding conditions above Metaline Falls. The efficacy of implementing new operational procedures during flood events as well as possible non-operational measures will be given additional treatment in the license application. Finally, further testing during high flows to evaluate the effects of lower forebay elevations would be required to support conclusions regarding the efficacy of any operational procedures designed to significantly reduce the hydraulic influences from Project operations above Metaline Falls because the model has not been calibrated for pool levels below 1,983.70 feet NAVD 88 (1,979.67 feet NGVD 29).

Table 6.0-1. Summary of present value of estimated cost over 50-year future license associated with each option.

Option	Forebay Elevation during Flood Flow Conditions (120,000 cfs or greater)		Total Cost for the 50-Year Duration of a Future License (2007 Prices)[1,2,3,4,5]
	(ft NAVD29)	(ft NAVD 88)	
1	1,979.00	1,983.03	$3,905,500
2	1,971.00	1,975.03	$7,532,000
3	1,960.00	1,964.03	$11,946,700

Notes:
1 Inclusive of capital costs and the cost associated with the reduction in generation capacity.
2 Loss of revenue is calculated using an assumed average market cost during a May/June timeframe of $47.24 per MWh (2007$) and is based on the mean loss of revenue estimated during similar hydrologic conditions experienced during the 1972, 1974, and 1997 flood events.
3 Total cost is based on a discount rate of 3.12 percent for a 50-year period of analysis. The potential for revenue loss was assumed to have a 0.10 probability of occurrence for any one year for a 50-year period of analysis. All capital costs were assumed to occur prior to the end of year 1.
4 Costs are for comparative purposes only and are based on simplifying assumptions. As more sophisticated models are made available, cost could be revisited.
5 The cost of inflation was not included in these calculations.

7 VARIANCES FROM FERC-APPROVED STUDY PLAN AND PROPOSED MODIFICATIONS

There were no variances from the approved FERC study plan or proposed modifications to it.

8 REFERENCES

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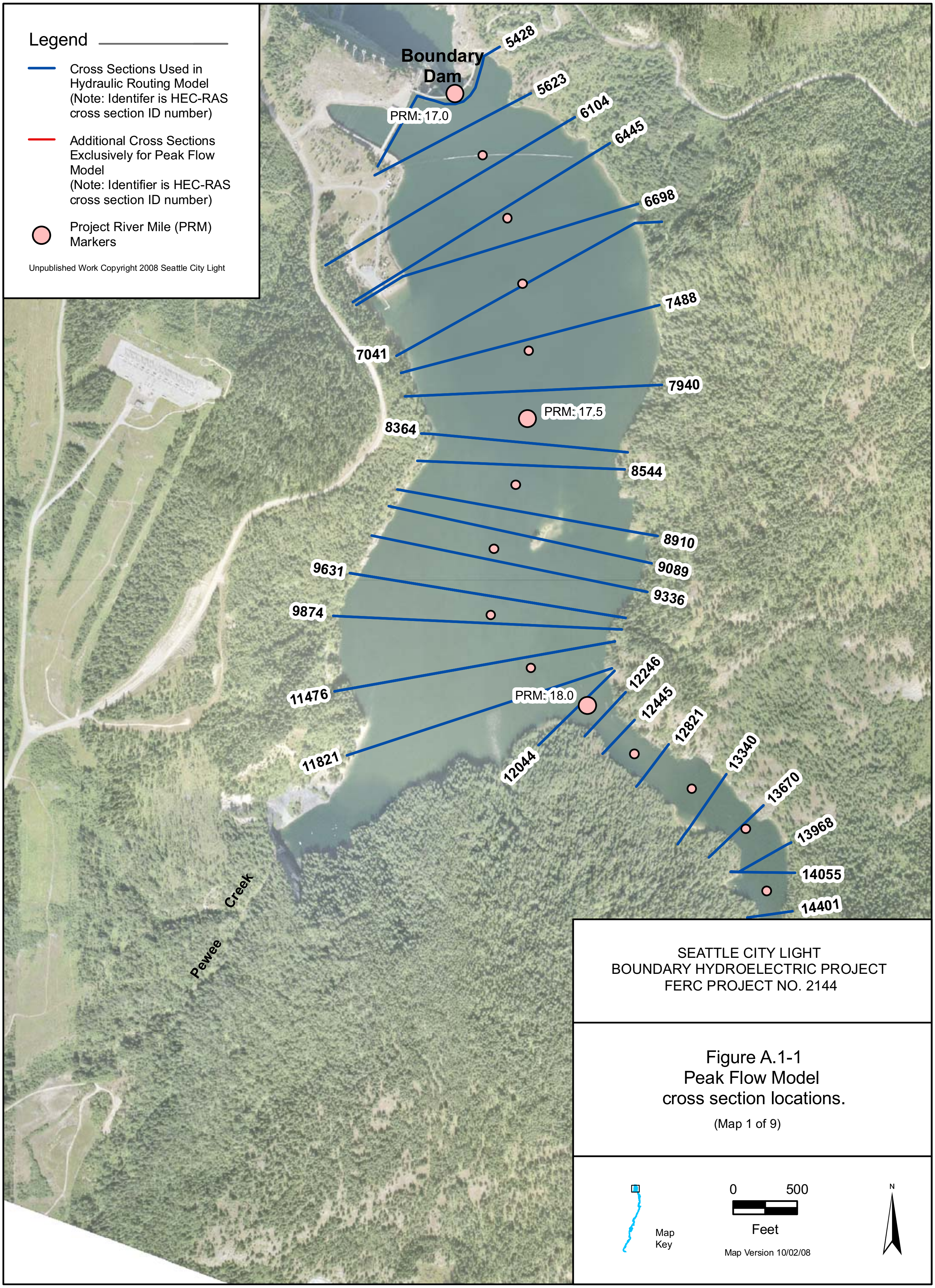
Appendix 1: Peak Flow Model Cross Section Locations



SEATTLE CITY LIGHT
BOUNDARY HYDROELECTRIC PROJECT
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Figure A.1-1
Peak Flow Model
cross section locations.

(Map 1 of 9)

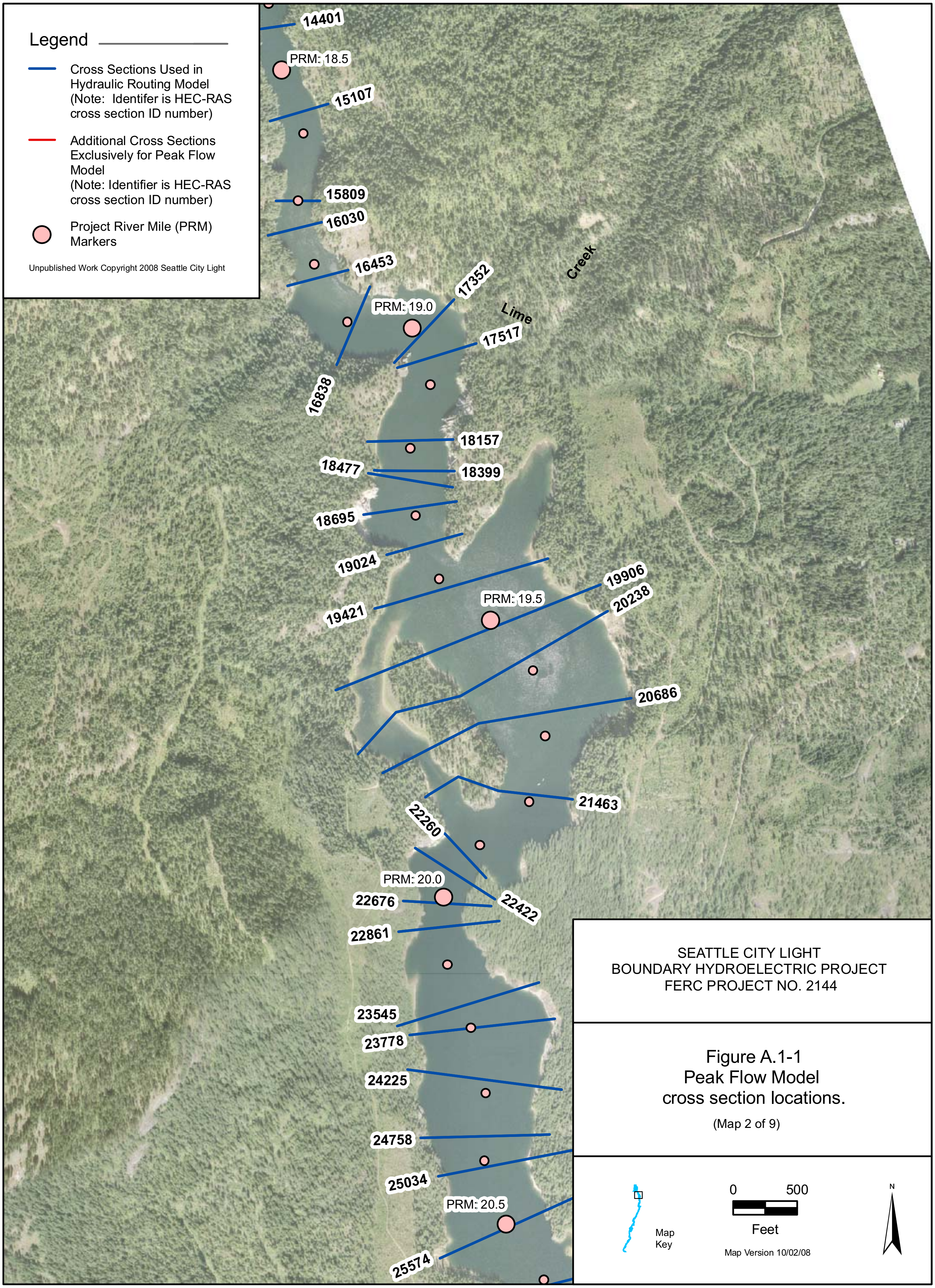

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Figure A.1-1
Peak Flow Model
cross section locations.
(Map 2 of 9)

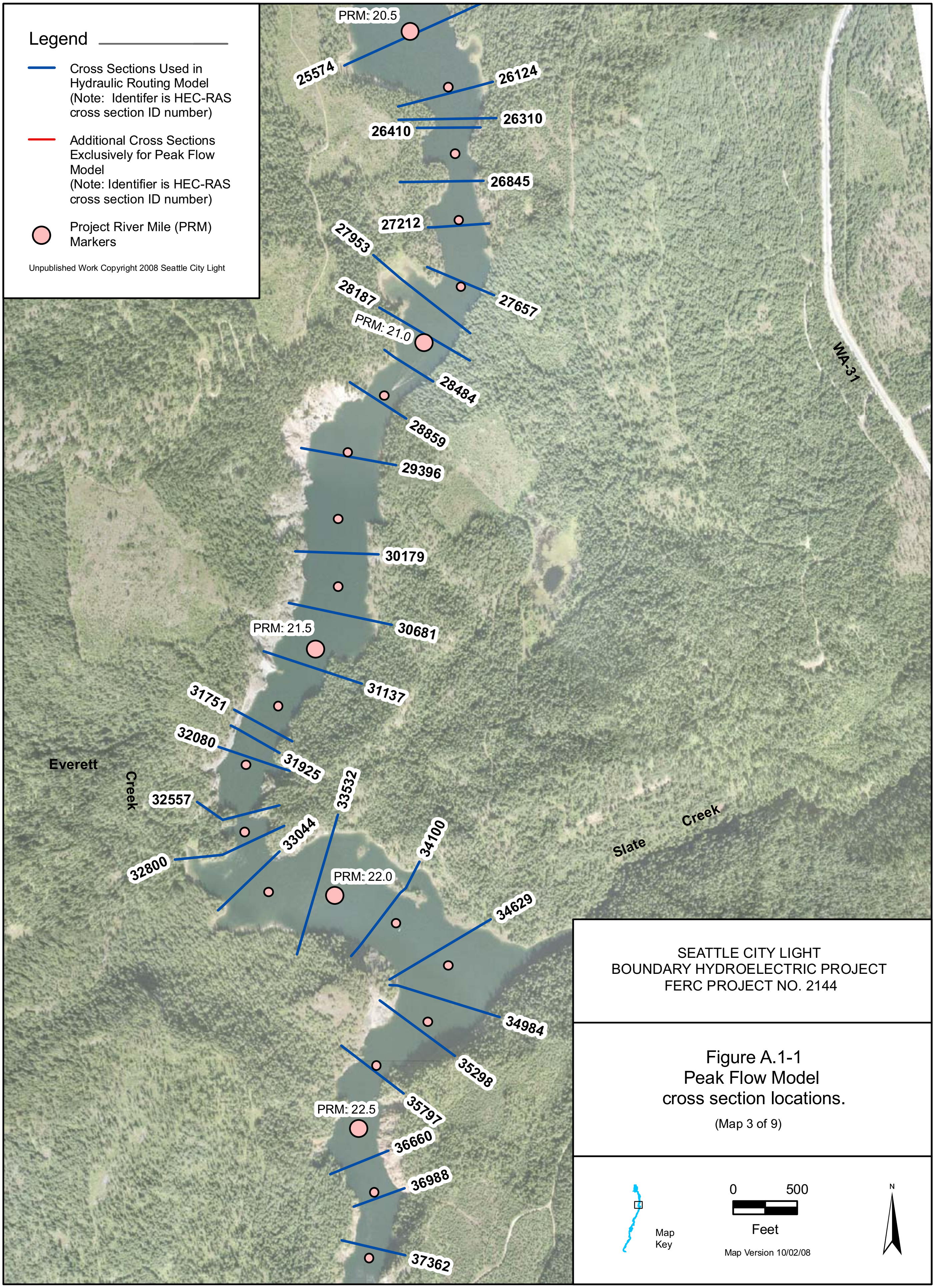

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Figure A.1-1
Peak Flow Model
cross section locations.
(Map 3 of 9)

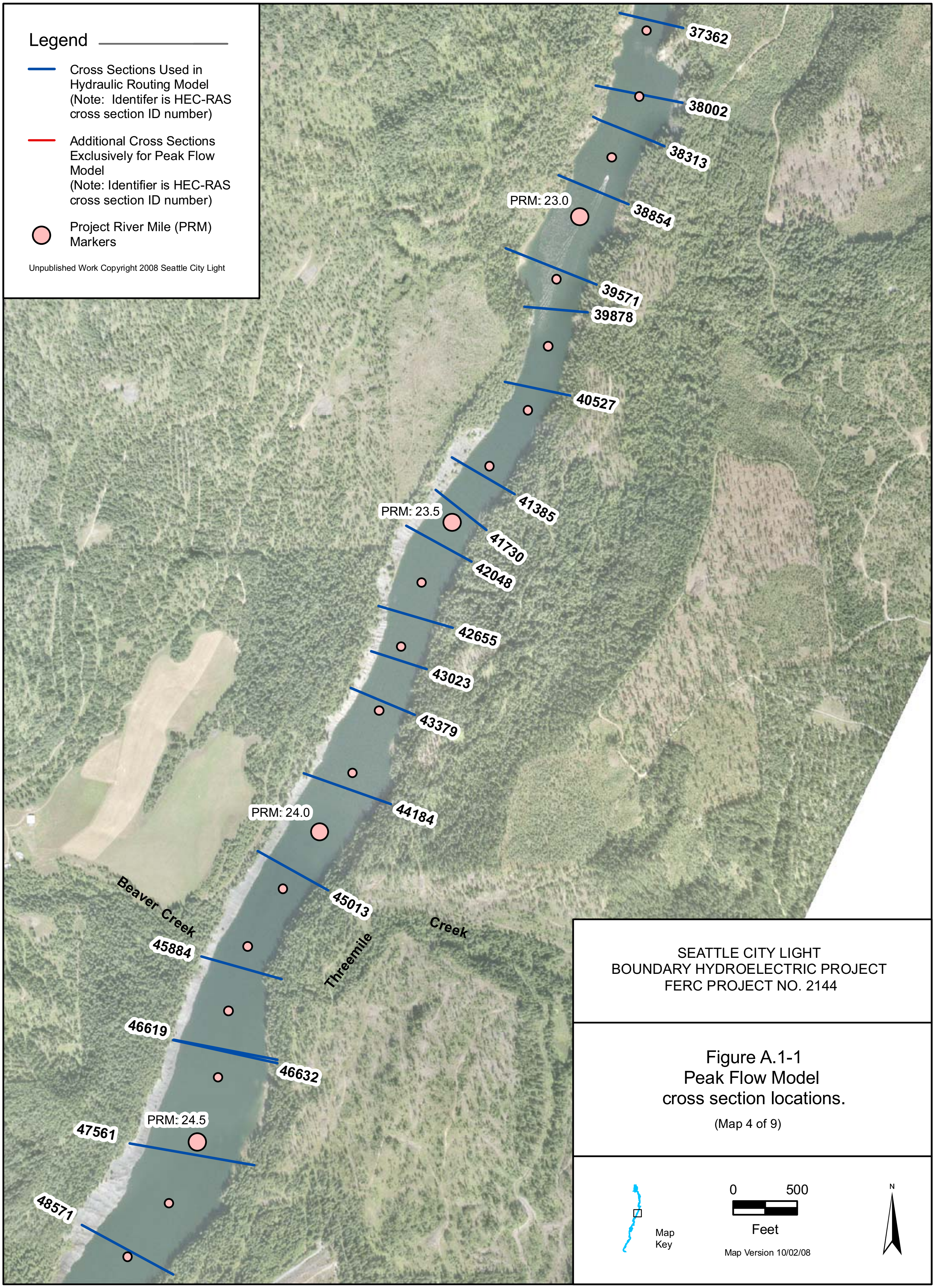

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Figure A.1-1
Peak Flow Model
cross section locations.
(Map 4 of 9)

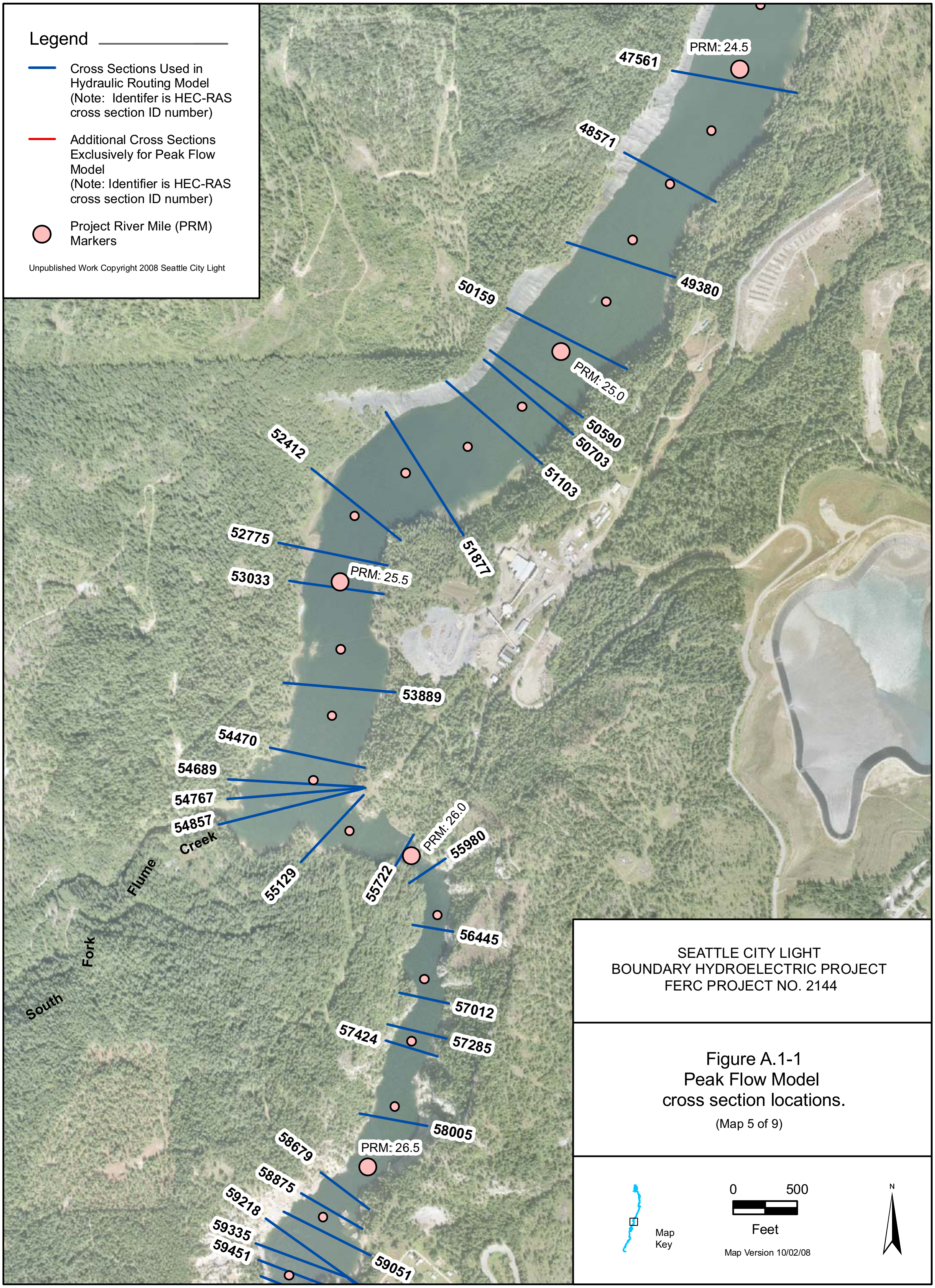

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Figure A.1-1
Peak Flow Model
cross section locations.
(Map 5 of 9)

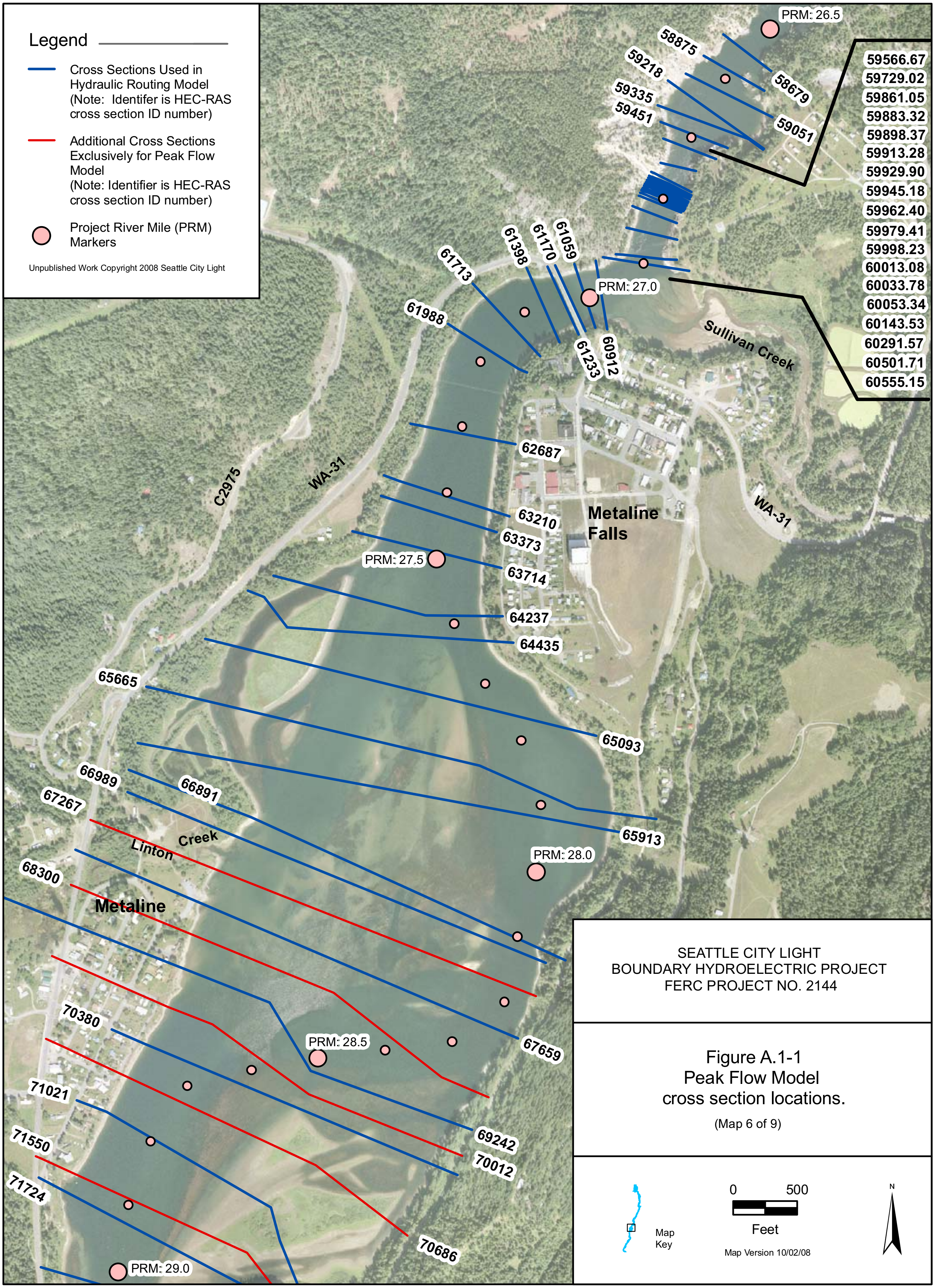


Figure A.1-1
Peak Flow Model
cross section locations.
(Map 6 of 9)

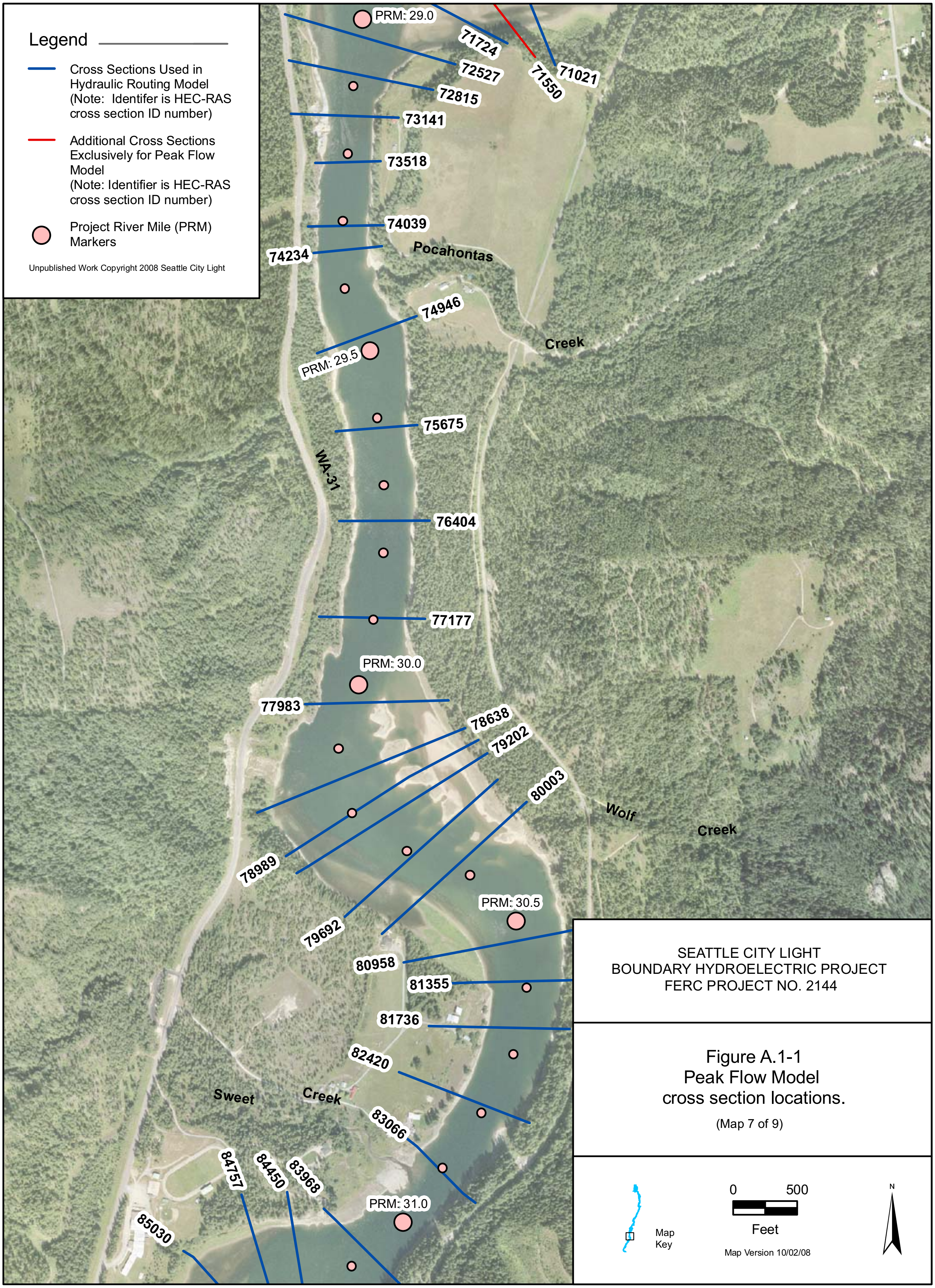
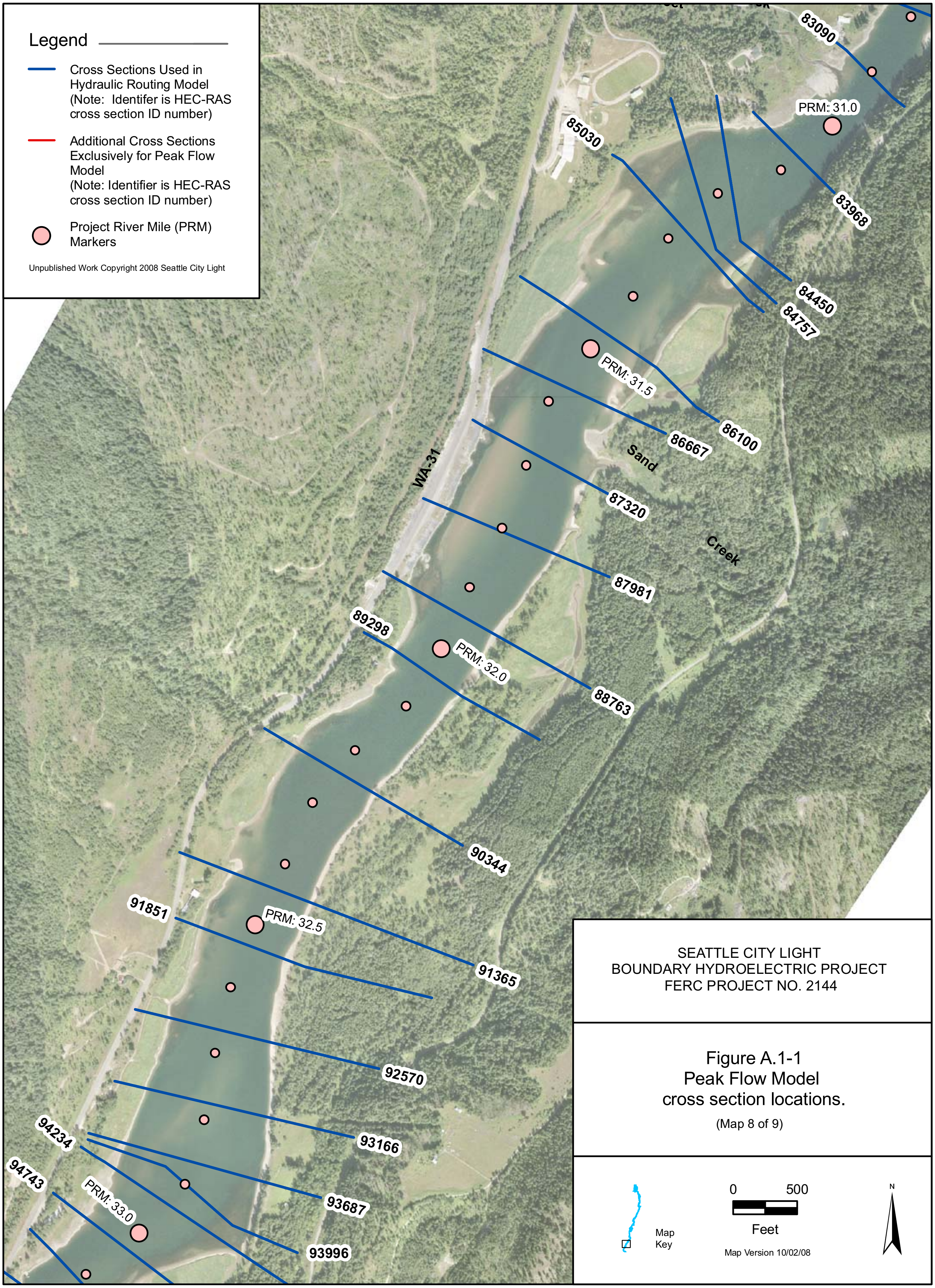


Figure A.1-1
Peak Flow Model
cross section locations.
(Map 7 of 9)



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BOUNDARY HYDROELECTRIC PROJECT
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Figure A.1-1
Peak Flow Model
cross section locations.
(Map 8 of 9)

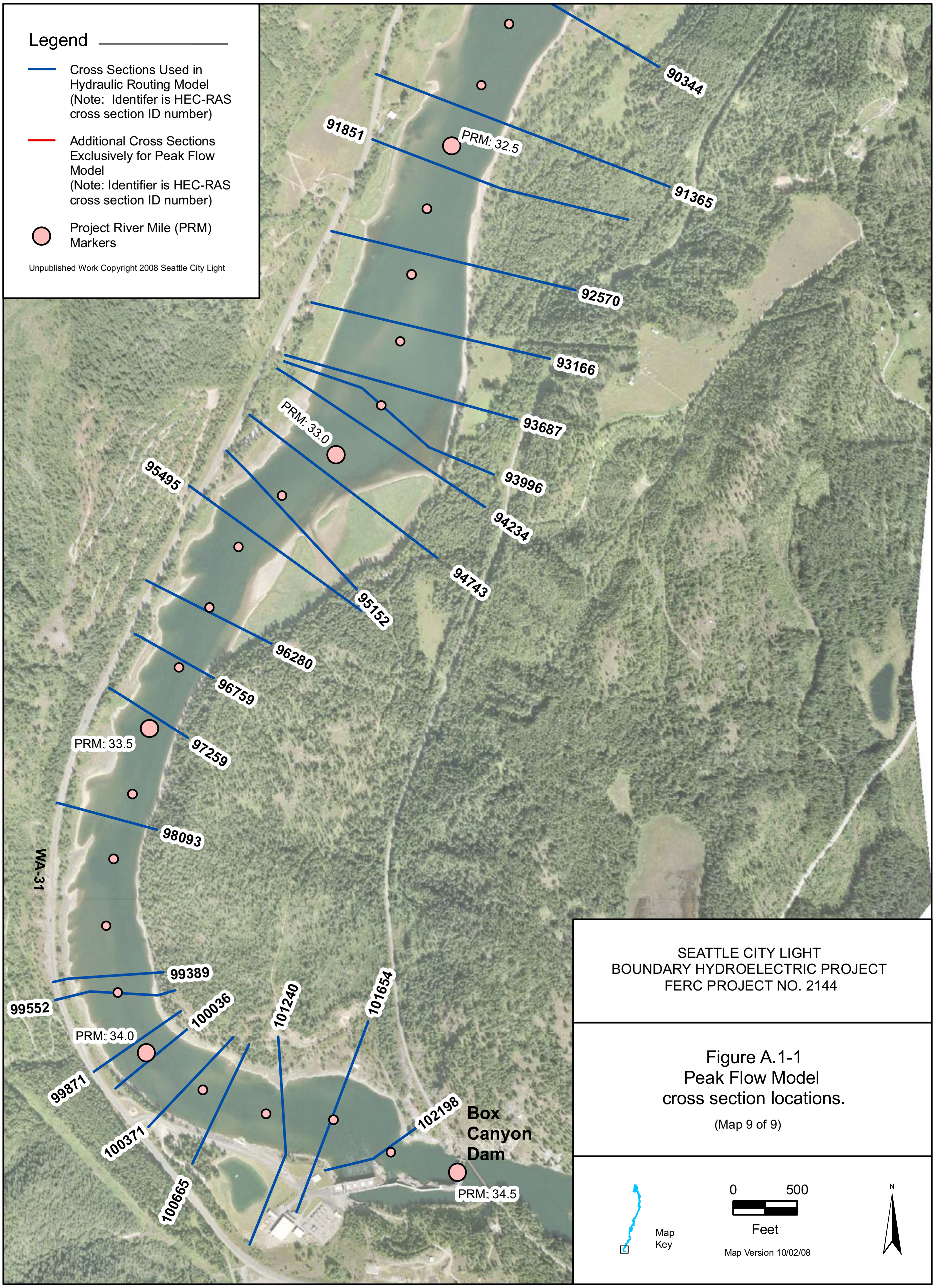

Figure A.1-1
Peak Flow Model
cross section locations.
(Map 9 of 9)

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Appendix 2: Stage and Discharge Boundary Conditions for the 1972, 1974, 1997, and 2008 Flood Event Periods to Calibrate Hydraulic Routing Model

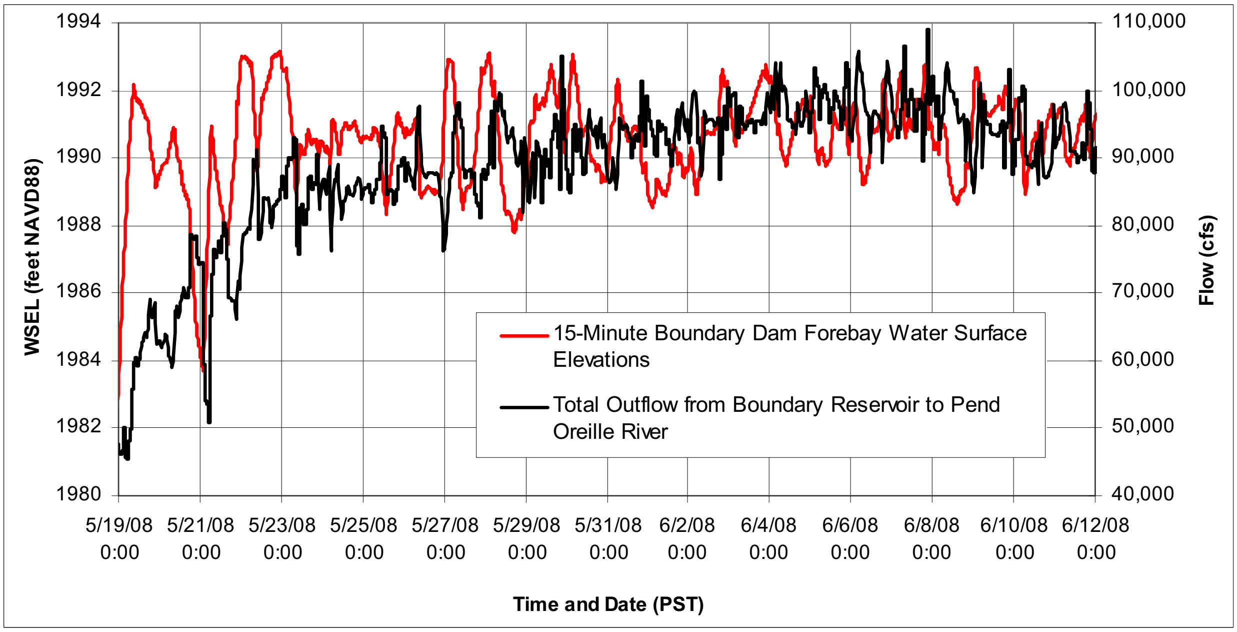


Figure A.2-1. Boundary forebay 15-minute stage and outflow hydrograph for the 2008 period of high water.

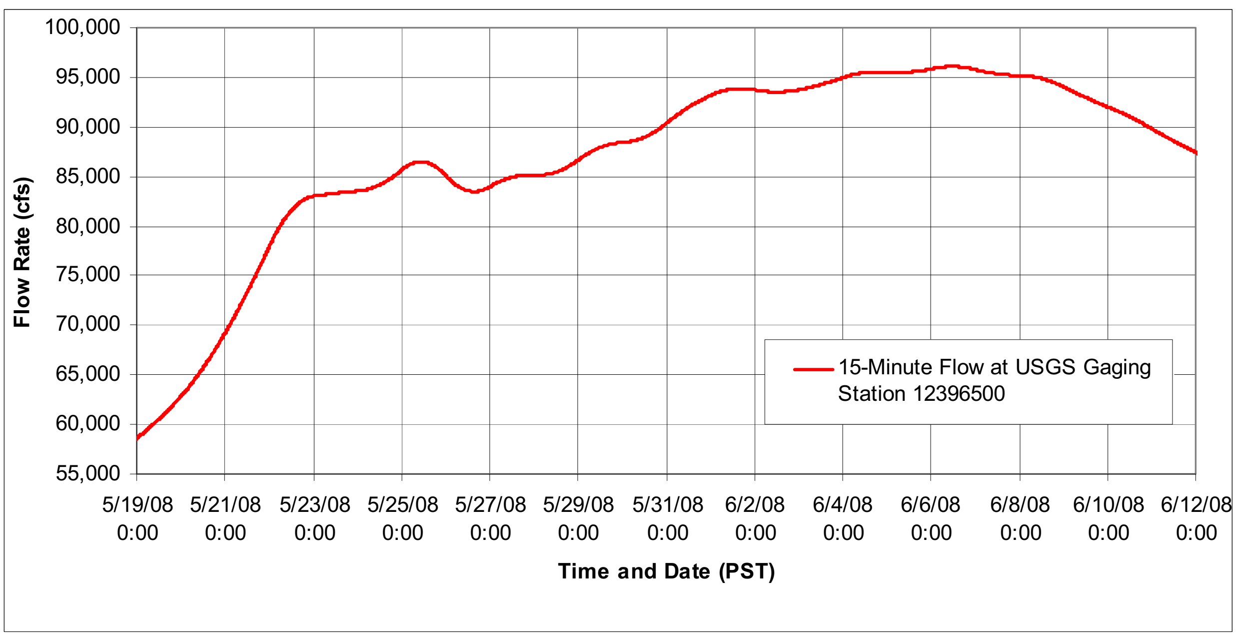


Figure A.2-2. USGS gage No. 12396500 15-minute flow hydrograph for the 2008 period of high water.

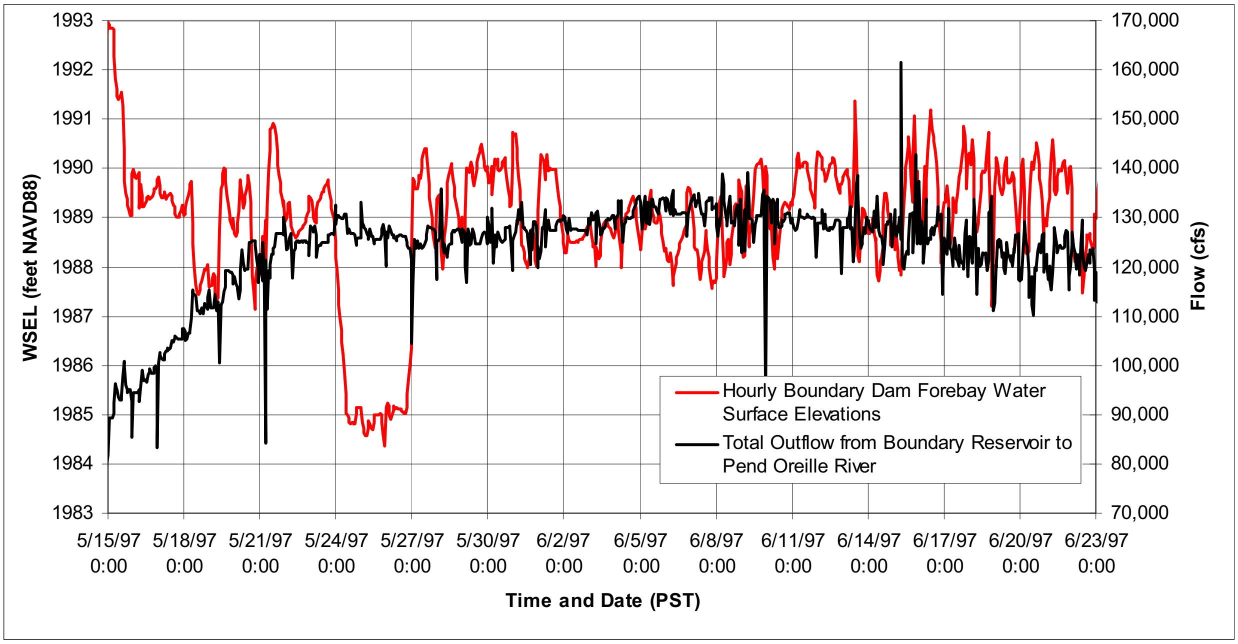


Figure A.2-3. Boundary forebay 15-minute stage and outflow hydrograph for the 1997 period of high water.

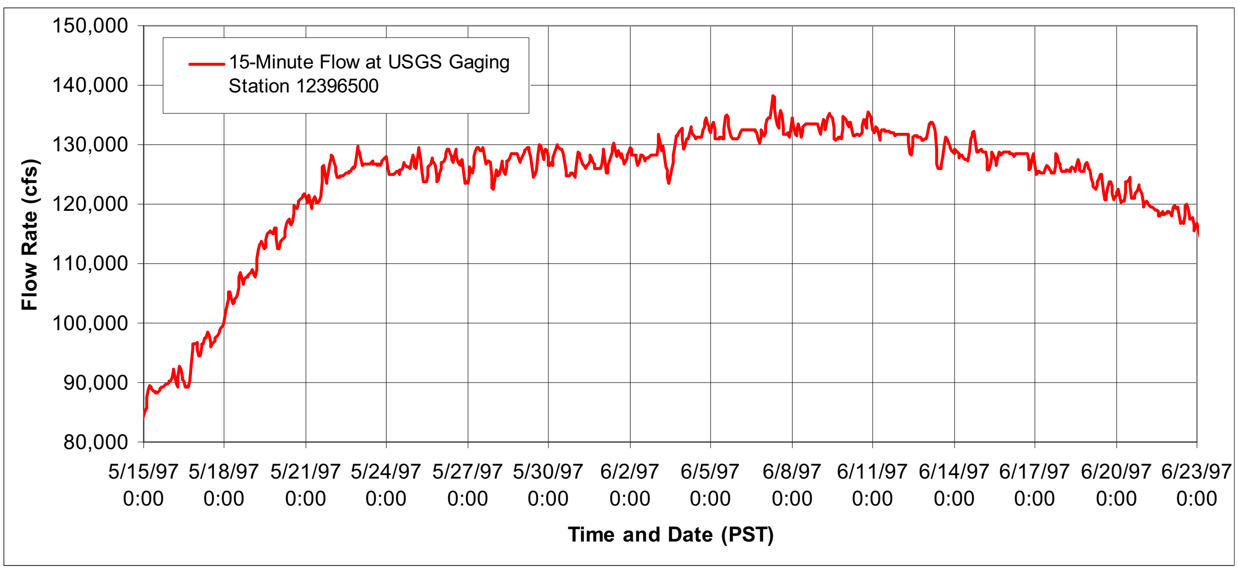


Figure A.2-4. USGS gage No. 12396500 15-minute flow hydrograph for the 1997 period of high water.

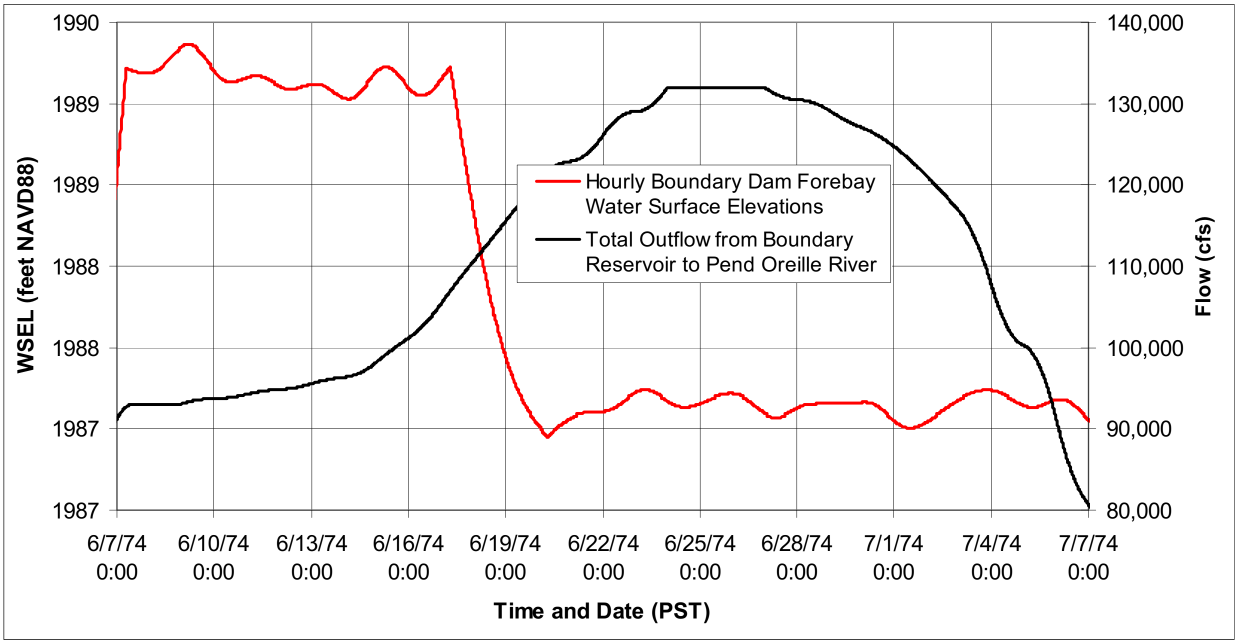


Figure A.2-5. Boundary forebay 15-minute stage and outflow hydrograph for the 1974 period of high water.

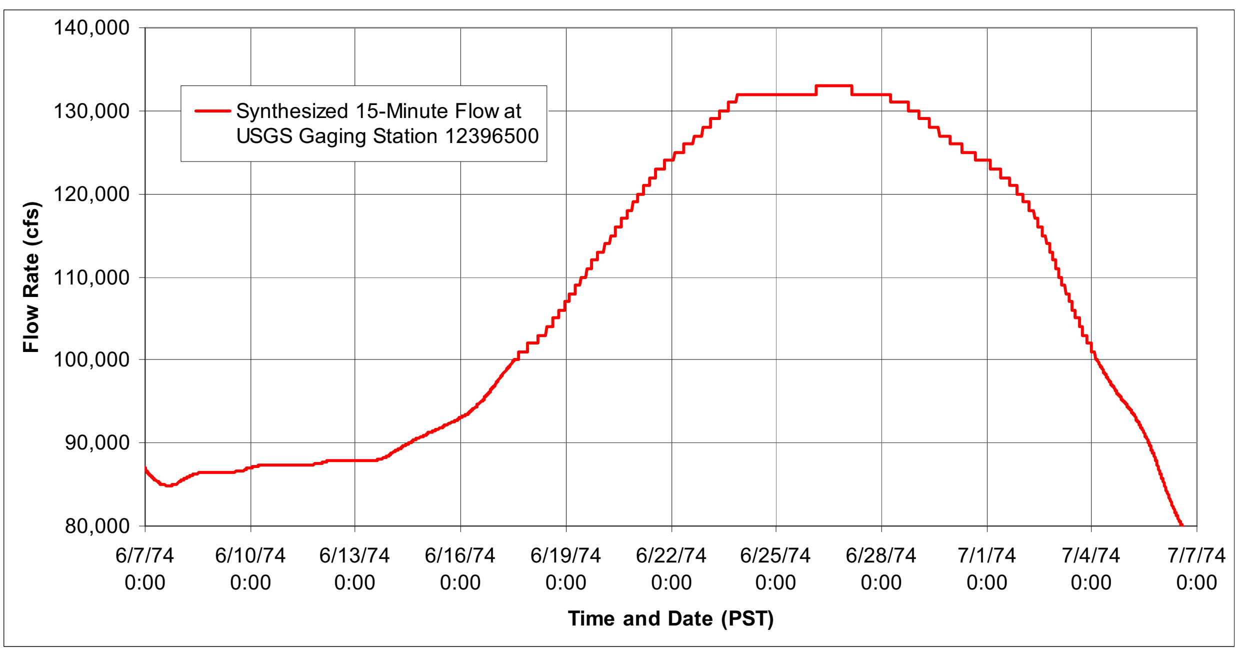


Figure A.2-6. USGS gage No. 12396500 15-minute flow hydrograph for the 1974 period of high water.

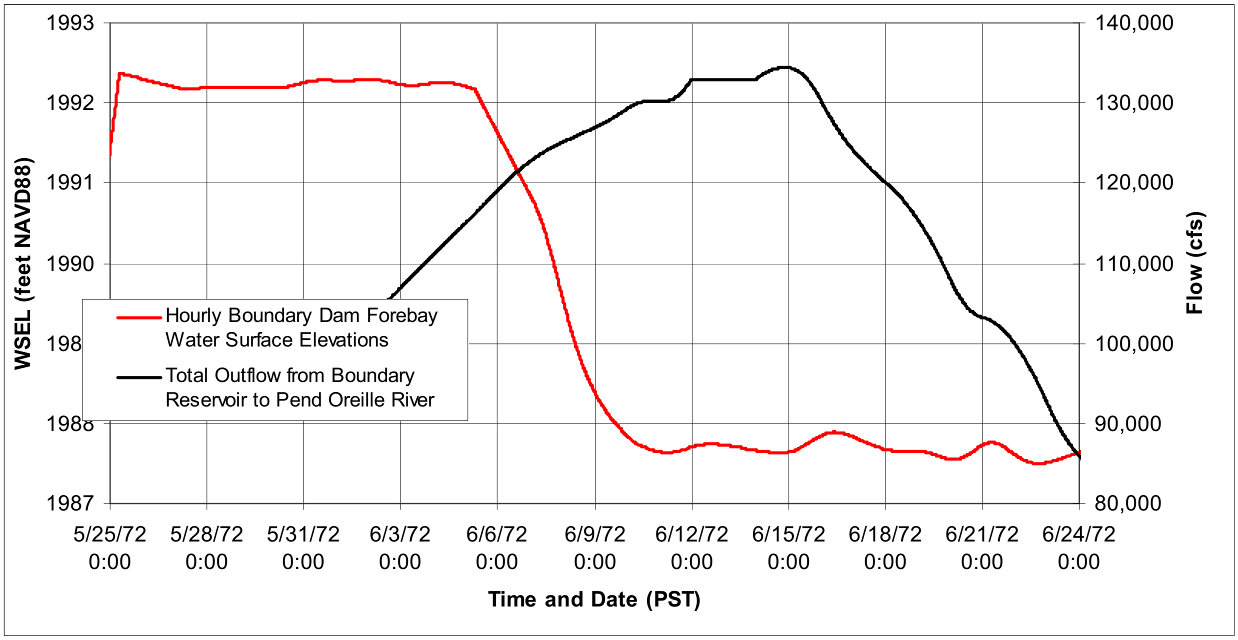


Figure A.2-7. Boundary forebay 15-minute stage and outflow hydrograph for the 1972 period of high water.

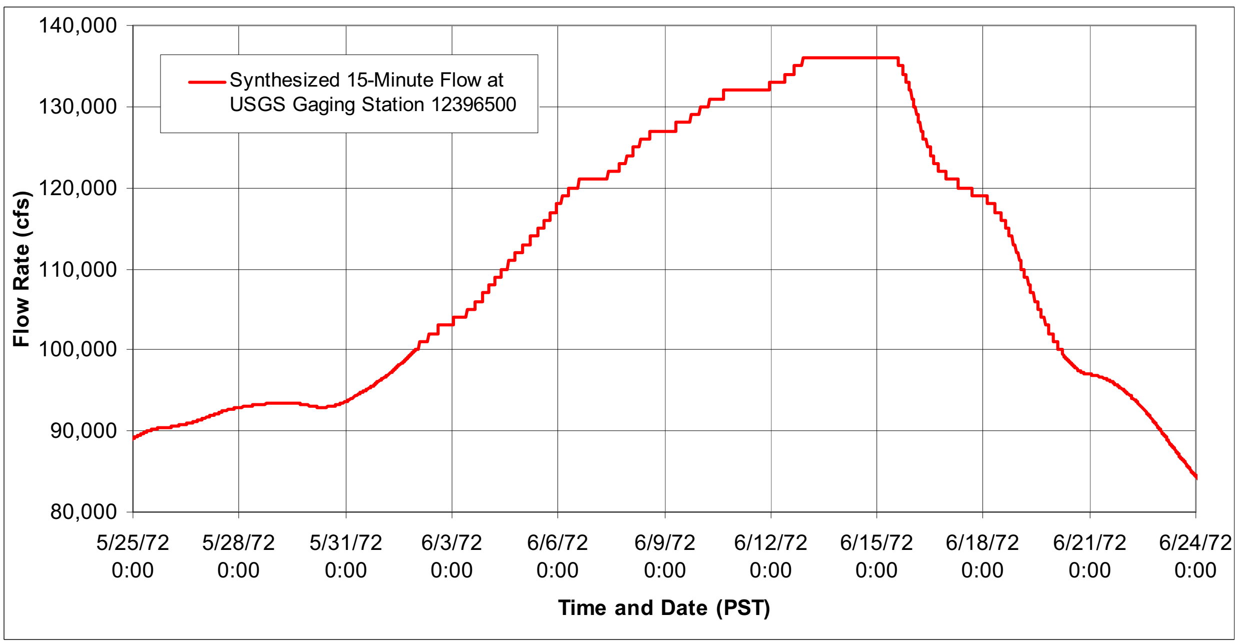


Figure A.2-8. USGS gage No. 12396500 15-minute flow hydrograph for the 1972 period of high water.

Appendix 3: Comparison of Model-Simulated Water Surface Elevations with Observed Records during the 1972, 1974, 1997, and 2008 Flood Event Periods

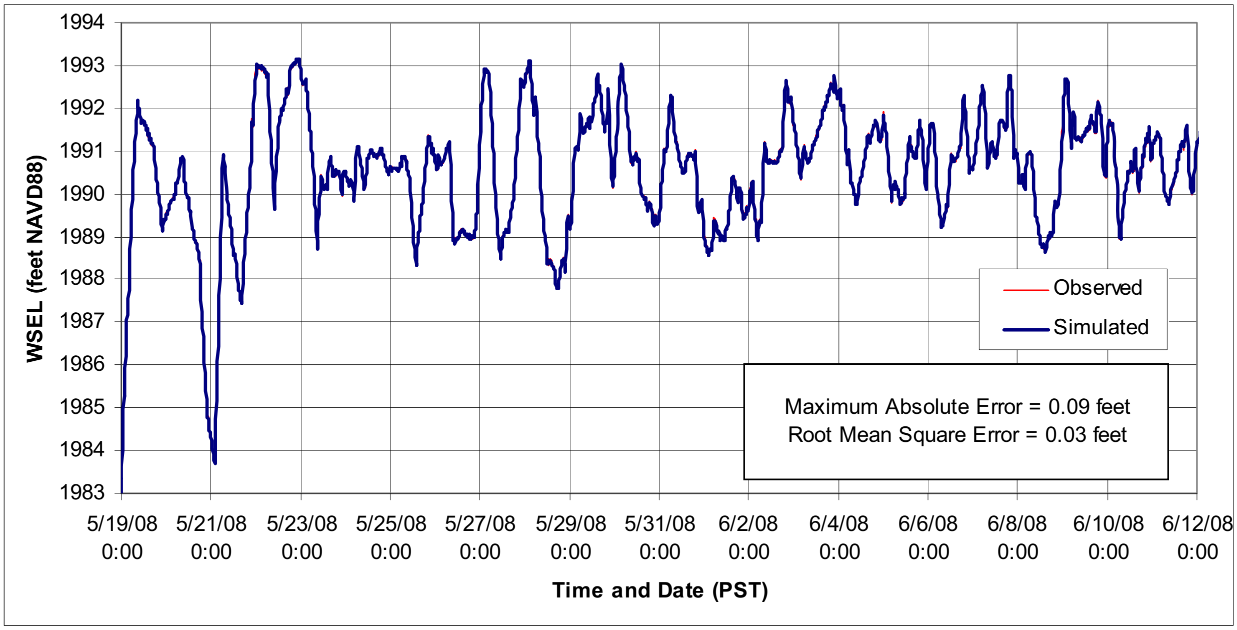


Figure A.3-1. Model calibration results for CANYON pressure transducer during the 2008 period of high water.

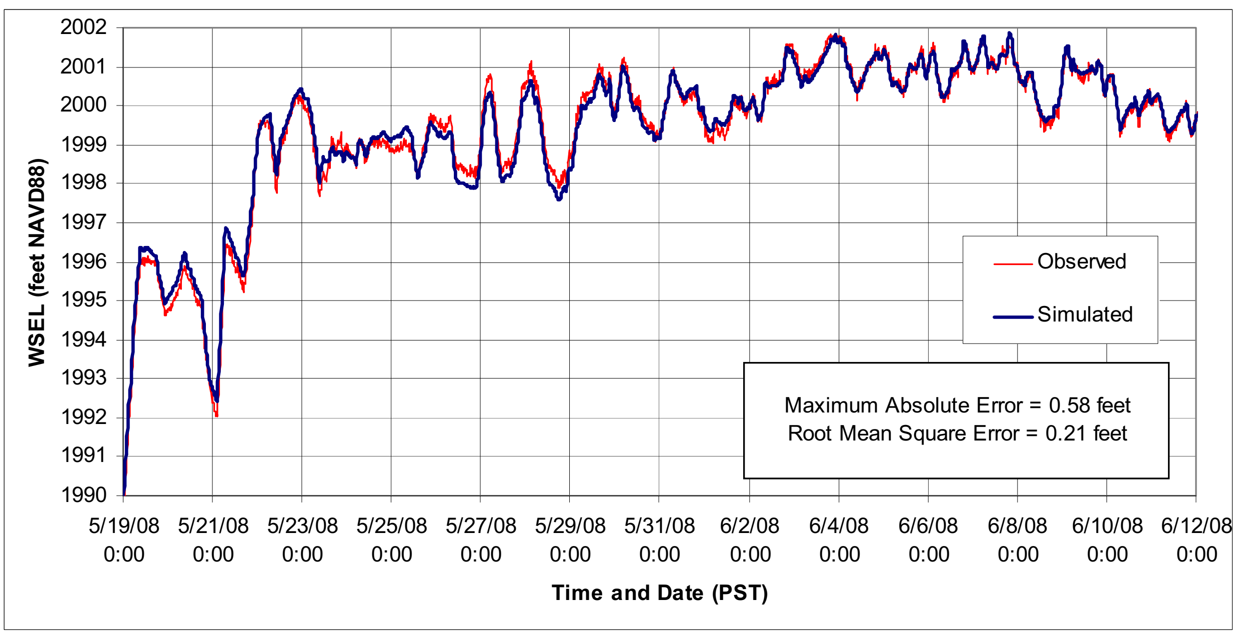


Figure A.3-2. Model calibration results for DS_MET pressure transducer during the 2008 period of high water.

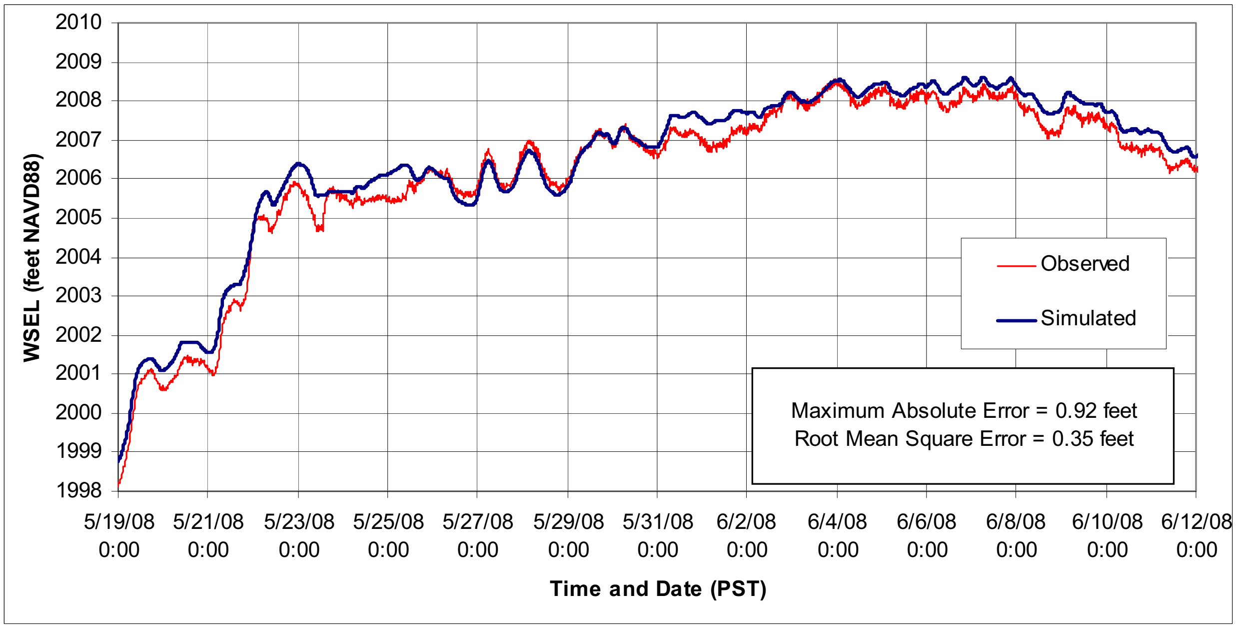


Figure A.3-3. Model calibration results for US_MET pressure transducer during the 2008 period of high water.

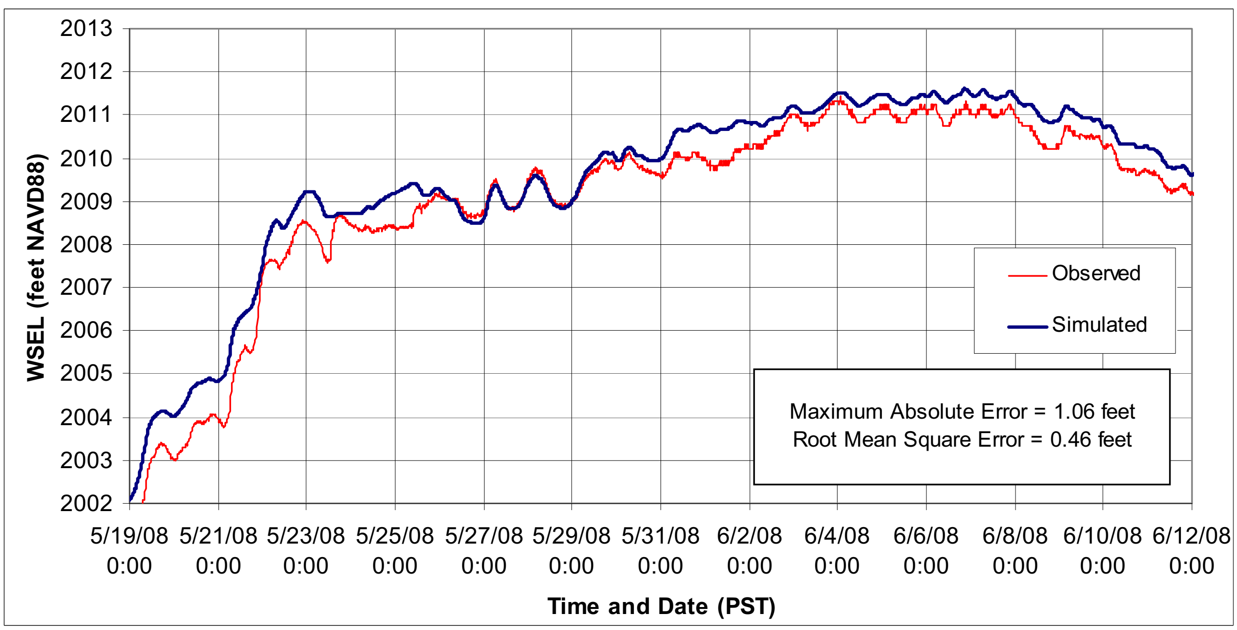


Figure A.3-4. Model calibration results for USGS gage 12396500 (Auxiliary) during the 2008 period of high water.

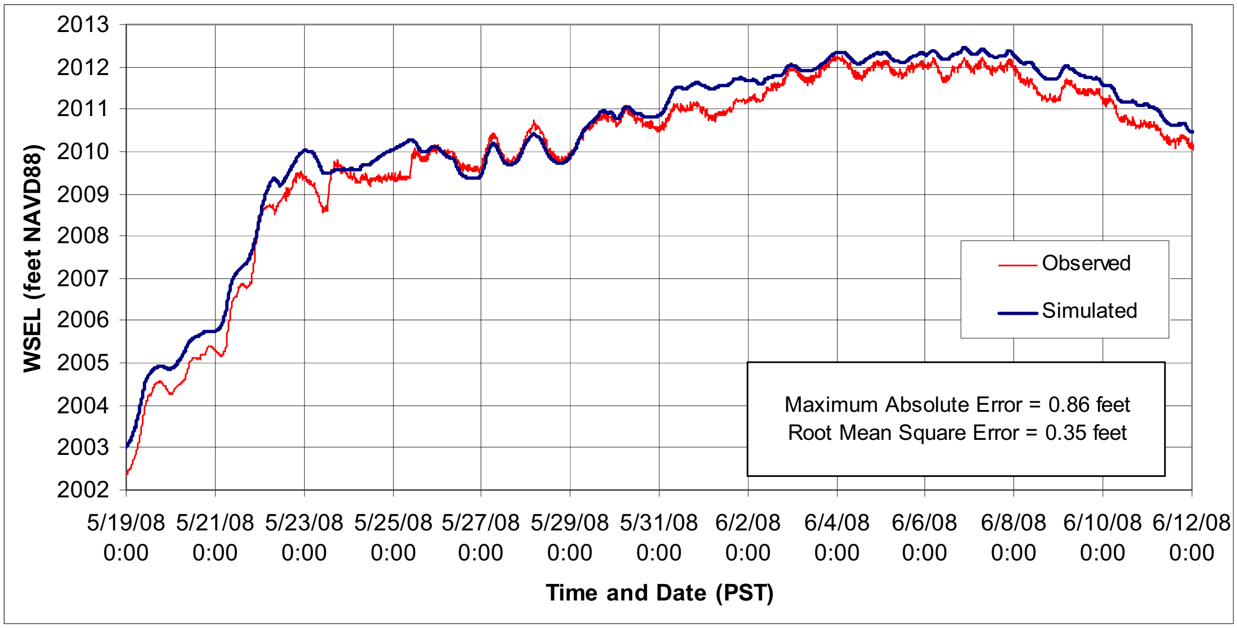


Figure A.3-5. Model calibration results for USGS gage 12396500 (Primary) during the 2008 period of high water.

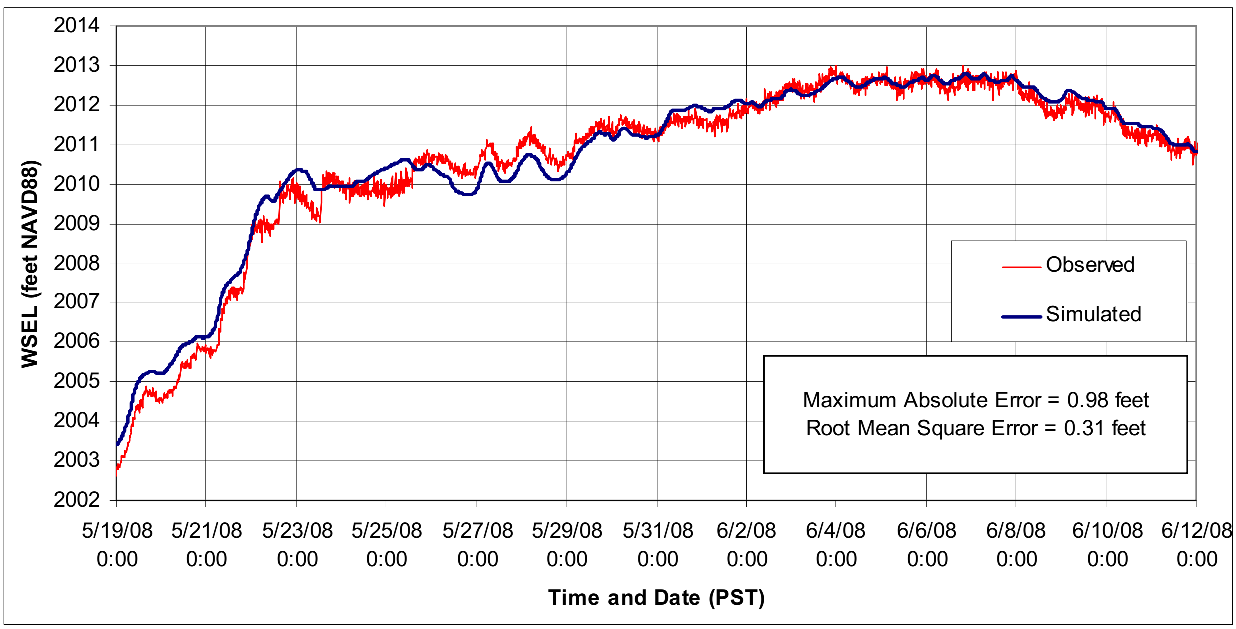


Figure A.3-6. Model calibration results for BOX_TR pressure transducer during the 2008 period of high water.

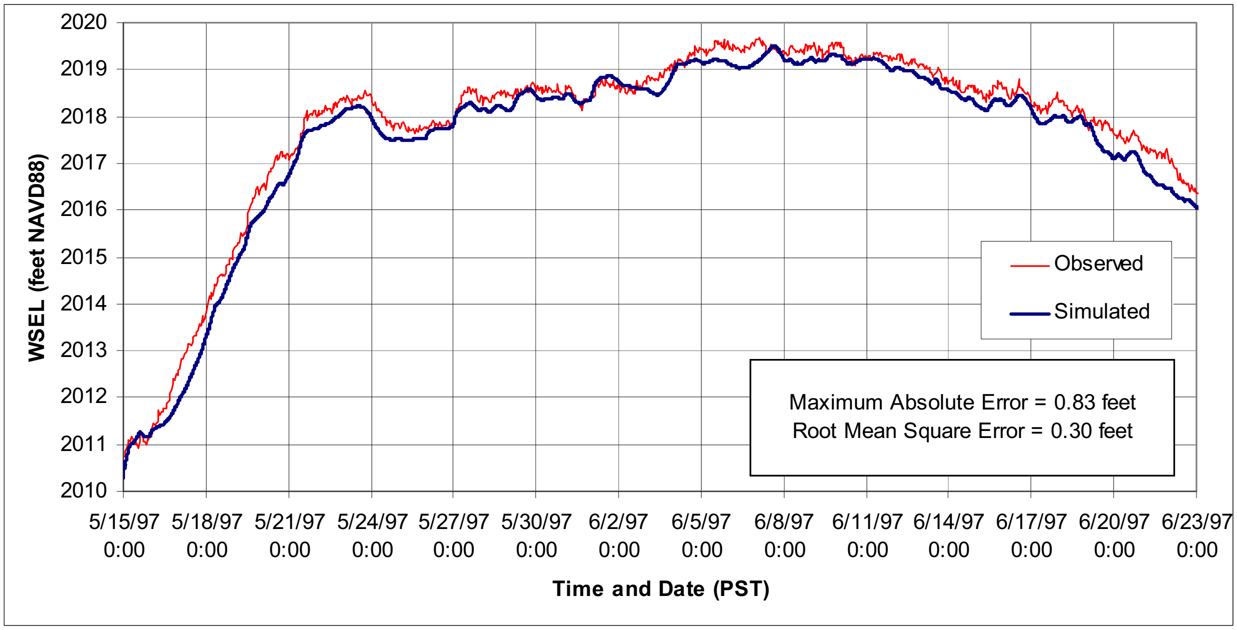


Figure A.3-7. Model calibration results for USGS gage 12396500 (Auxiliary) during the 1997 period of high water.

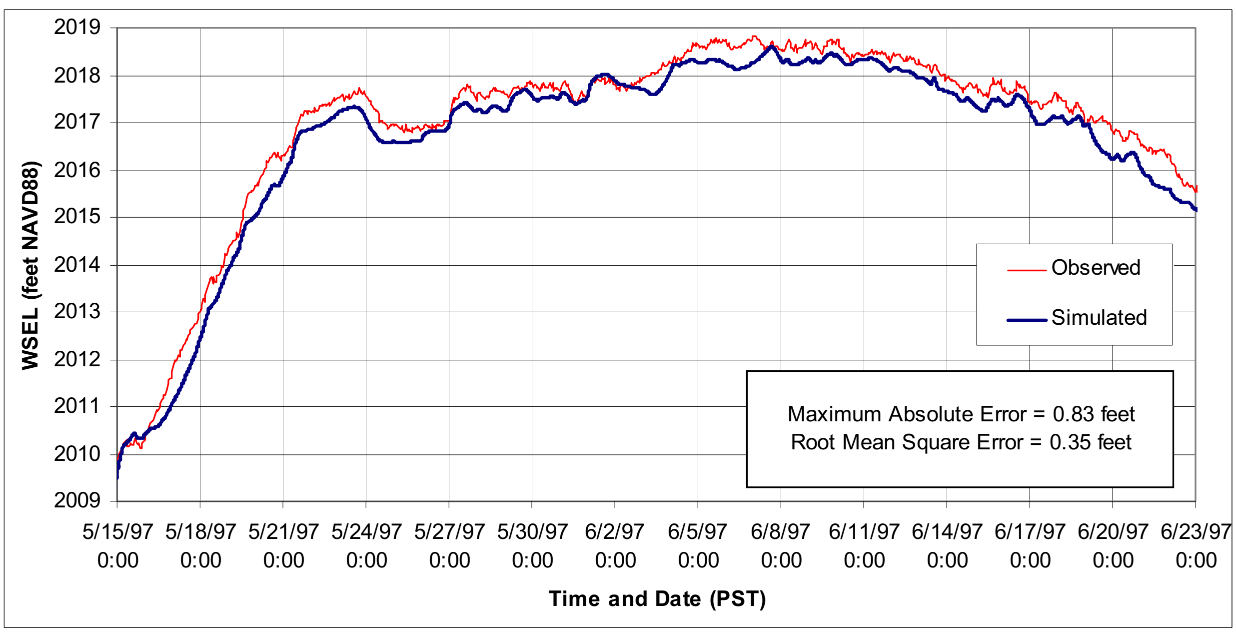


Figure A.3-8. Model calibration results for USGS gage 12396500 (Primary) during the 1997 period of high water.

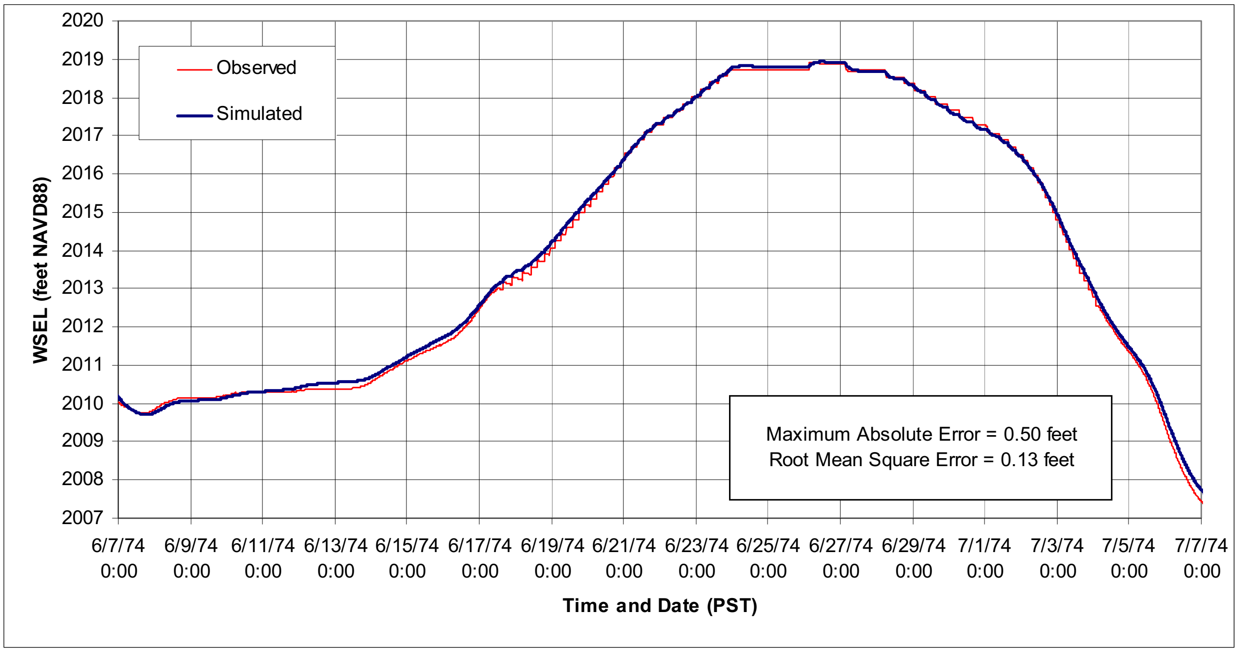


Figure A.3-9. Model calibration results for USGS gage 12396500 (Auxiliary) during the 1974 period of high water.

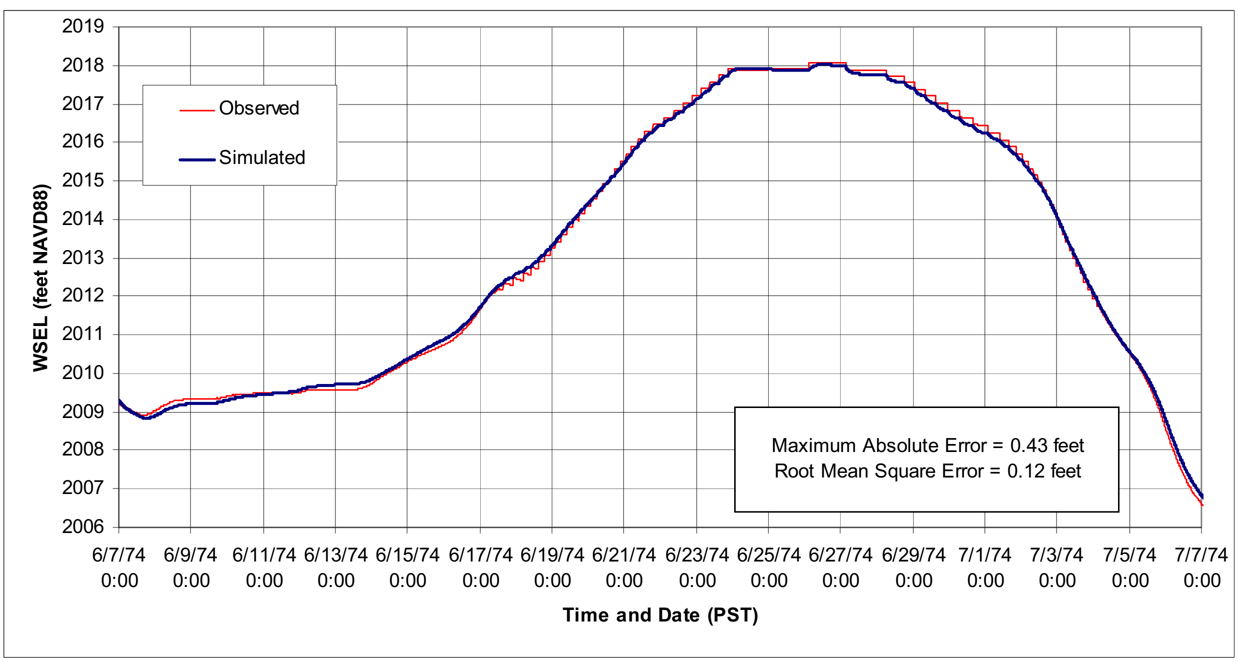


Figure A.3-10. Model calibration results for USGS gage 12396500 (Primary) during the 1974 period of high water.

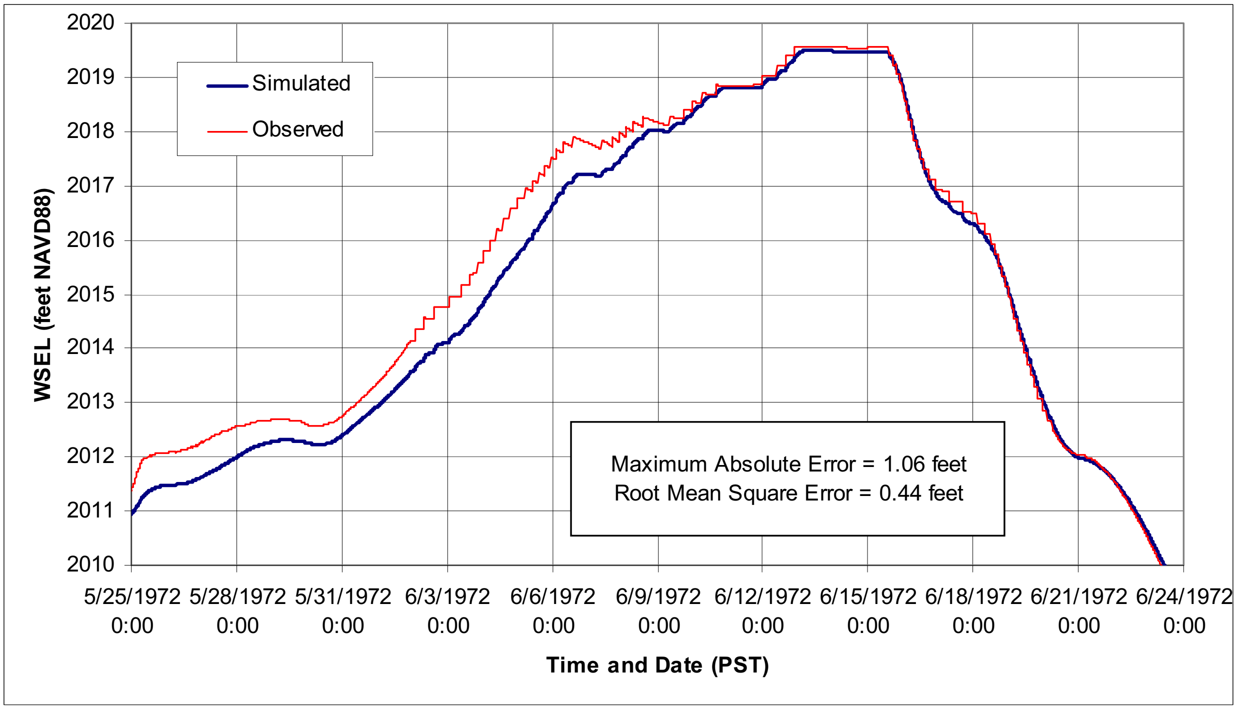


Figure A.3-11. Model calibration results for USGS gage 12396500 (Auxiliary) during the 1972 period of high water.

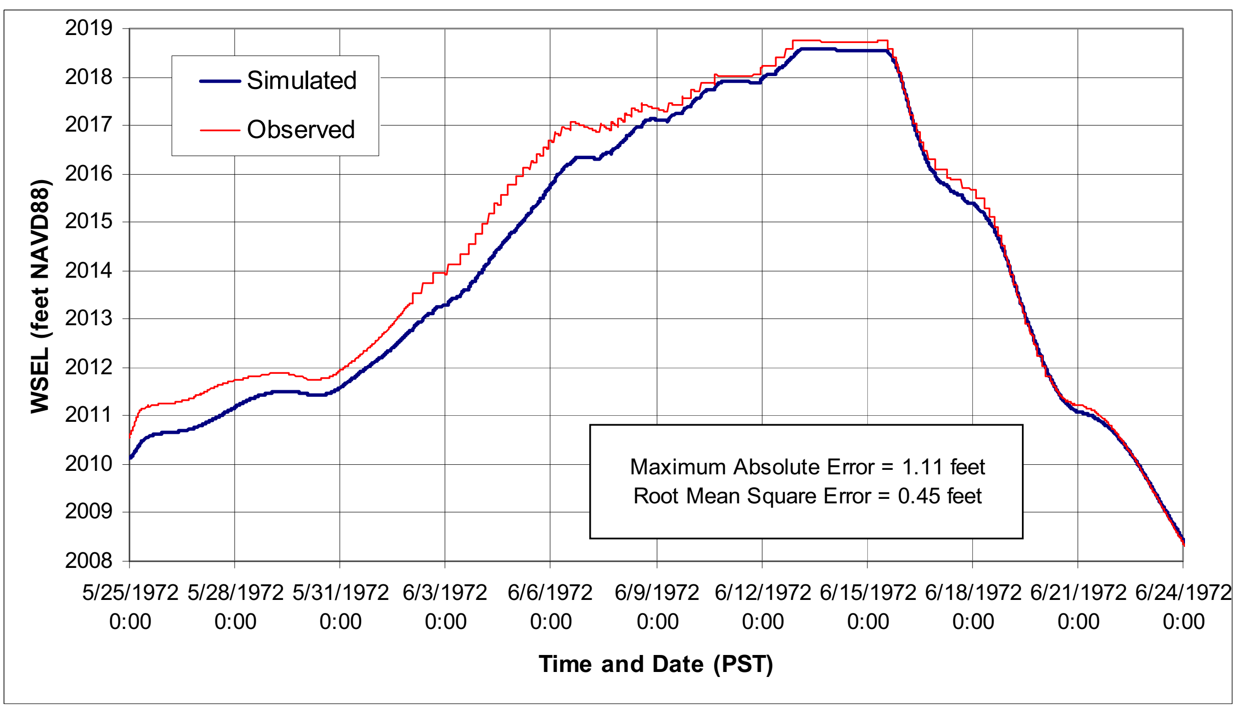
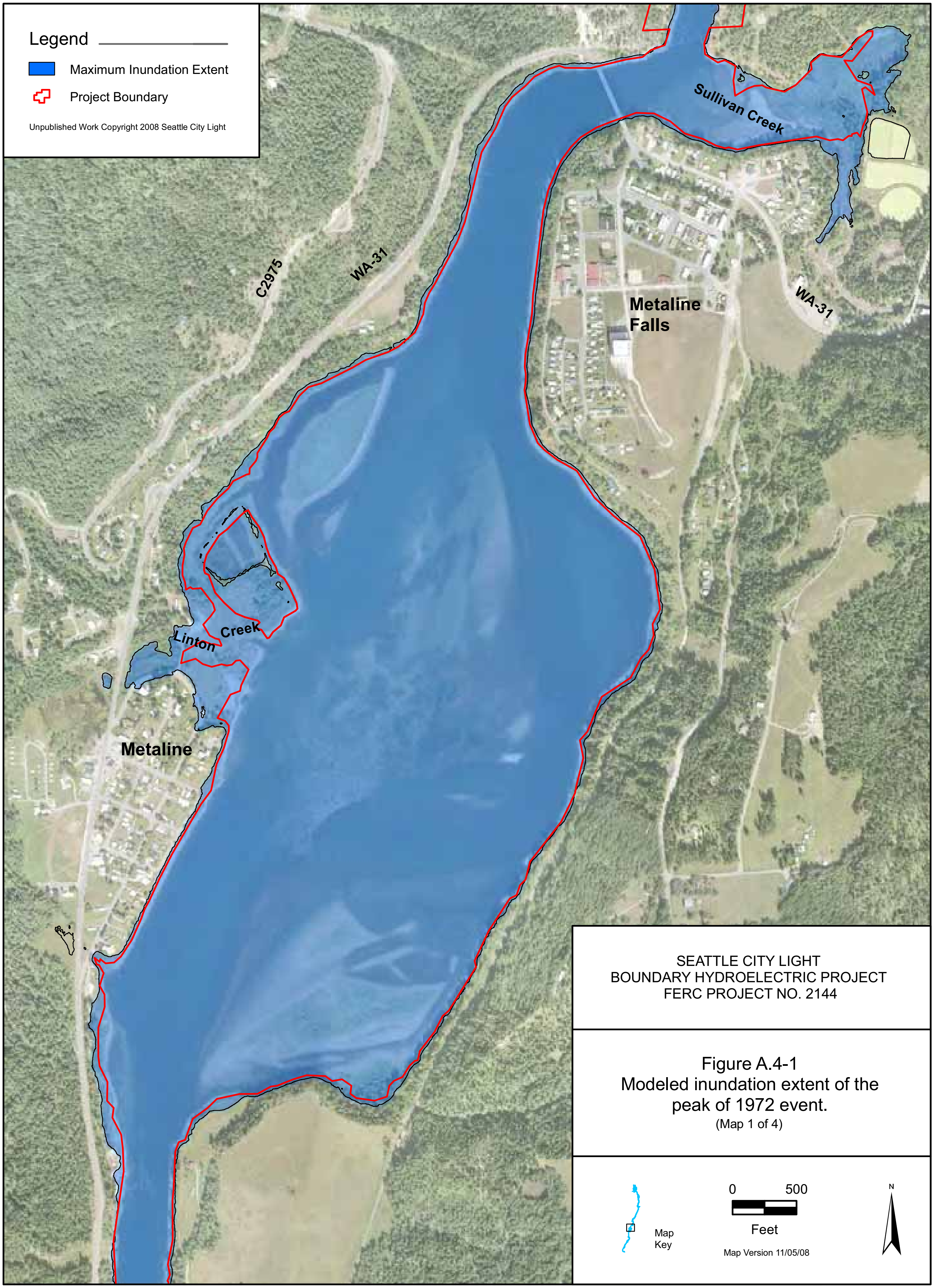


Figure A.3-12. Model calibration results for USGS gage 12396500 (Primary) during the 1972 period of high water.

Appendix 4: Model-Simulated Inundation Extents during the 1972, 1974, and 1997 Flood Event Periods



SEATTLE CITY LIGHT
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Figure A.4-1
Modeled inundation extent of the peak of 1972 event.
(Map 1 of 4)

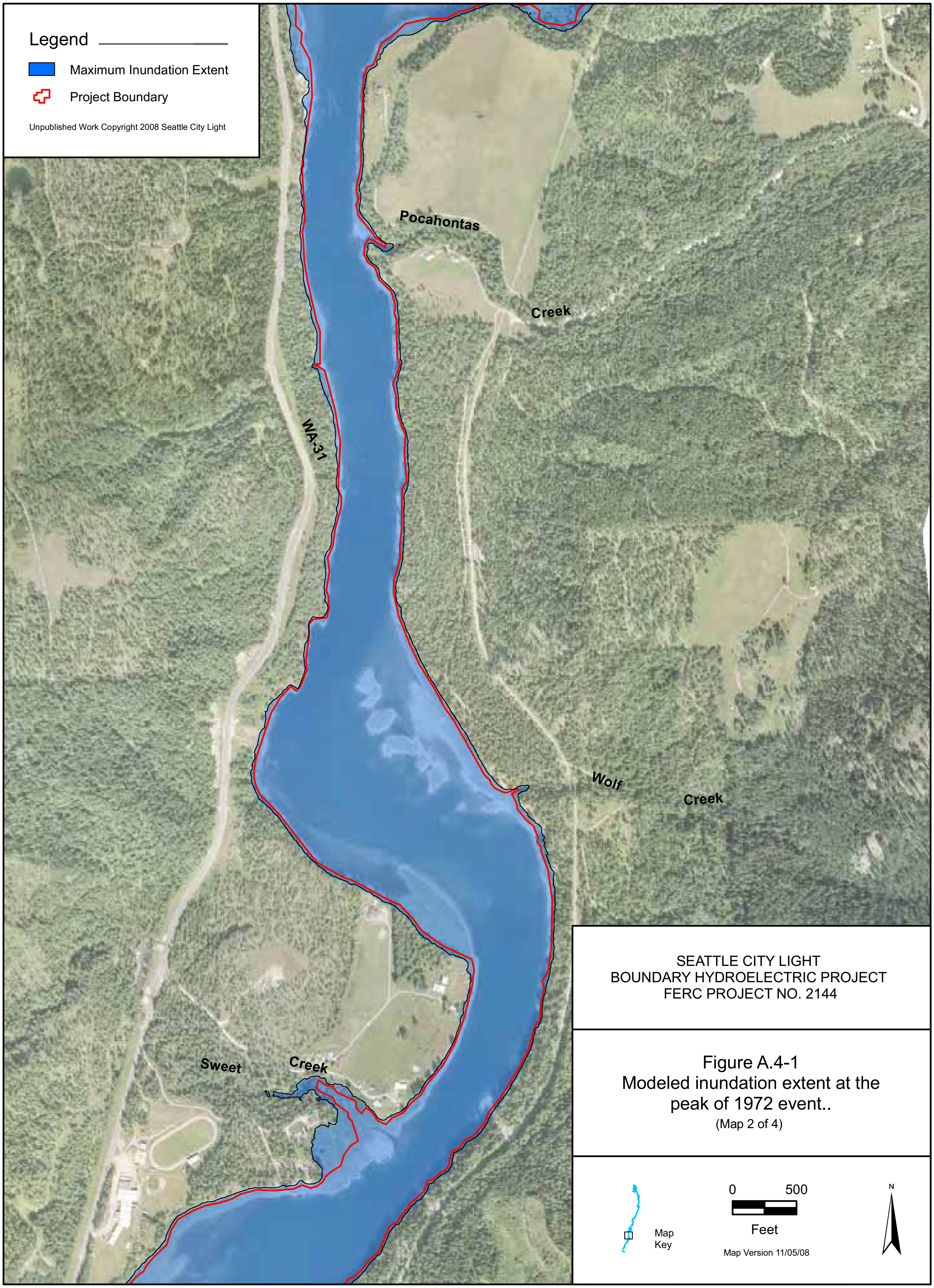

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Figure A.4-1
Modeled inundation extent at the peak of 1972 event..
(Map 2 of 4)

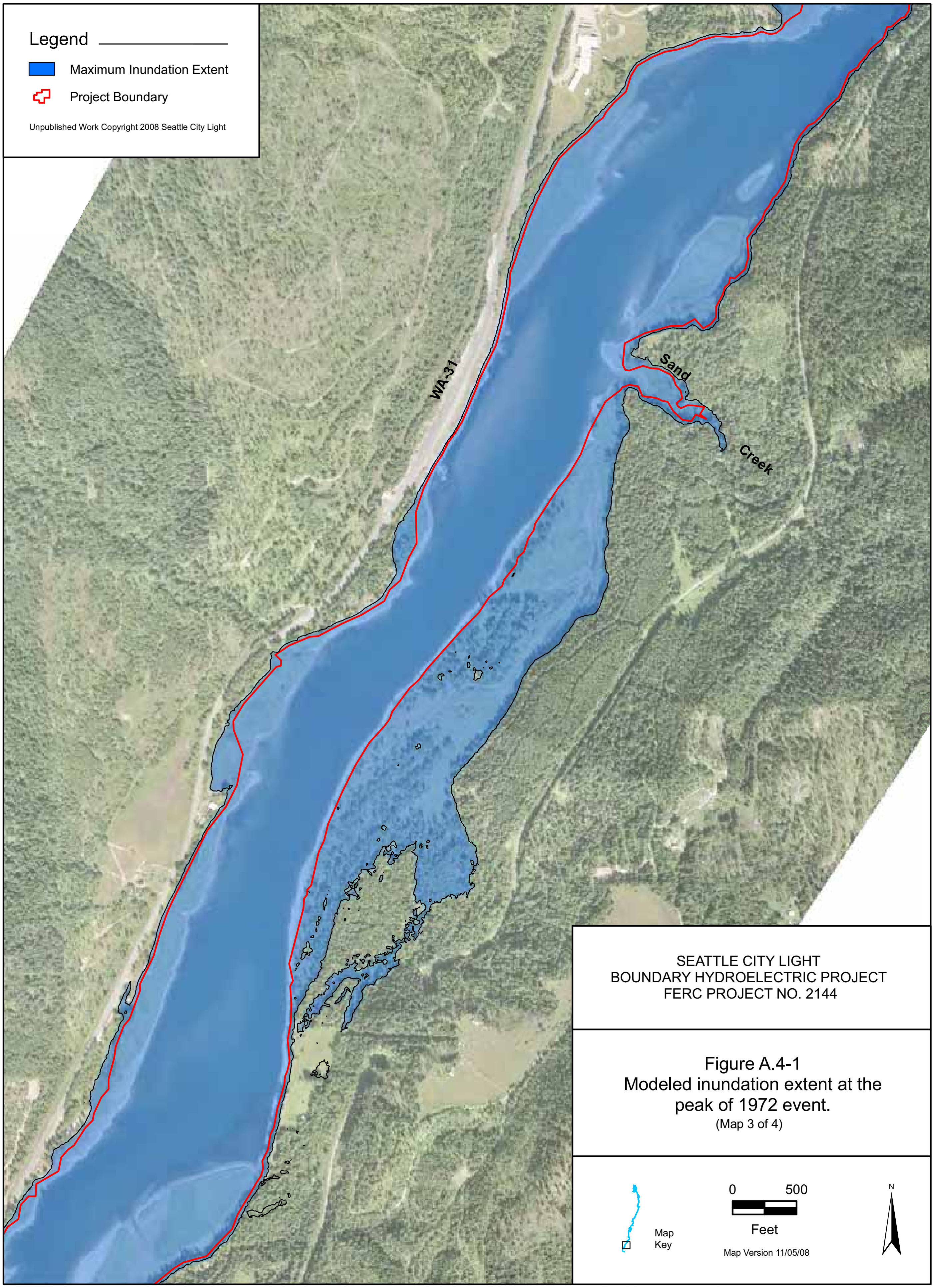

Figure A.4-1
Modeled inundation extent at the peak of 1972 event.
(Map 3 of 4)

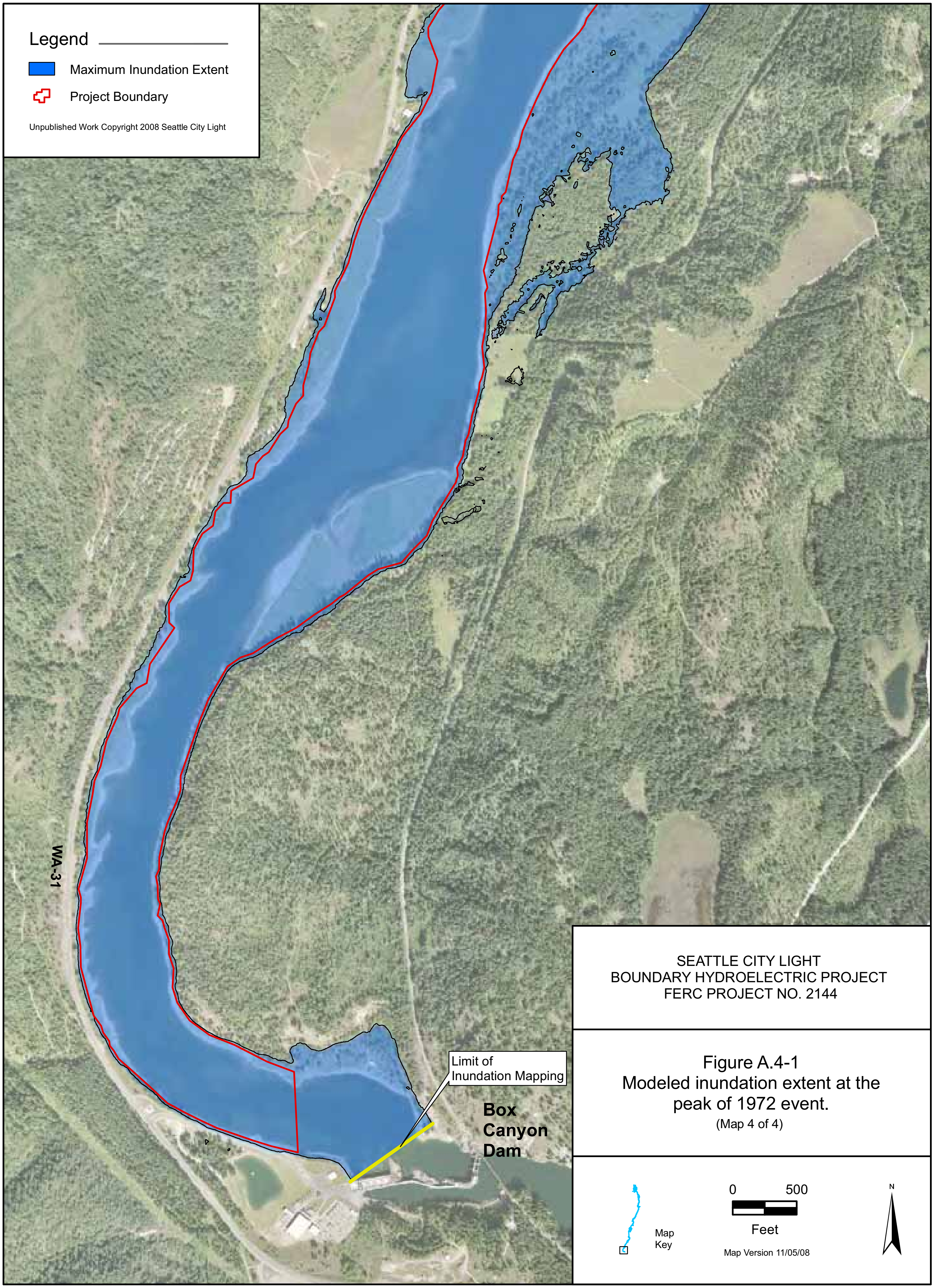

Figure A.4-1
Modeled inundation extent at the peak of 1972 event.
(Map 4 of 4)

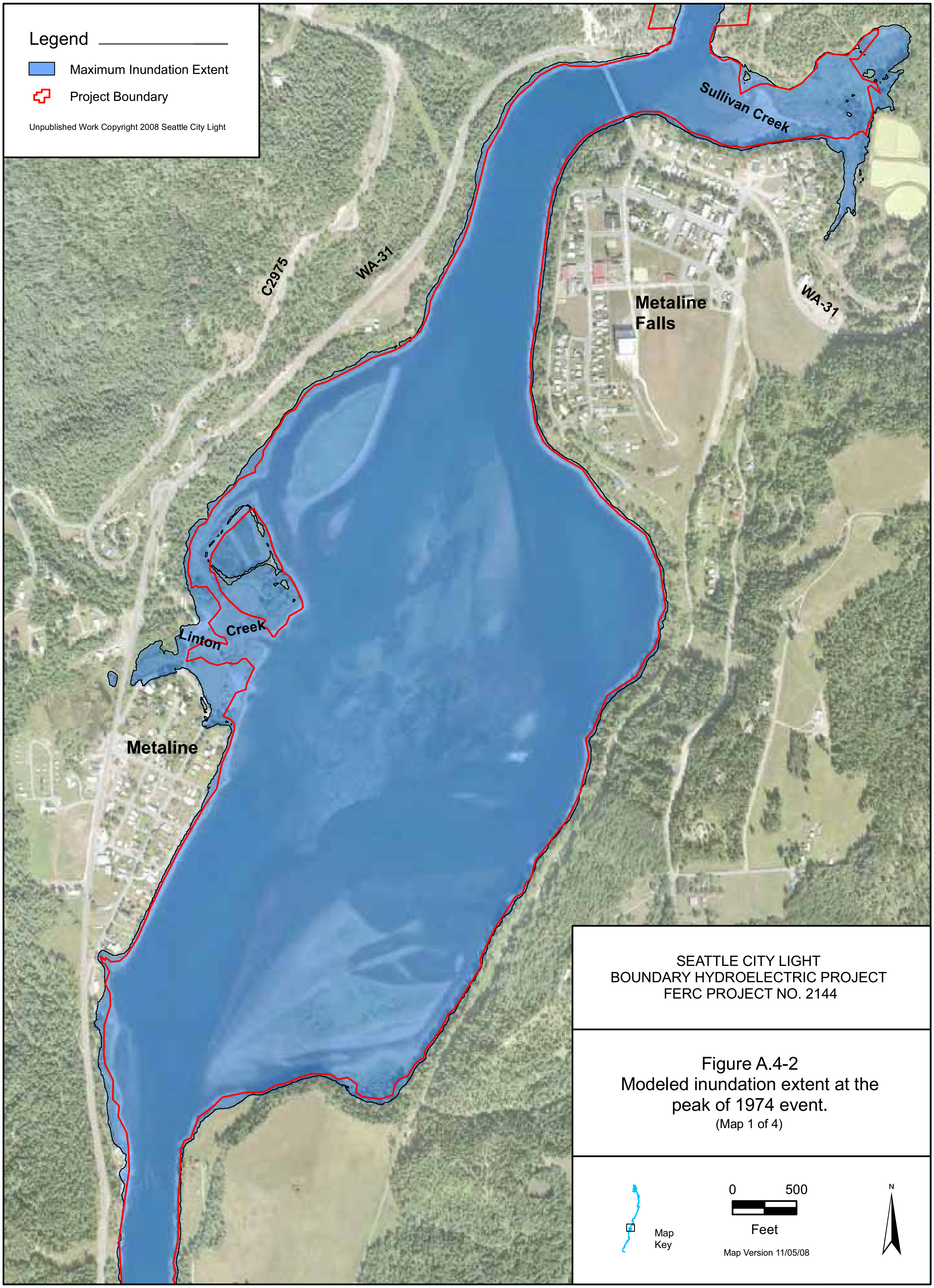

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Figure A.4-2
Modeled inundation extent at the peak of 1974 event.
(Map 1 of 4)

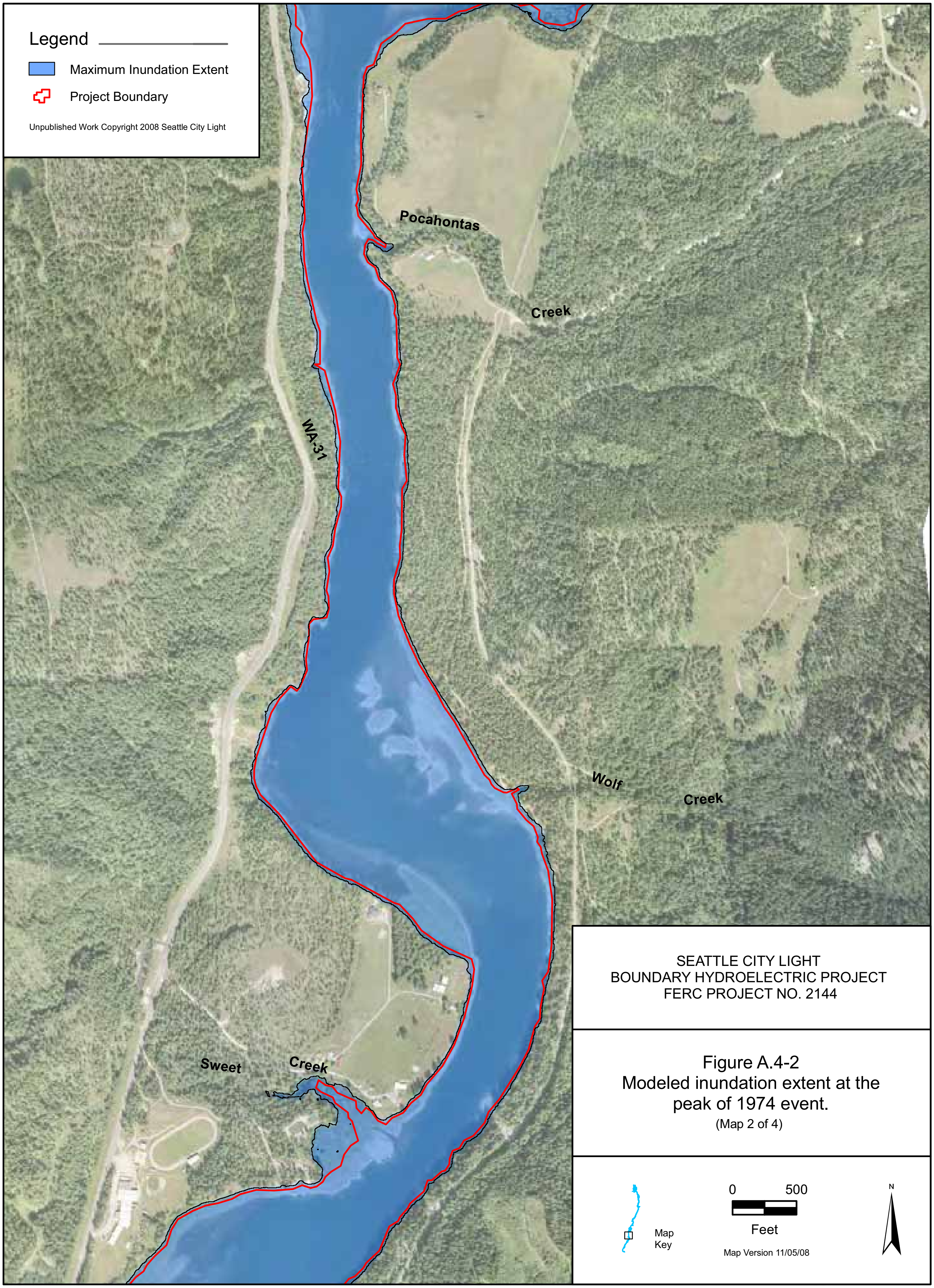

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Figure A.4-2
Modeled inundation extent at the peak of 1974 event.
(Map 2 of 4)

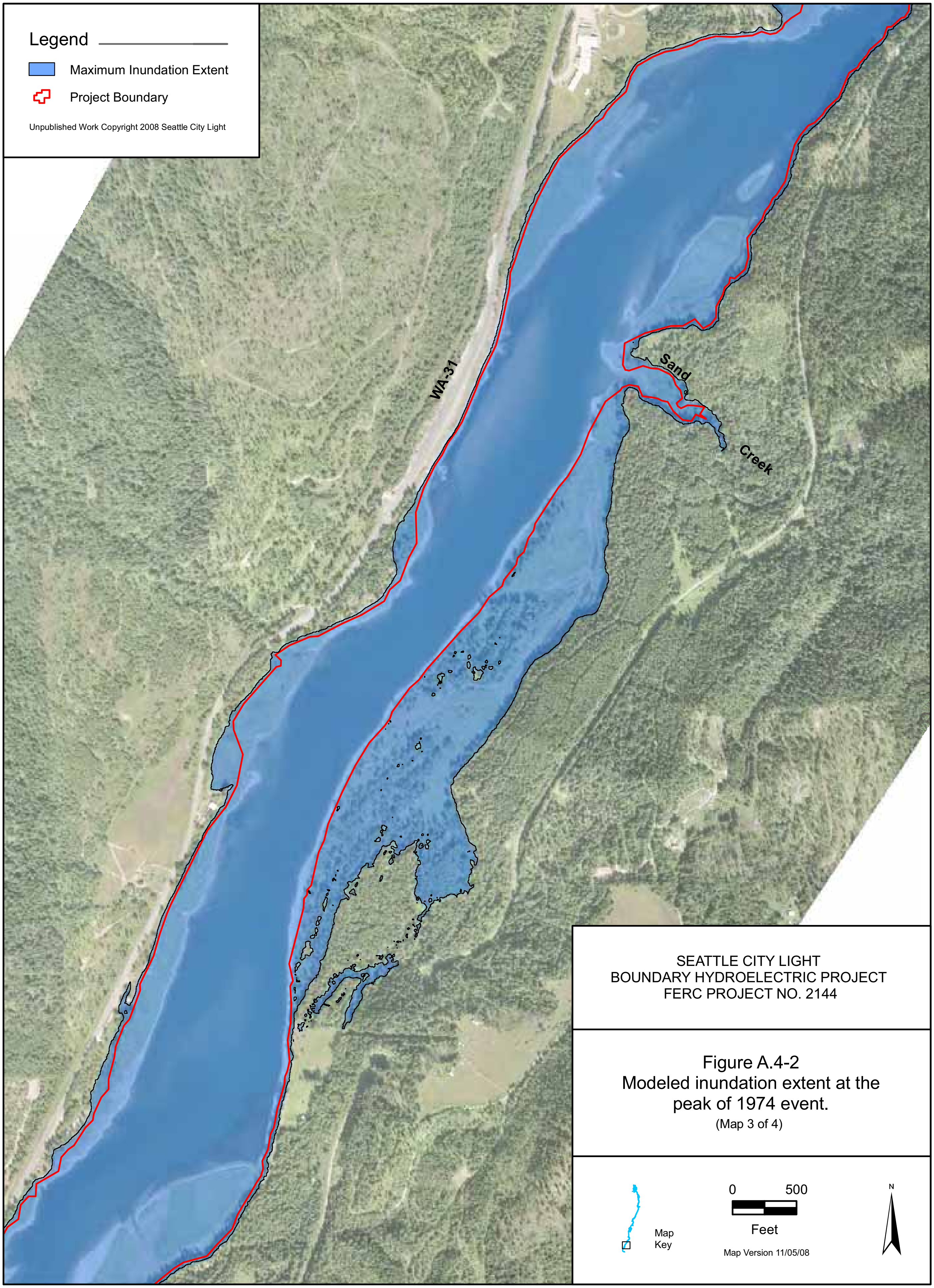

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Figure A.4-2
Modeled inundation extent at the peak of 1974 event.
(Map 3 of 4)

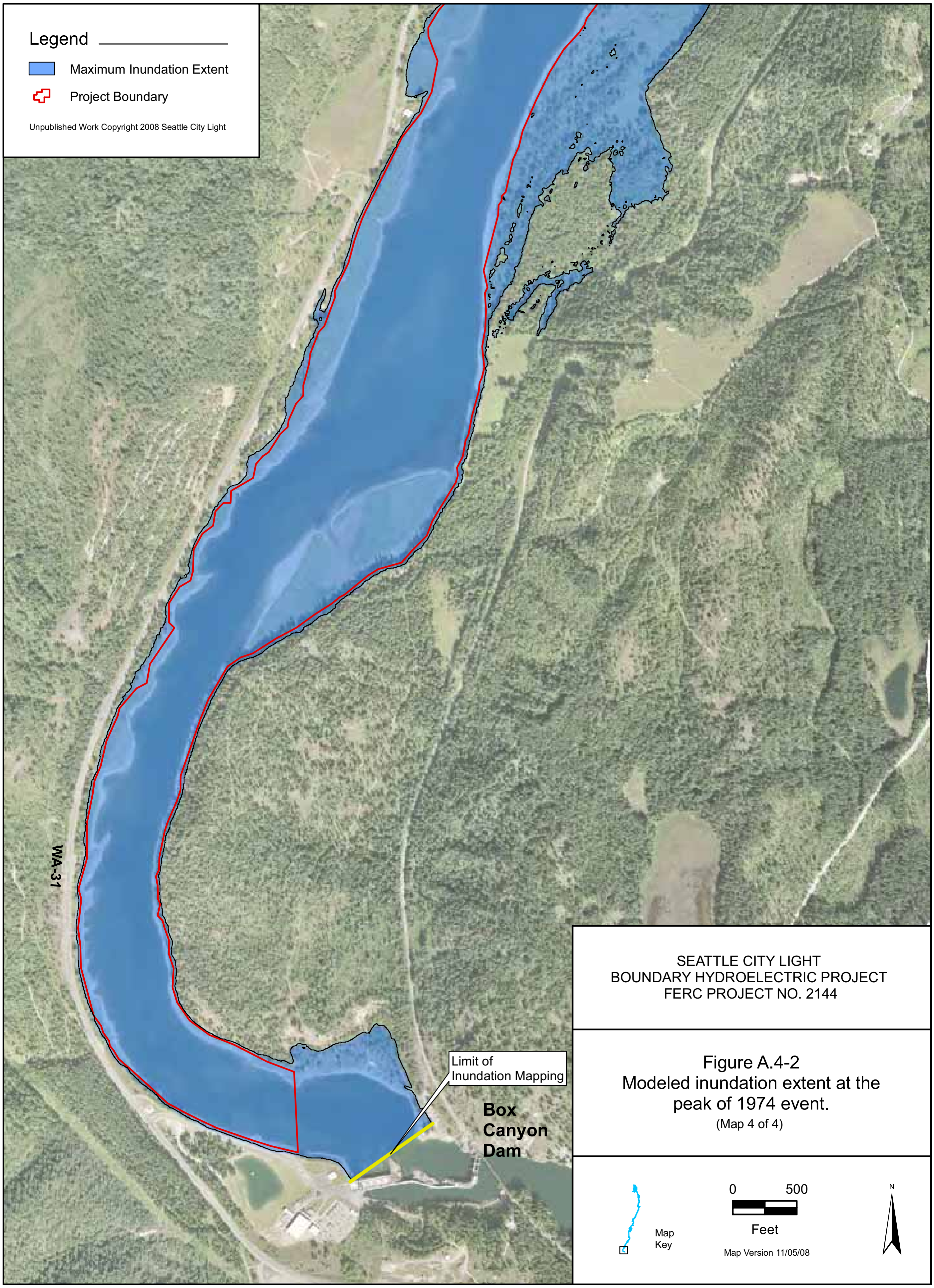

Figure A.4-2
Modeled inundation extent at the peak of 1974 event.
(Map 4 of 4)

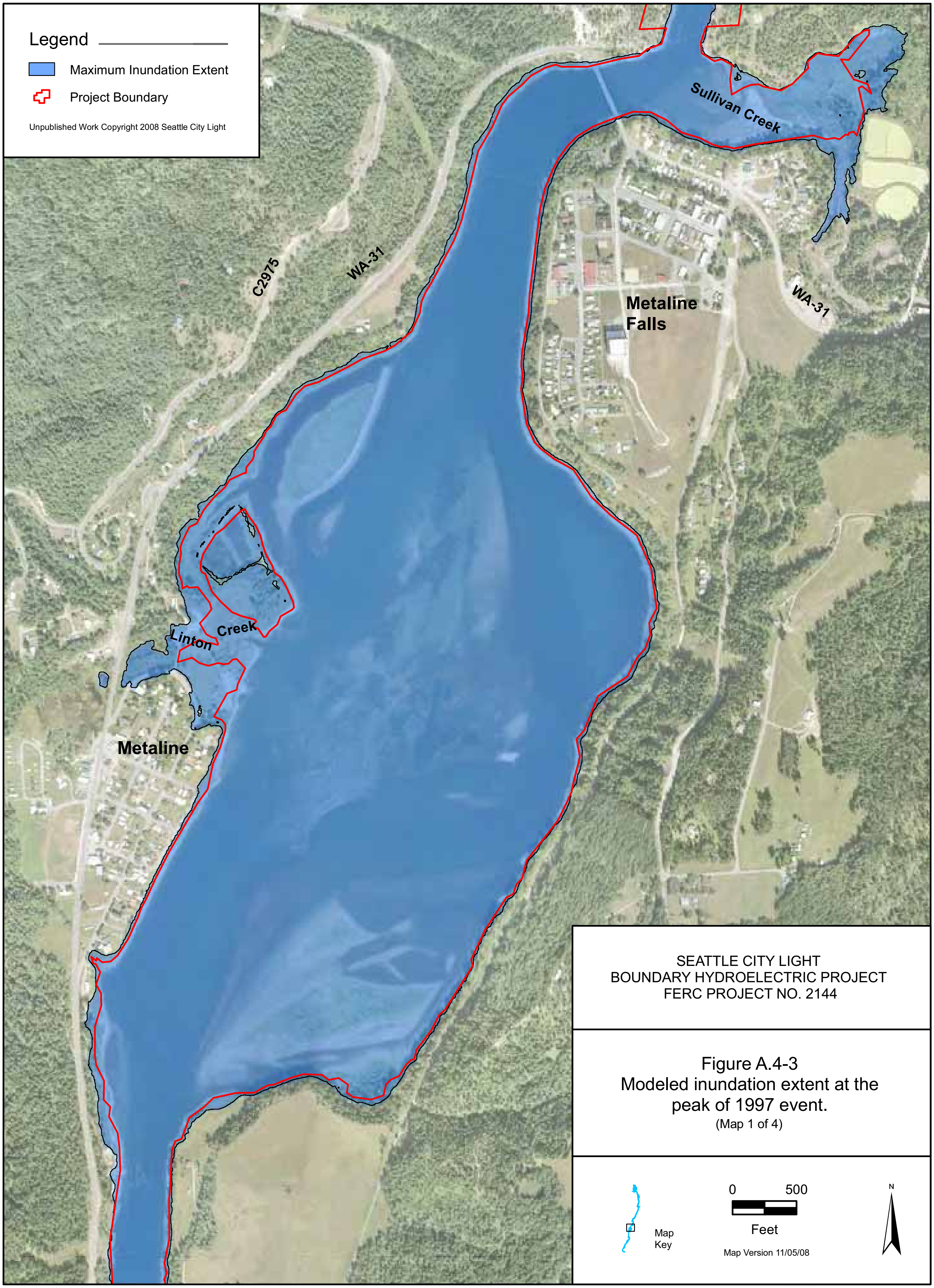

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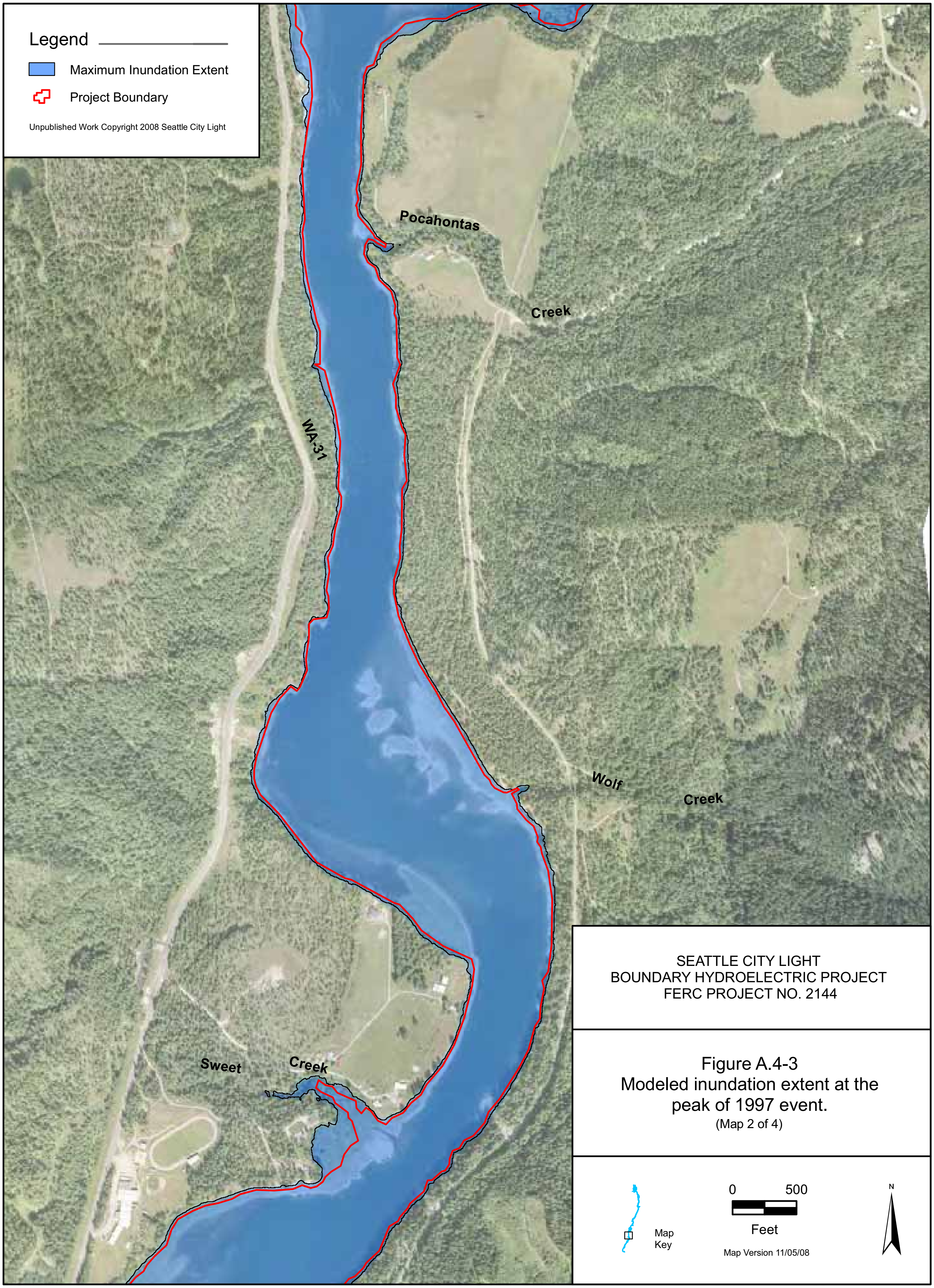
Figure A.4-3
Modeled inundation extent at the peak of 1997 event.
(Map 1 of 4)


Legend
Maximum Inundation Extent
Project Boundary
Unpublished Work Copyright 2008 Seattle City Light
Pocahontas
Creek
WA-31
Wolf
Creek
Sweet
Creek
SEATTLE CITY LIGHT
BOUNDARY HYDROELECTRIC PROJECT
FERC PROJECT NO. 2144
Figure A.4-3
Modeled inundation extent at the peak of 1997 event.
(Map 2 of 4)
Map Key
0
500
Feet
Map Version 11/05/08
N

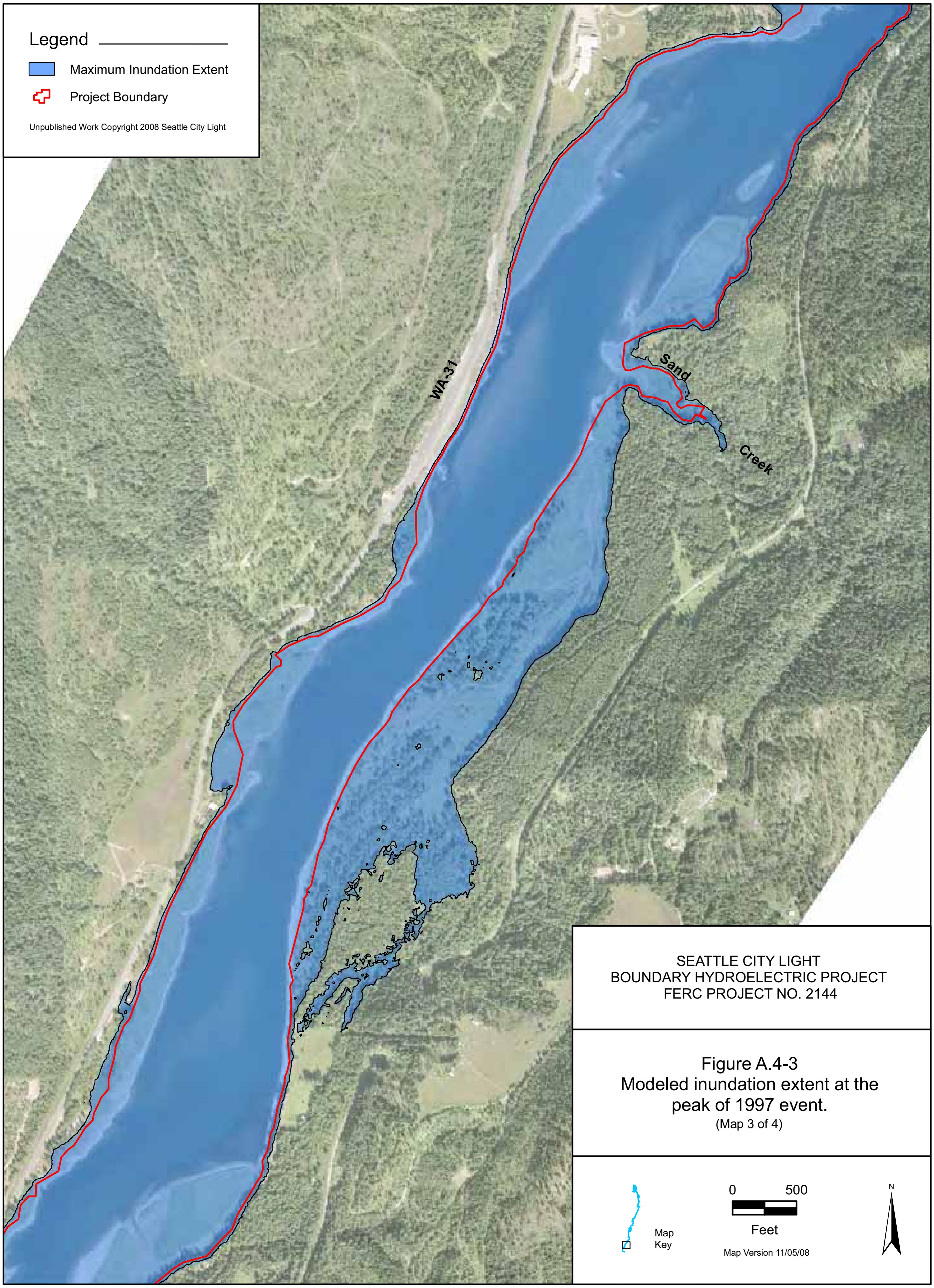


Figure A.4-3
Modeled inundation extent at the peak of 1997 event.
(Map 3 of 4)

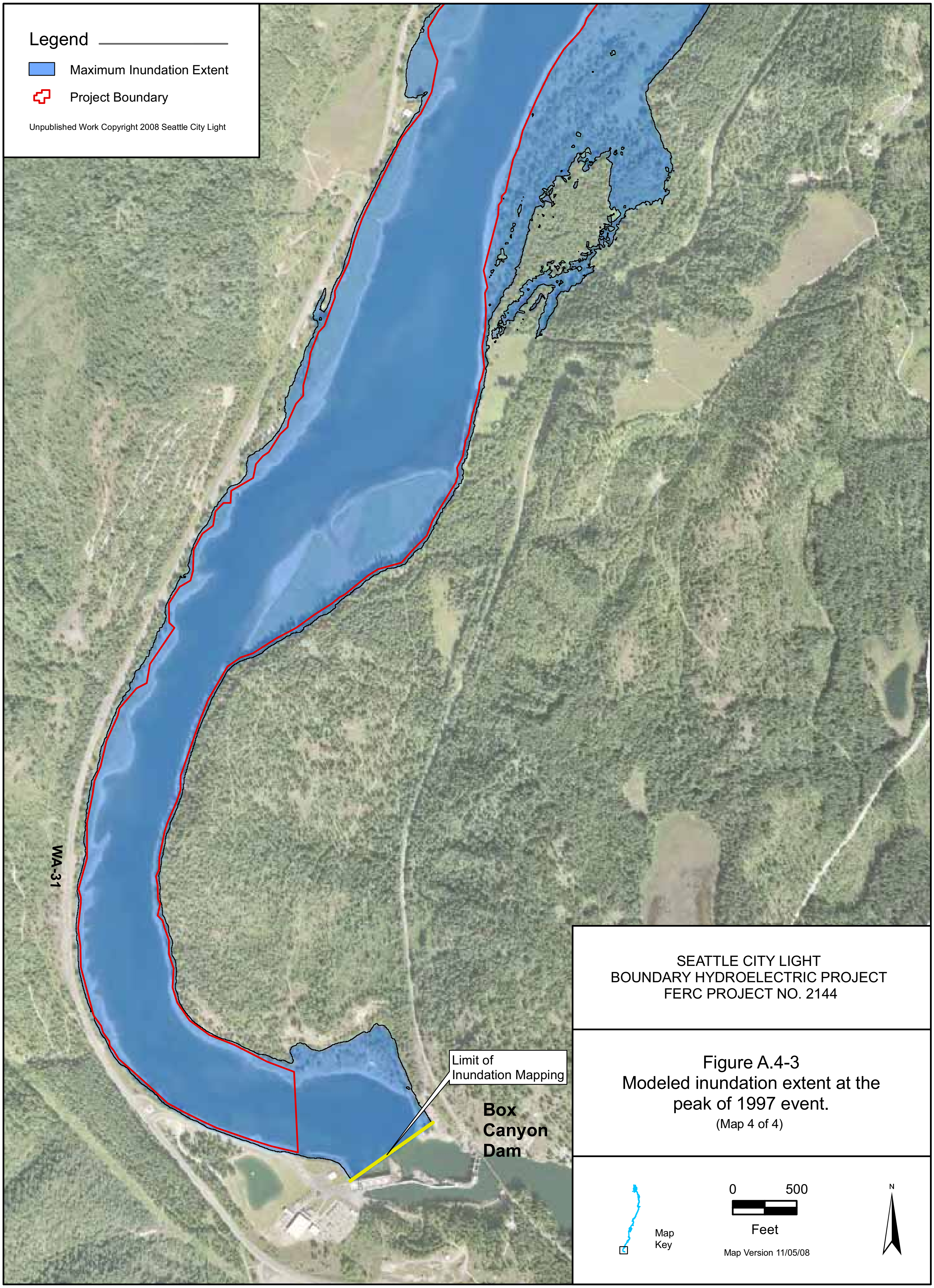


Figure A.4-3
Modeled inundation extent at the peak of 1997 event.
(Map 4 of 4)

Appendix 5: Model-Simulated Inundation Extents during the 1972, 1974, and 1997 Flood Event Periods without the Hydraulic Influence of Project Operations

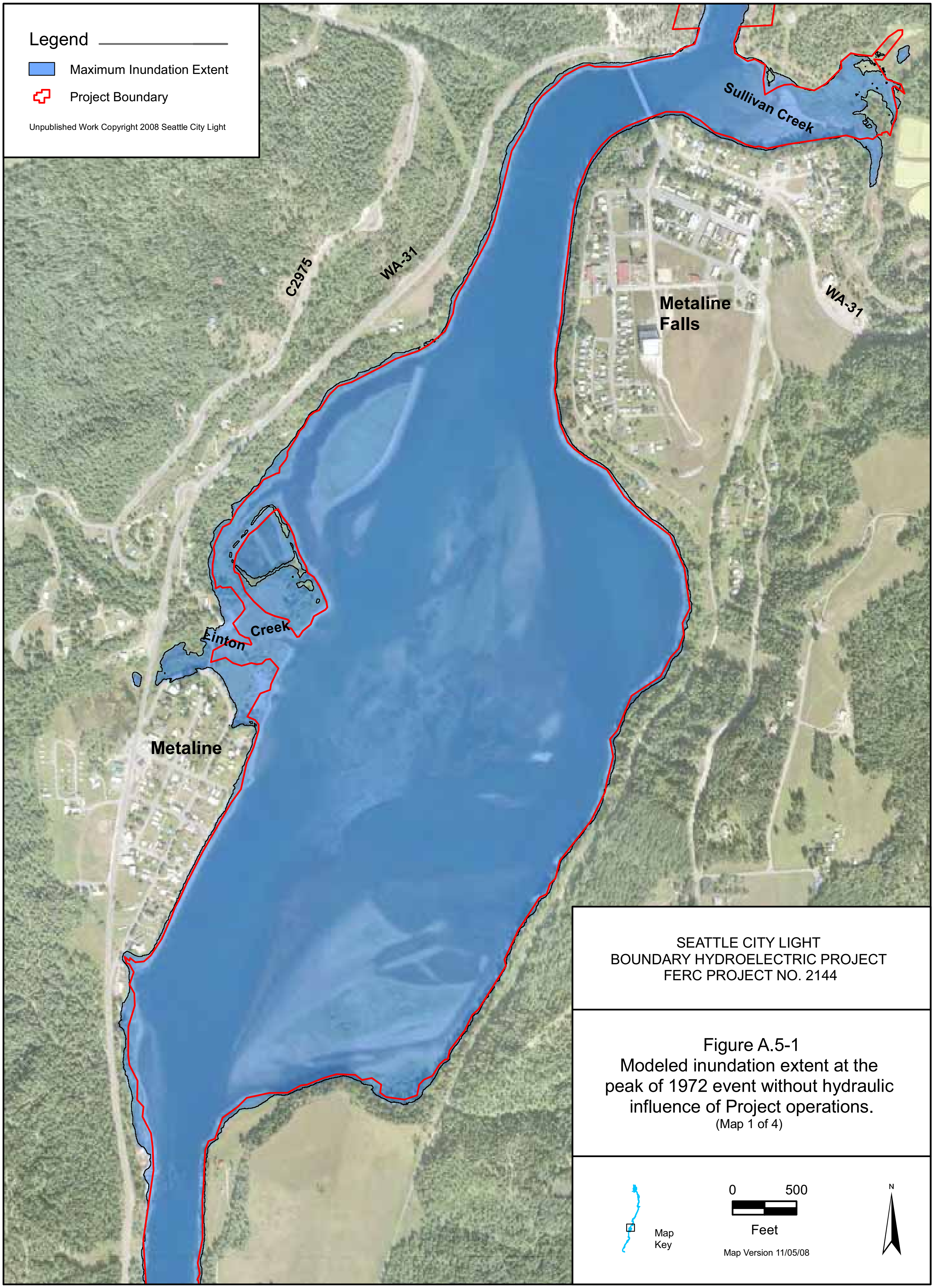


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Figure A.5-1
Modeled inundation extent at the peak of 1972 event without hydraulic influence of Project operations.
(Map 1 of 4)

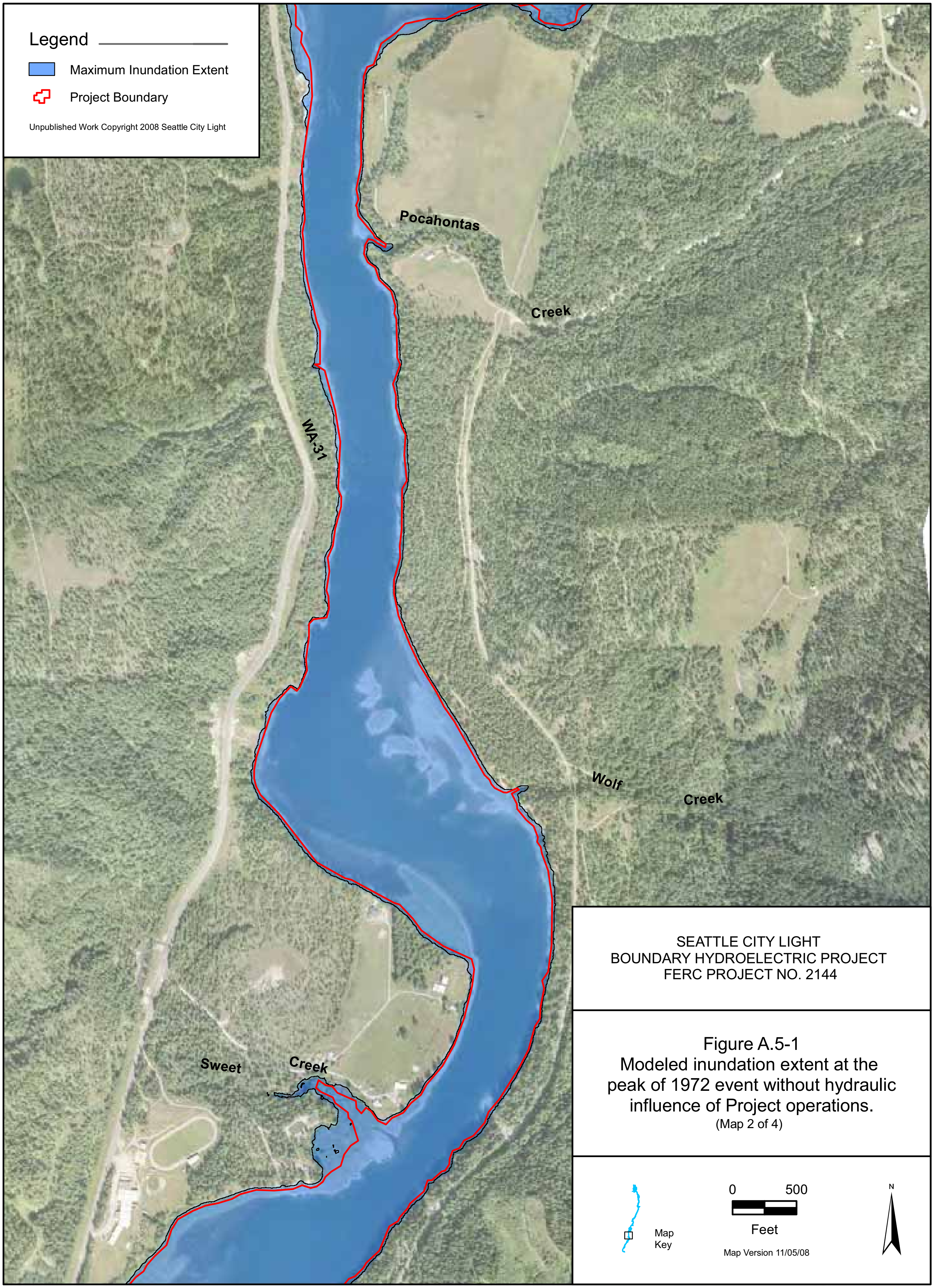

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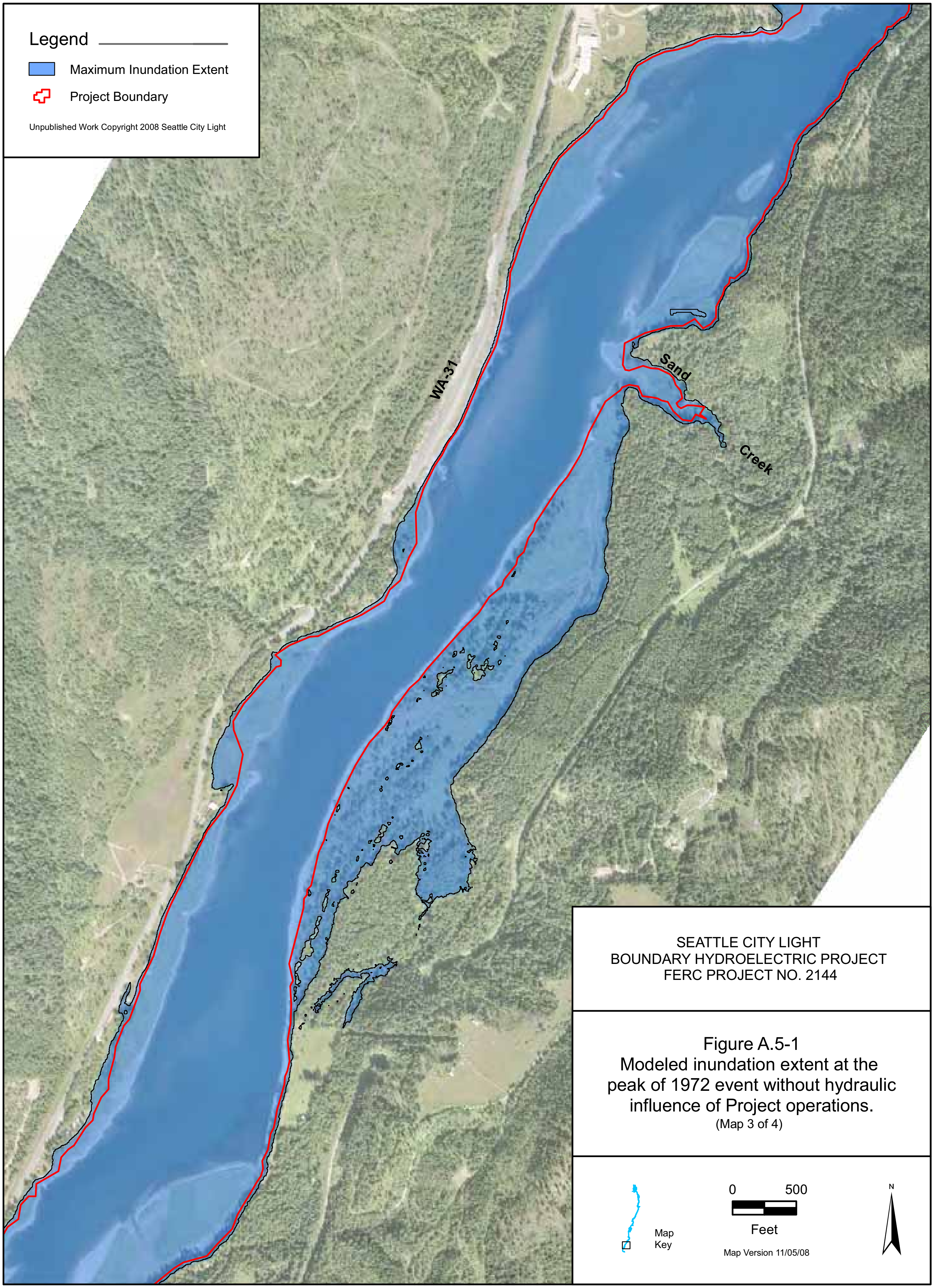
Figure A.5-1
Modeled inundation extent at the peak of 1972 event without hydraulic influence of Project operations.
(Map 2 of 4)



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FERC PROJECT NO. 2144

Figure A.5-1
Modeled inundation extent at the peak of 1972 event without hydraulic influence of Project operations.
(Map 3 of 4)

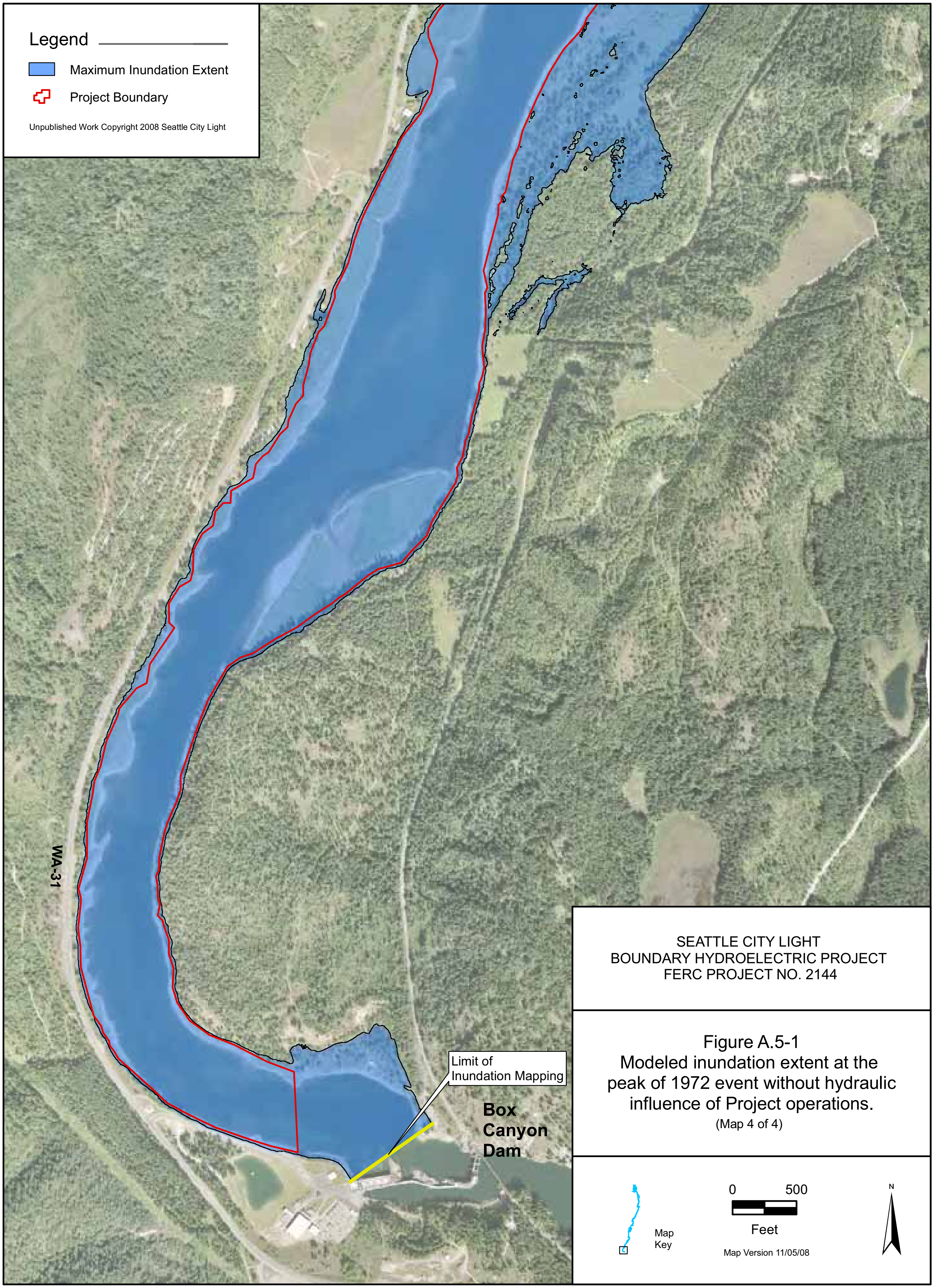

Figure A.5-1
Modeled inundation extent at the peak of 1972 event without hydraulic influence of Project operations.
(Map 4 of 4)

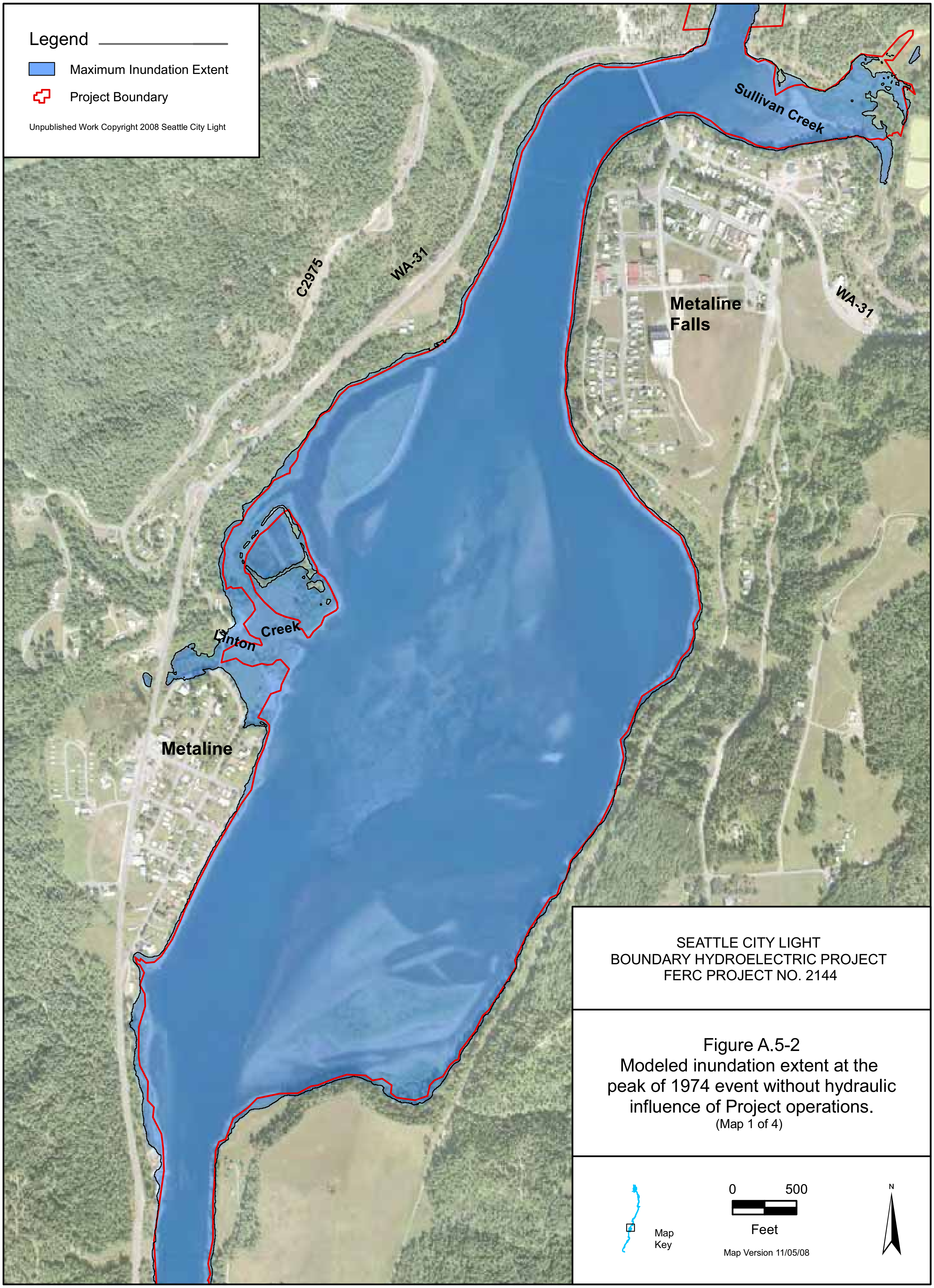

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Figure A.5-2
Modeled inundation extent at the peak of 1974 event without hydraulic influence of Project operations.
(Map 1 of 4)

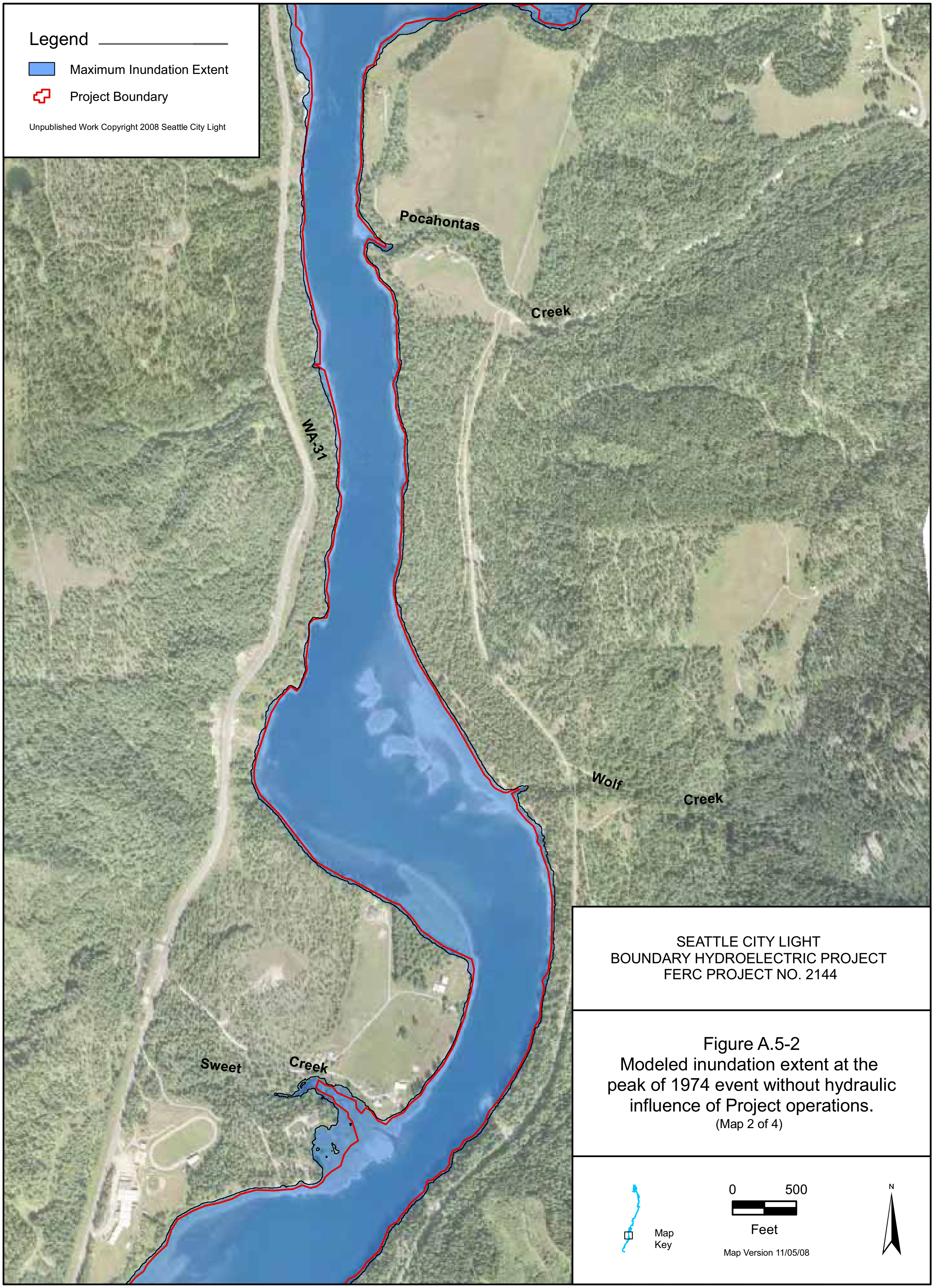

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Figure A.5-2
Modeled inundation extent at the peak of 1974 event without hydraulic influence of Project operations.
(Map 2 of 4)

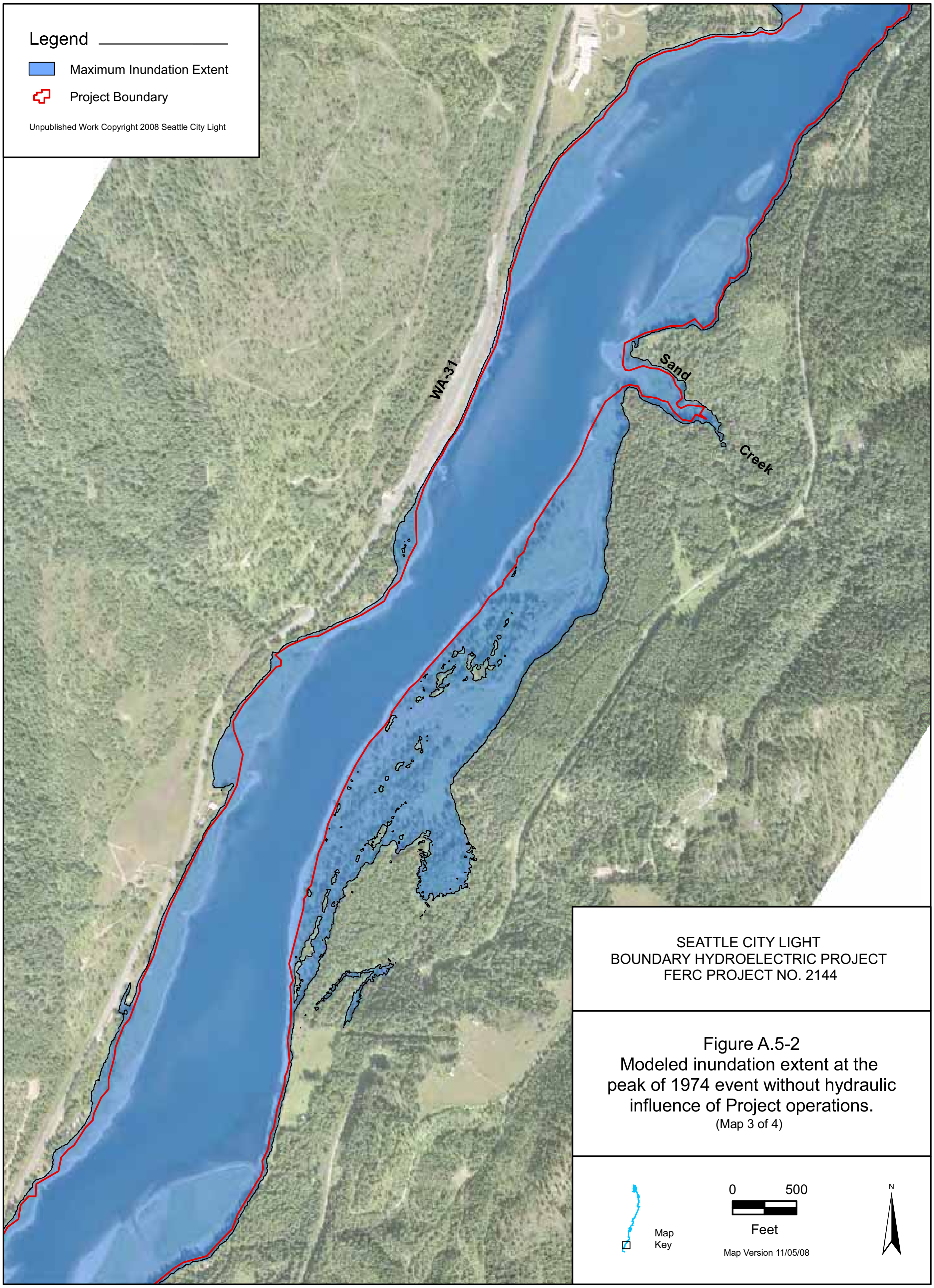

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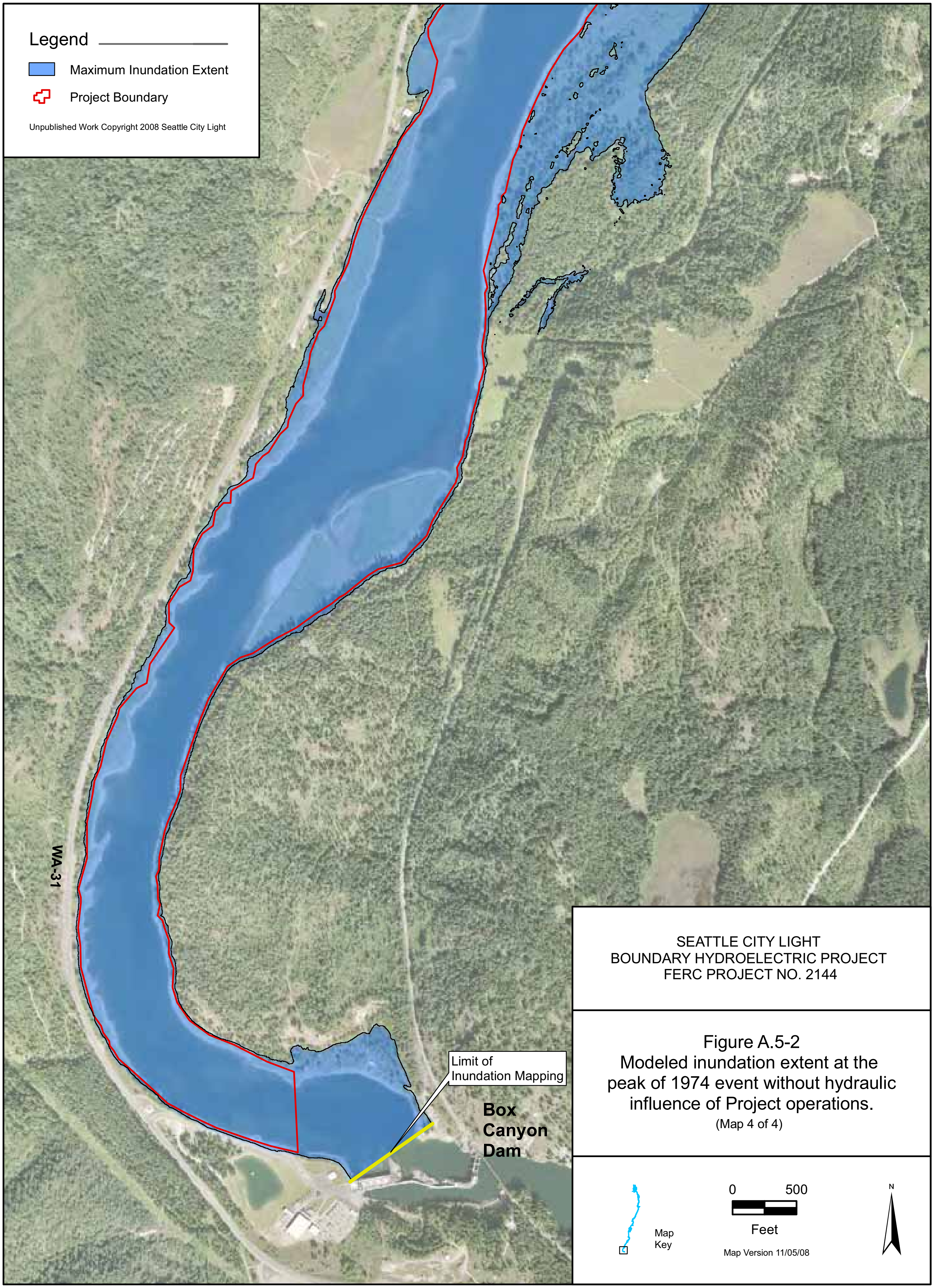
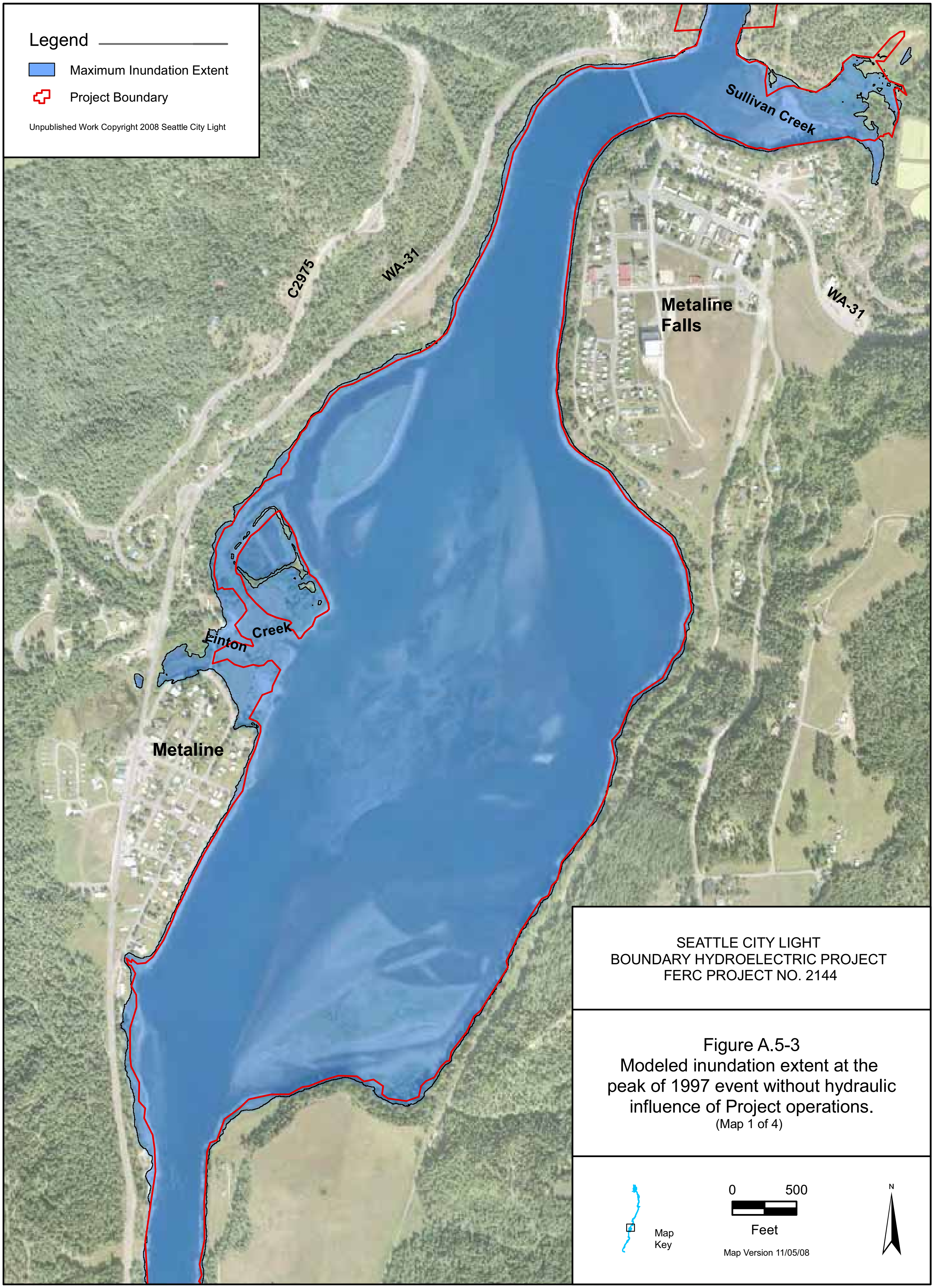
Figure A.5-2
Modeled inundation extent at the peak of 1974 event without hydraulic influence of Project operations.
(Map 3 of 4)



SEATTLE CITY LIGHT
BOUNDARY HYDROELECTRIC PROJECT
FERC PROJECT NO. 2144

Figure A.5-2
Modeled inundation extent at the peak of 1974 event without hydraulic influence of Project operations.
(Map 4 of 4)



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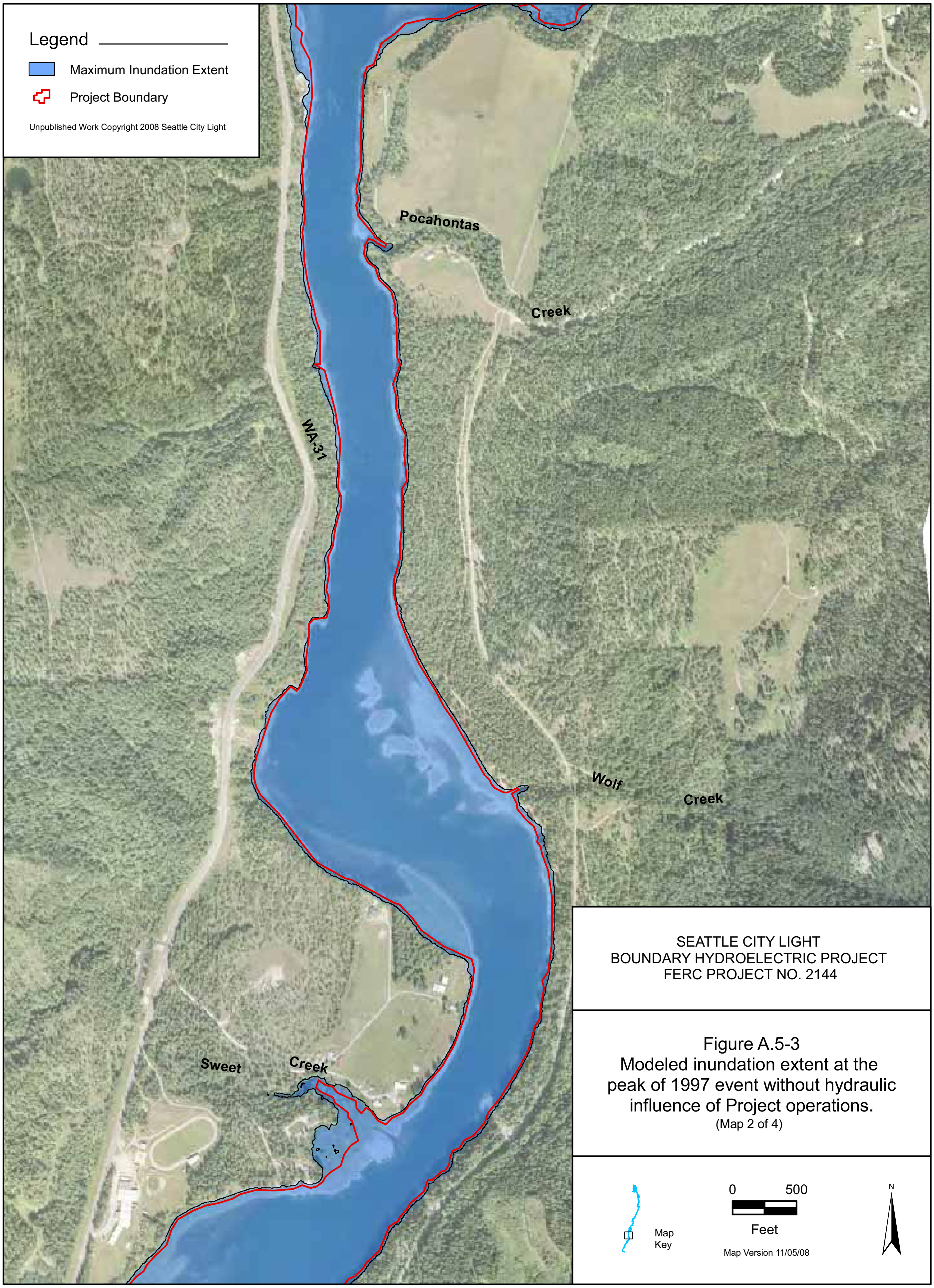
BOUNDARY HYDROELECTRIC PROJECT
FERC PROJECT NO. 2144

Figure A.5-3
Modeled inundation extent at the peak of 1997 event without hydraulic influence of Project operations.
(Map 1 of 4)



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BOUNDARY HYDROELECTRIC PROJECT
FERC PROJECT NO. 2144

Figure A.5-3
Modeled inundation extent at the peak of 1997 event without hydraulic influence of Project operations.
(Map 2 of 4)

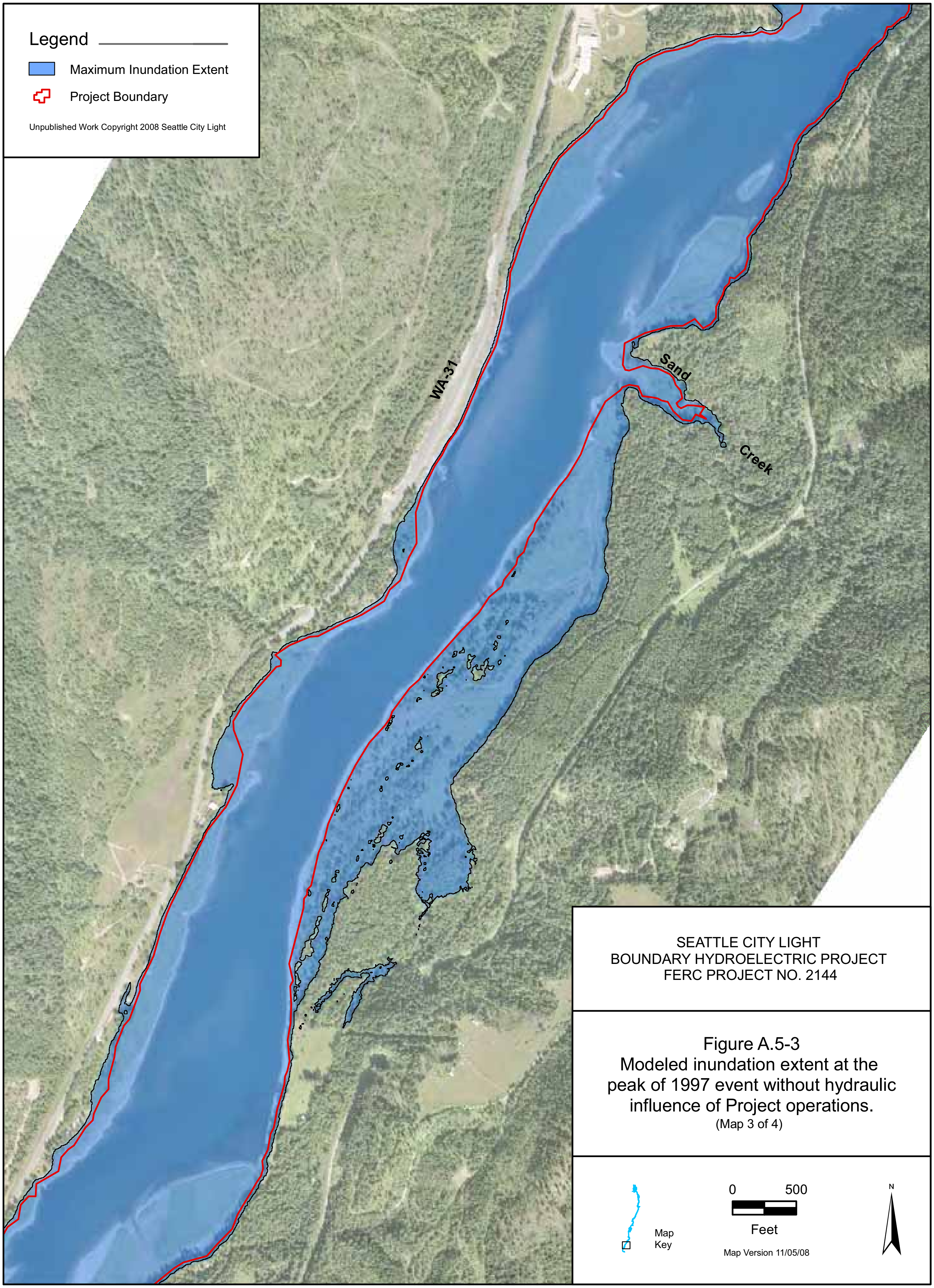

Figure A.5-3
Modeled inundation extent at the peak of 1997 event without hydraulic influence of Project operations.
(Map 3 of 4)

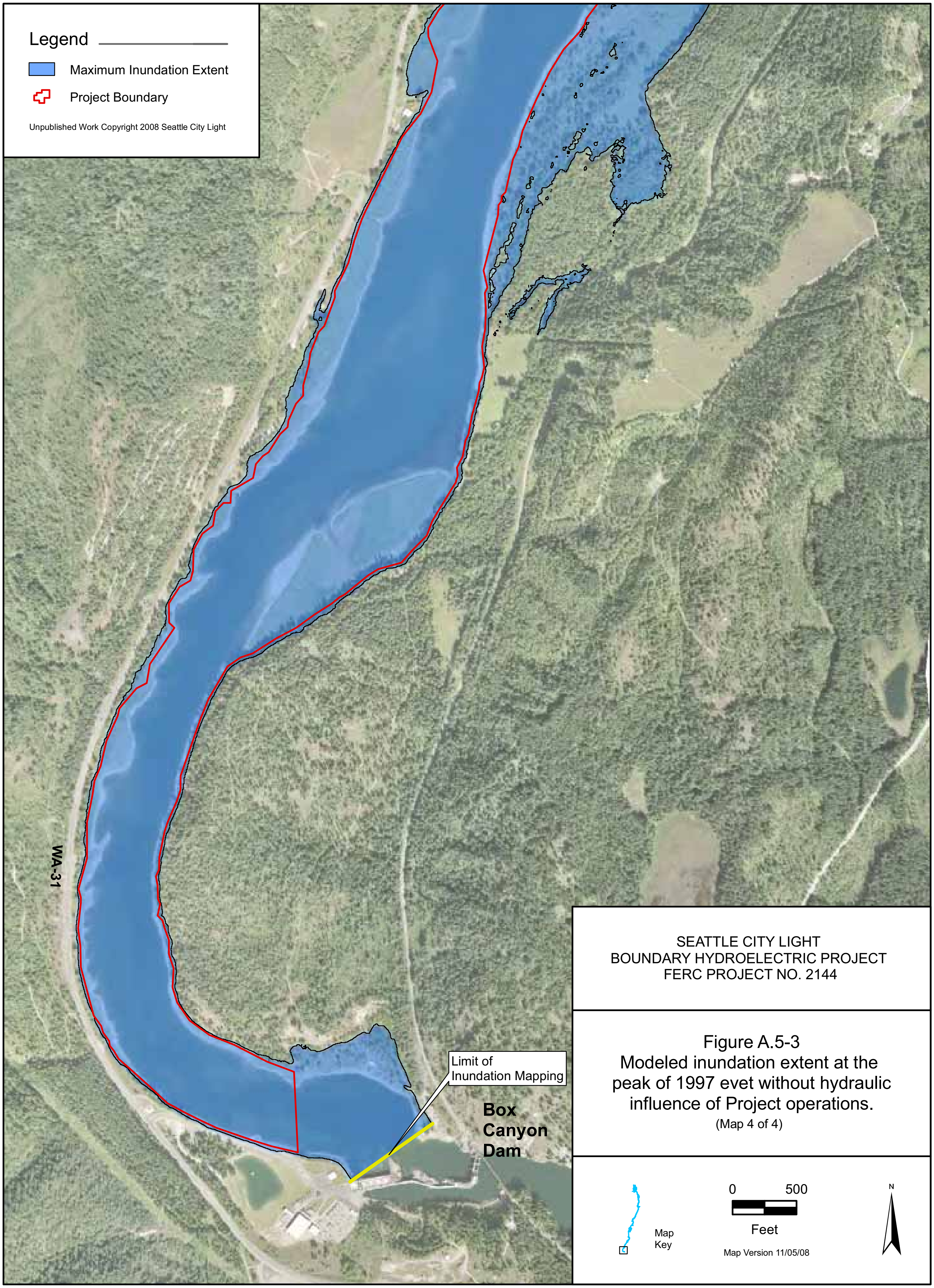

Figure A.5-3
Modeled inundation extent at the peak of 1997 evet without hydraulic influence of Project operations.
(Map 4 of 4)

Appendix 6: Estimated Project Effects during the 1972, 1974, and 1997 Flood Event Periods

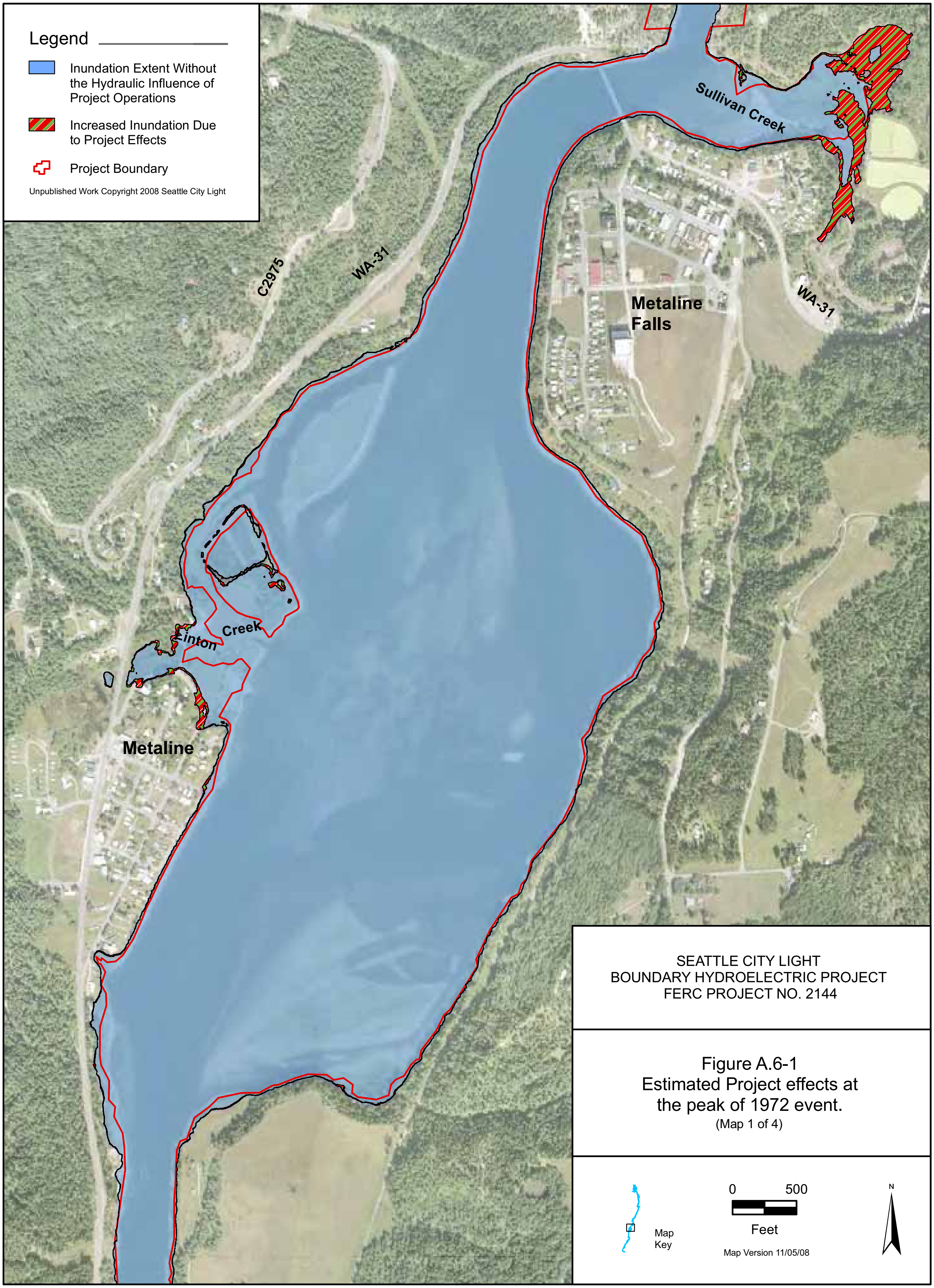

Figure A.6-1
Estimated Project effects at the peak of 1972 event.
(Map 1 of 4)

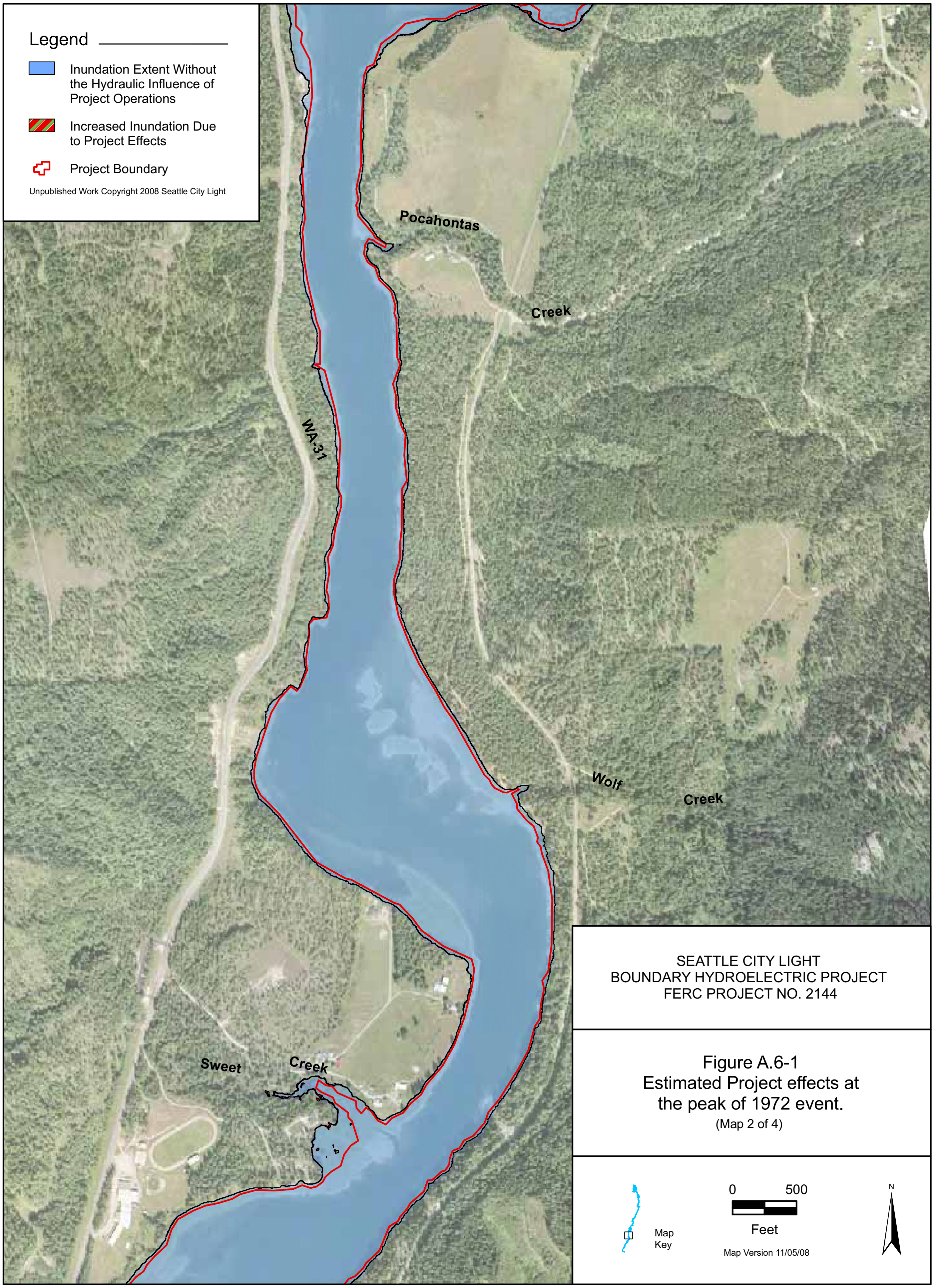

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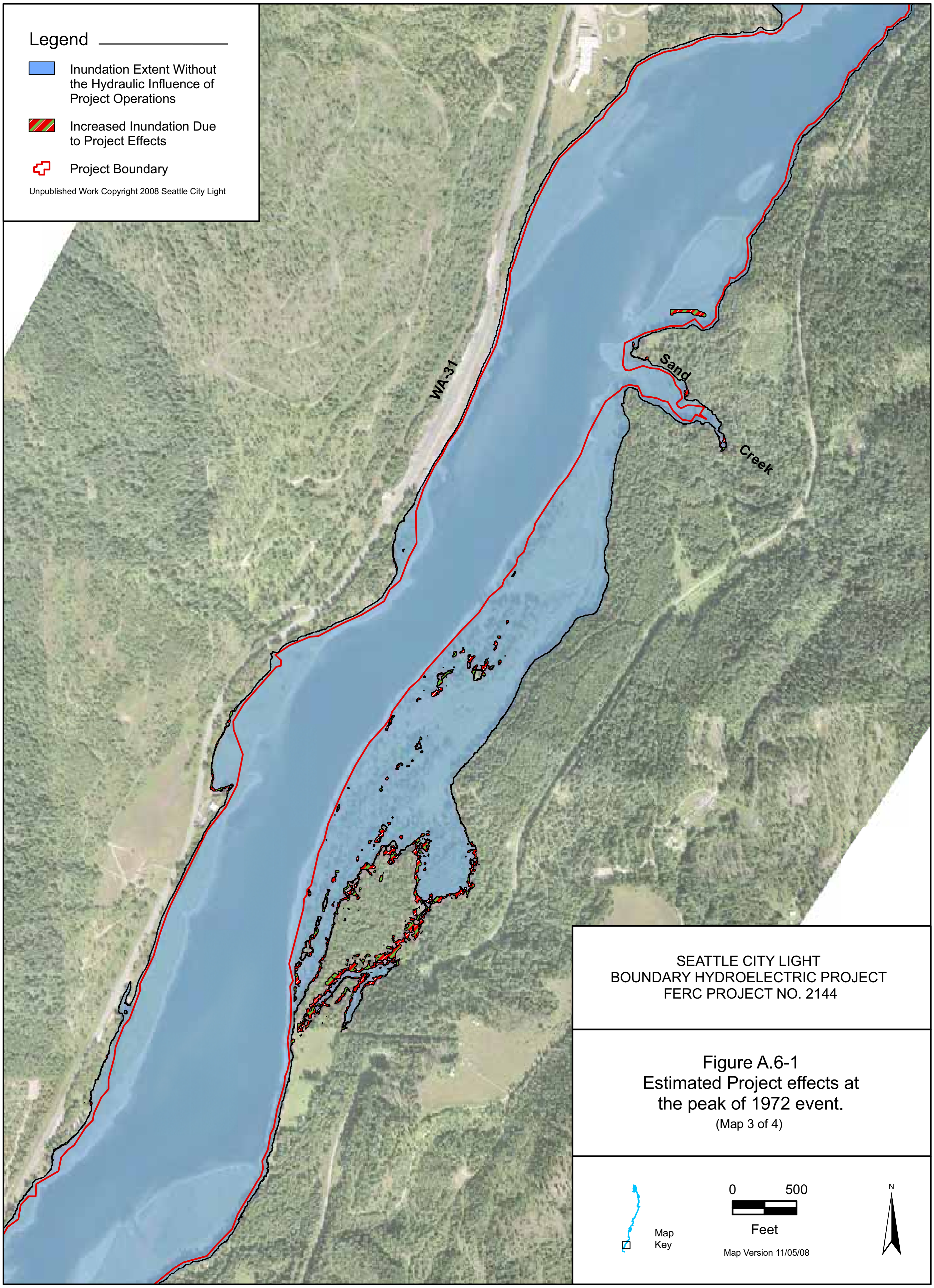
Figure A.6-1
Estimated Project effects at the peak of 1972 event.
(Map 2 of 4)


Legend
Inundation Extent Without the Hydraulic Influence of Project Operations
Increased Inundation Due to Project Effects
Project Boundary
Unpublished Work Copyright 2008 Seattle City Light
WA-31
Sand
Creek
SEATTLE CITY LIGHT
BOUNDARY HYDROELECTRIC PROJECT
FERC PROJECT NO. 2144
Figure A.6-1
Estimated Project effects at the peak of 1972 event.
(Map 3 of 4)
Map Key
0
500
Feet
Map Version 11/05/08
N

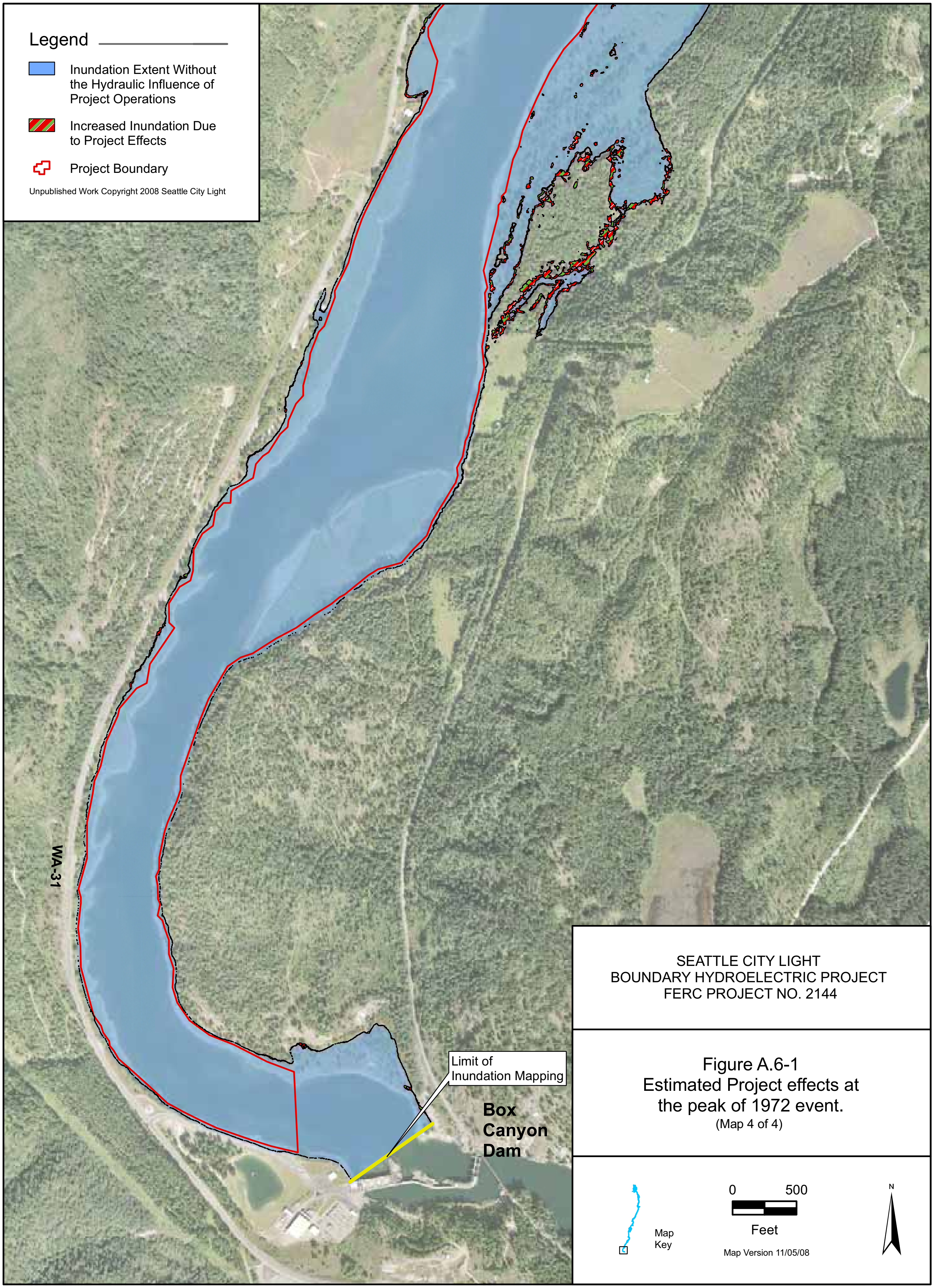


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Figure A.6-1
Estimated Project effects at the peak of 1972 event.
(Map 4 of 4)

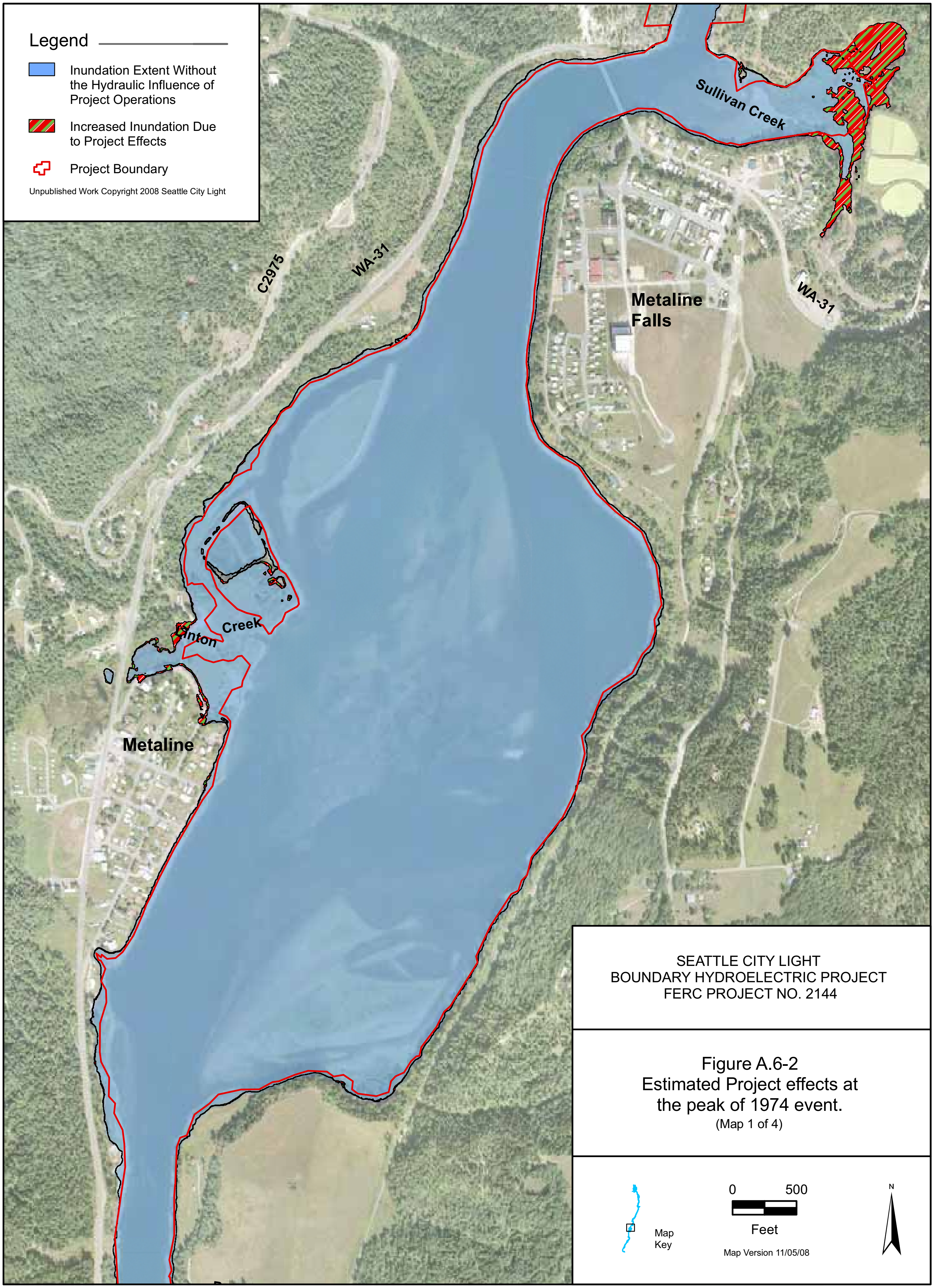

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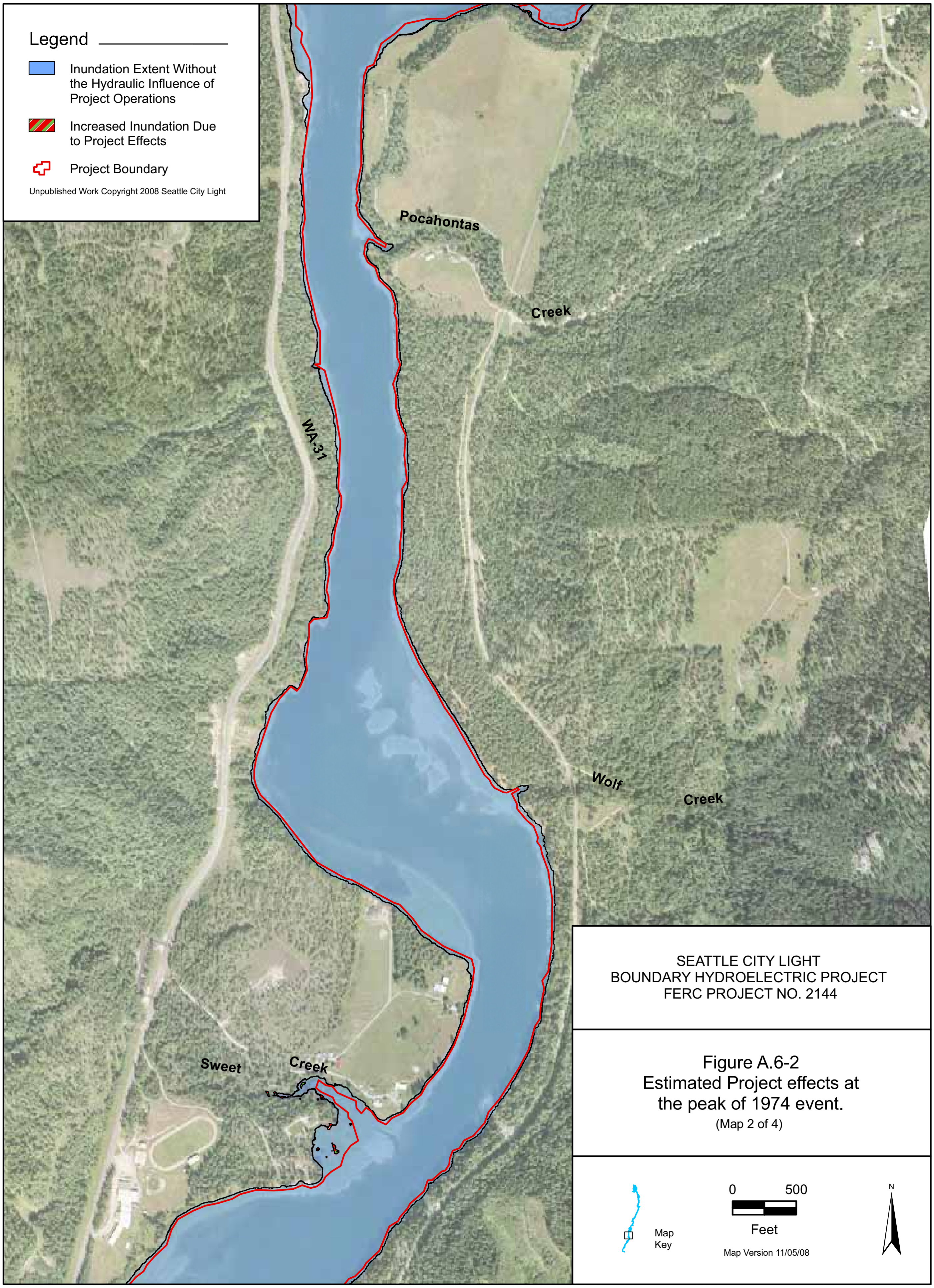
Figure A.6-2
Estimated Project effects at the peak of 1974 event.
(Map 1 of 4)



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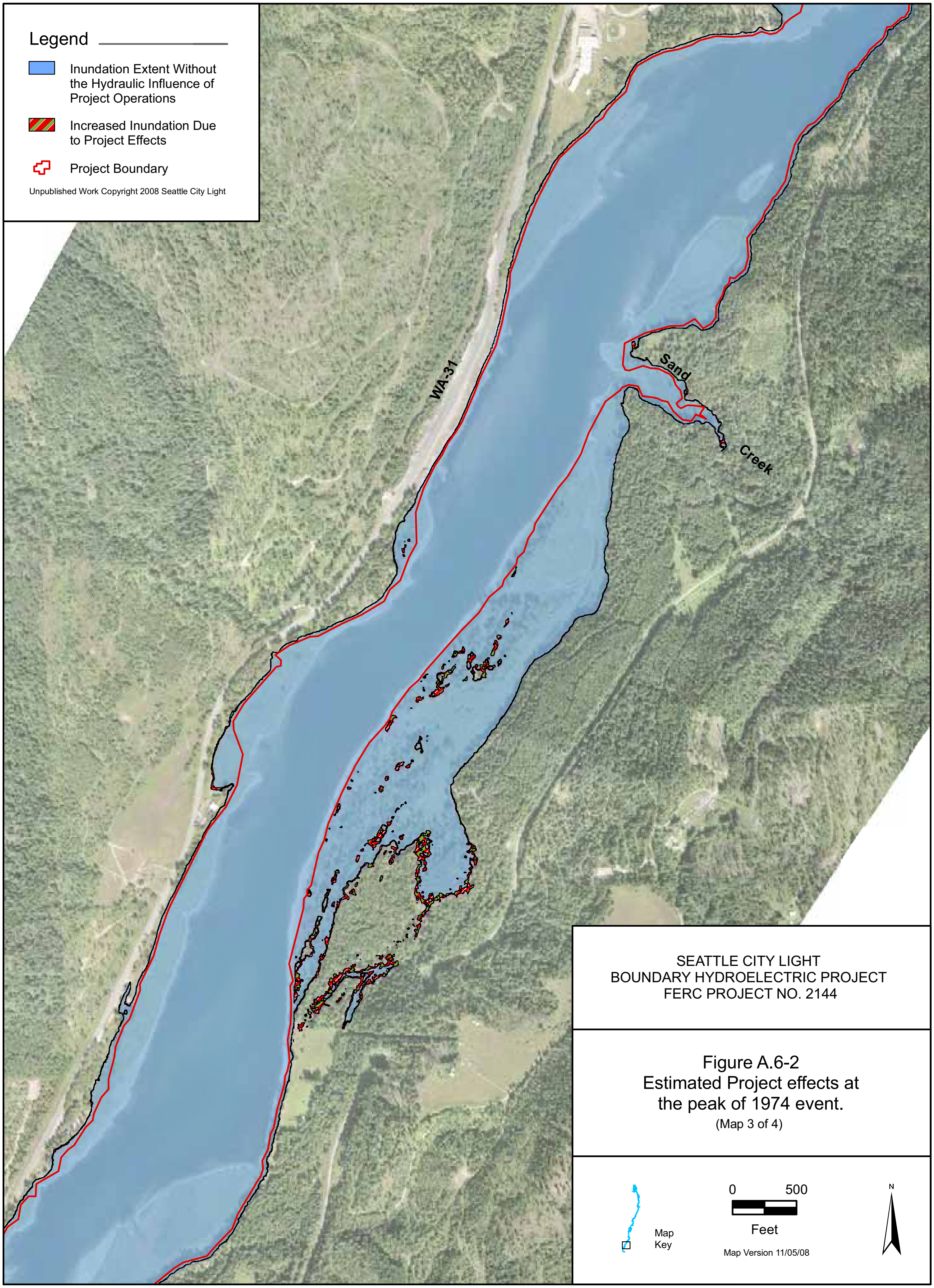
FERC PROJECT NO. 2144

Figure A.6-2
Estimated Project effects at the peak of 1974 event.
(Map 2 of 4)



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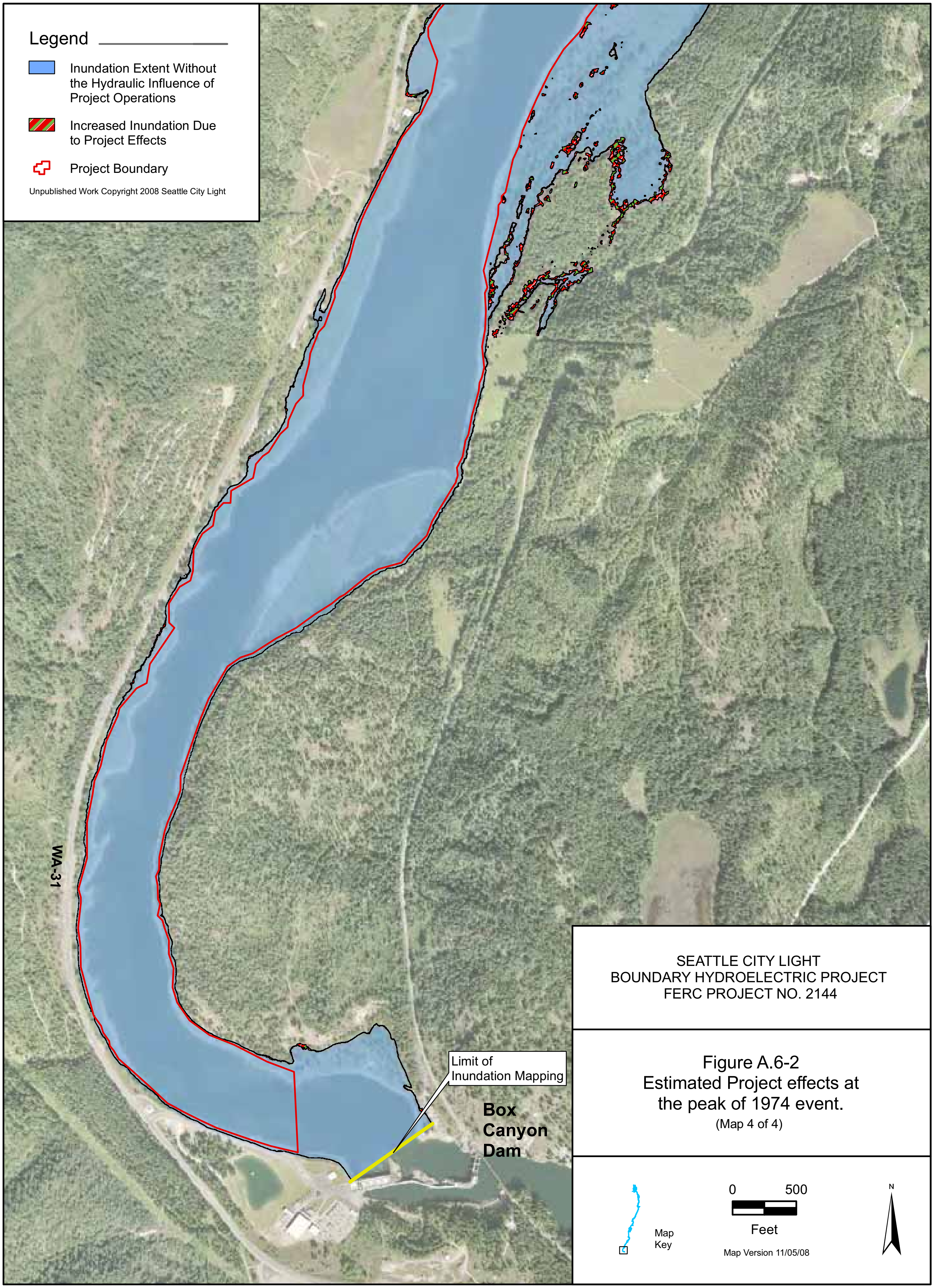
FERC PROJECT NO. 2144

Figure A.6-2
Estimated Project effects at the peak of 1974 event.
(Map 3 of 4)



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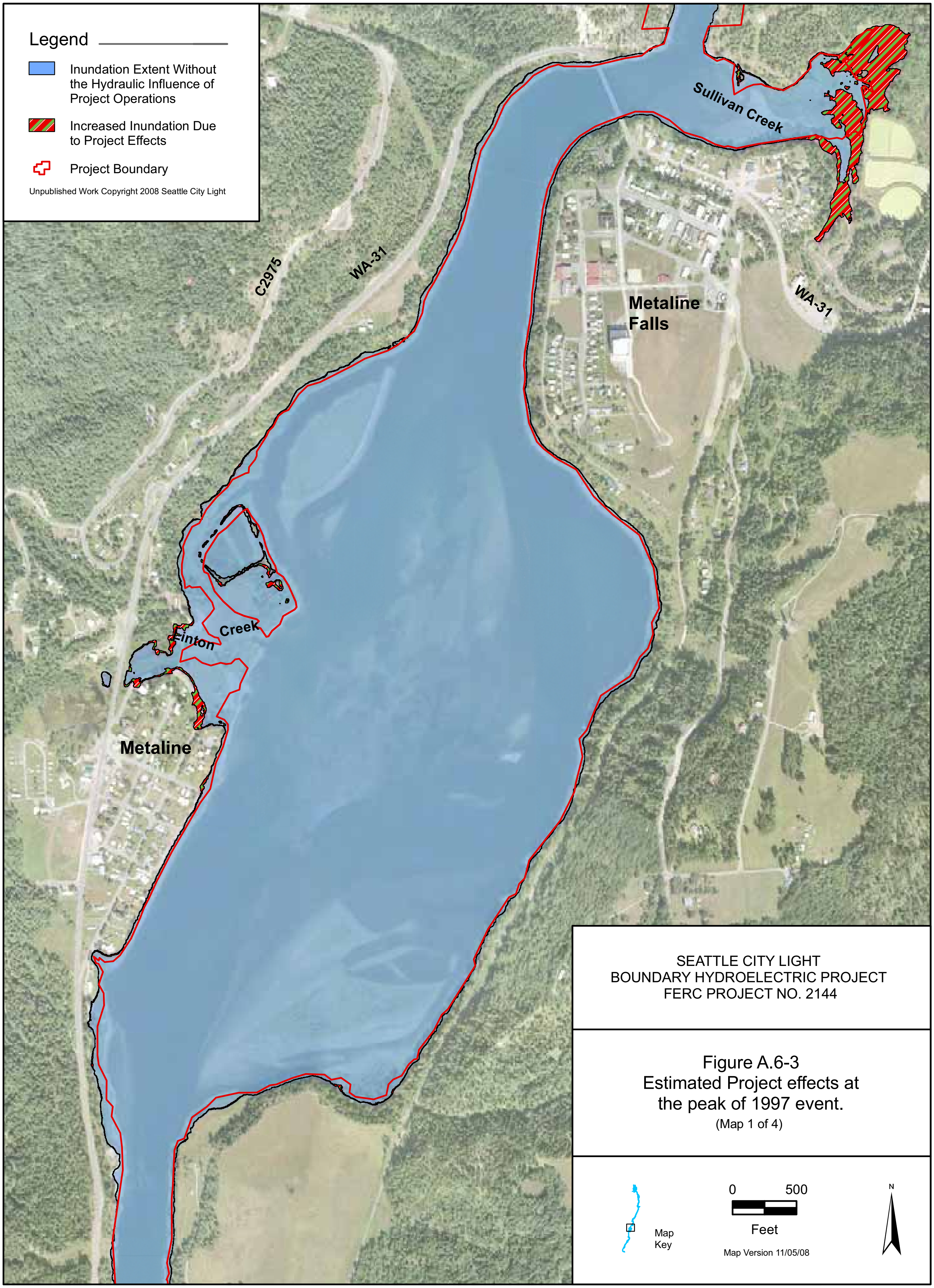
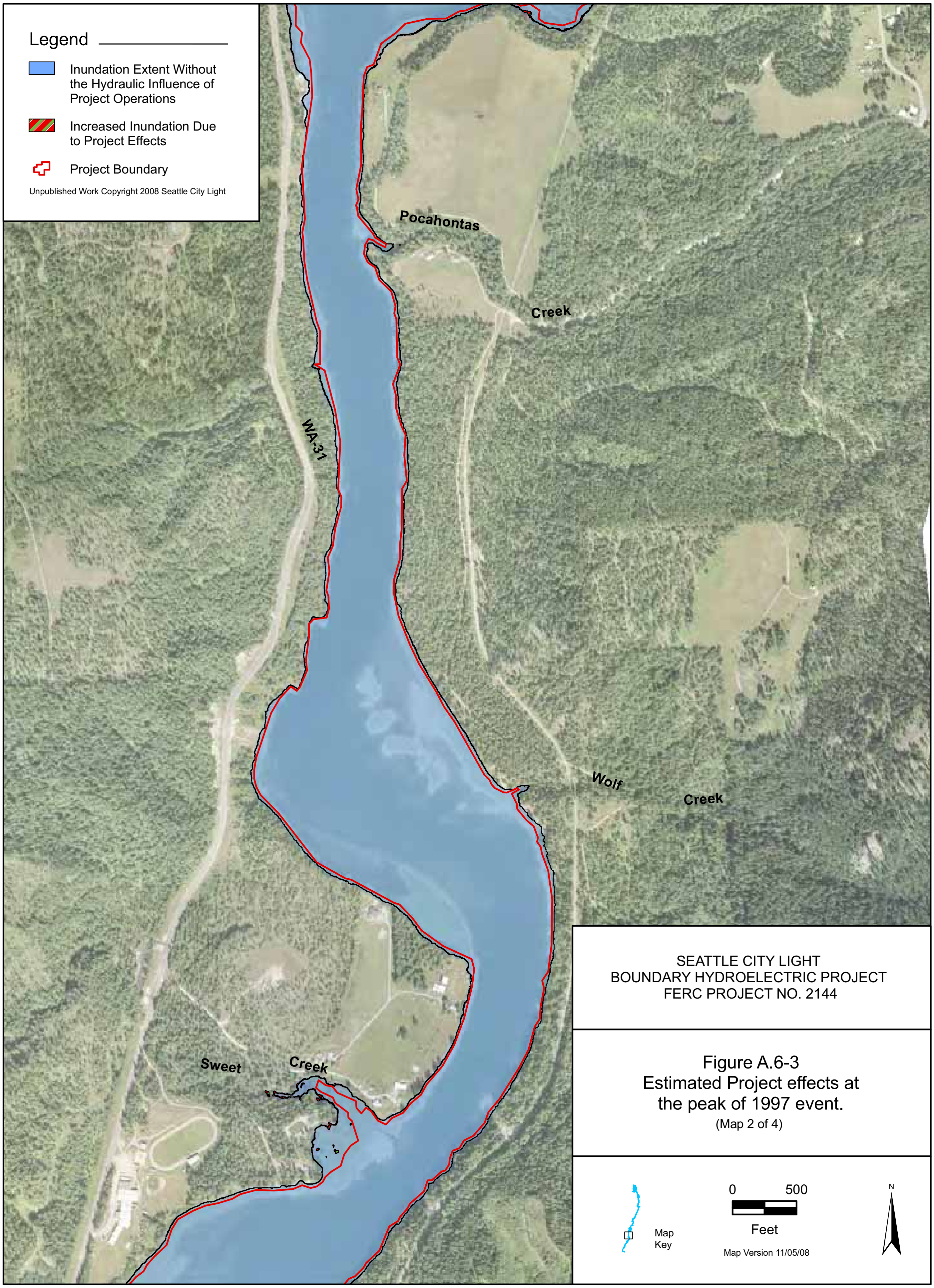
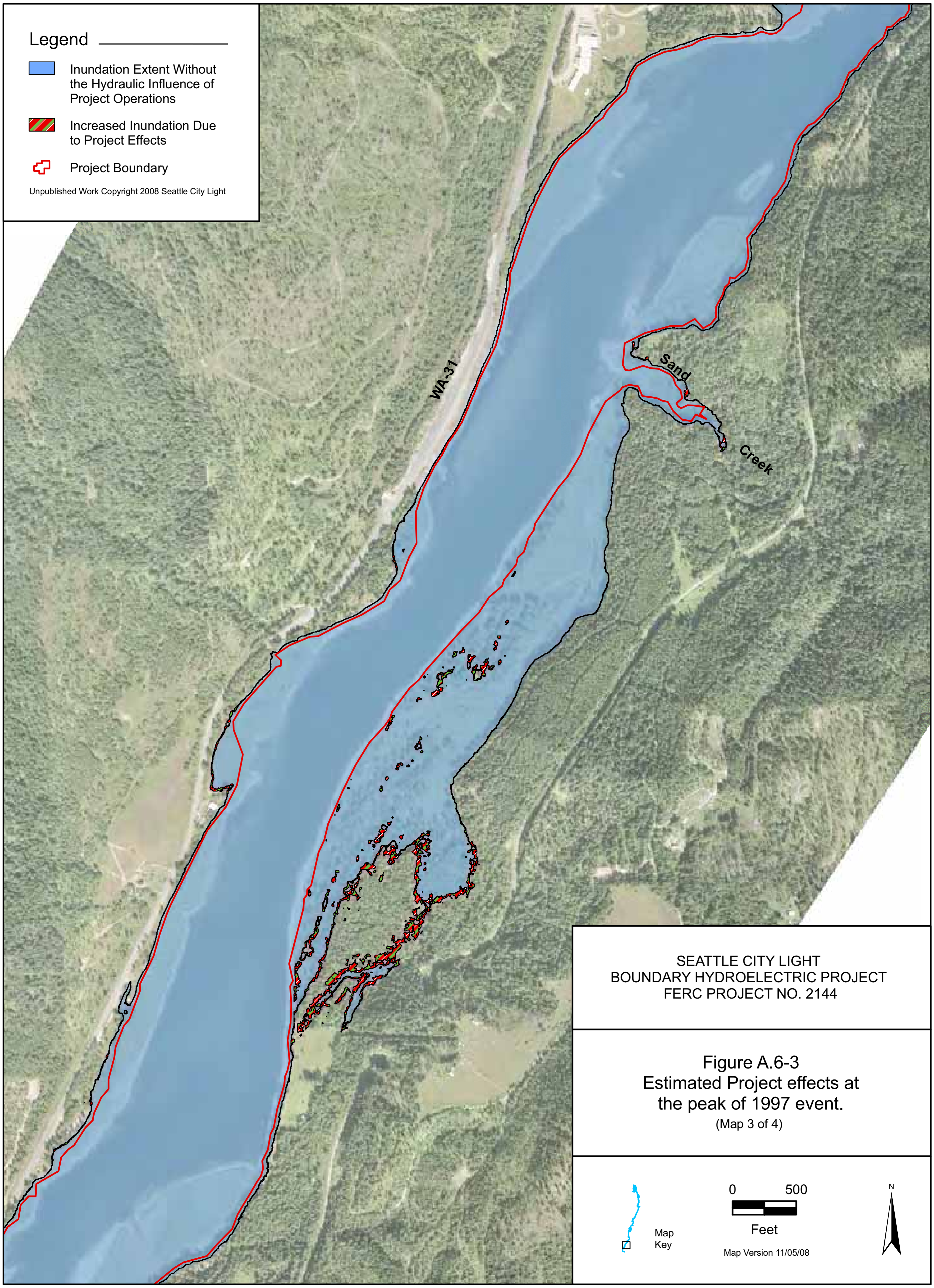
FERC PROJECT NO. 2144

Figure A.6-2
Estimated Project effects at the peak of 1974 event.
(Map 4 of 4)

Map Version 11/05/08


Legend
Inundation Extent Without the Hydraulic Influence of Project Operations
Increased Inundation Due to Project Effects
Project Boundary
Unpublished Work Copyright 2008 Seattle City Light
Sullivan Creek
C2975
WA-31
Metaline Falls
WA-31
Linton Creek
Metaline
SEATTLE CITY LIGHT
BOUNDARY HYDROELECTRIC PROJECT
FERC PROJECT NO. 2144
Figure A.6-3
Estimated Project effects at the peak of 1997 event.
(Map 1 of 4)
Map Key
0
500
Feet
Map Version 11/05/08
N


Legend
Inundation Extent Without the Hydraulic Influence of Project Operations
Increased Inundation Due to Project Effects
Project Boundary
Unpublished Work Copyright 2008 Seattle City Light
Pocahontas
Creek
WA-31
Wolf
Creek
Sweet
Creek
SEATTLE CITY LIGHT
BOUNDARY HYDROELECTRIC PROJECT
FERC PROJECT NO. 2144
Figure A.6-3
Estimated Project effects at the peak of 1997 event.
(Map 2 of 4)
Map Key
0
500
Feet
Map Version 11/05/08
N


Legend
Inundation Extent Without the Hydraulic Influence of Project Operations
Increased Inundation Due to Project Effects
Project Boundary
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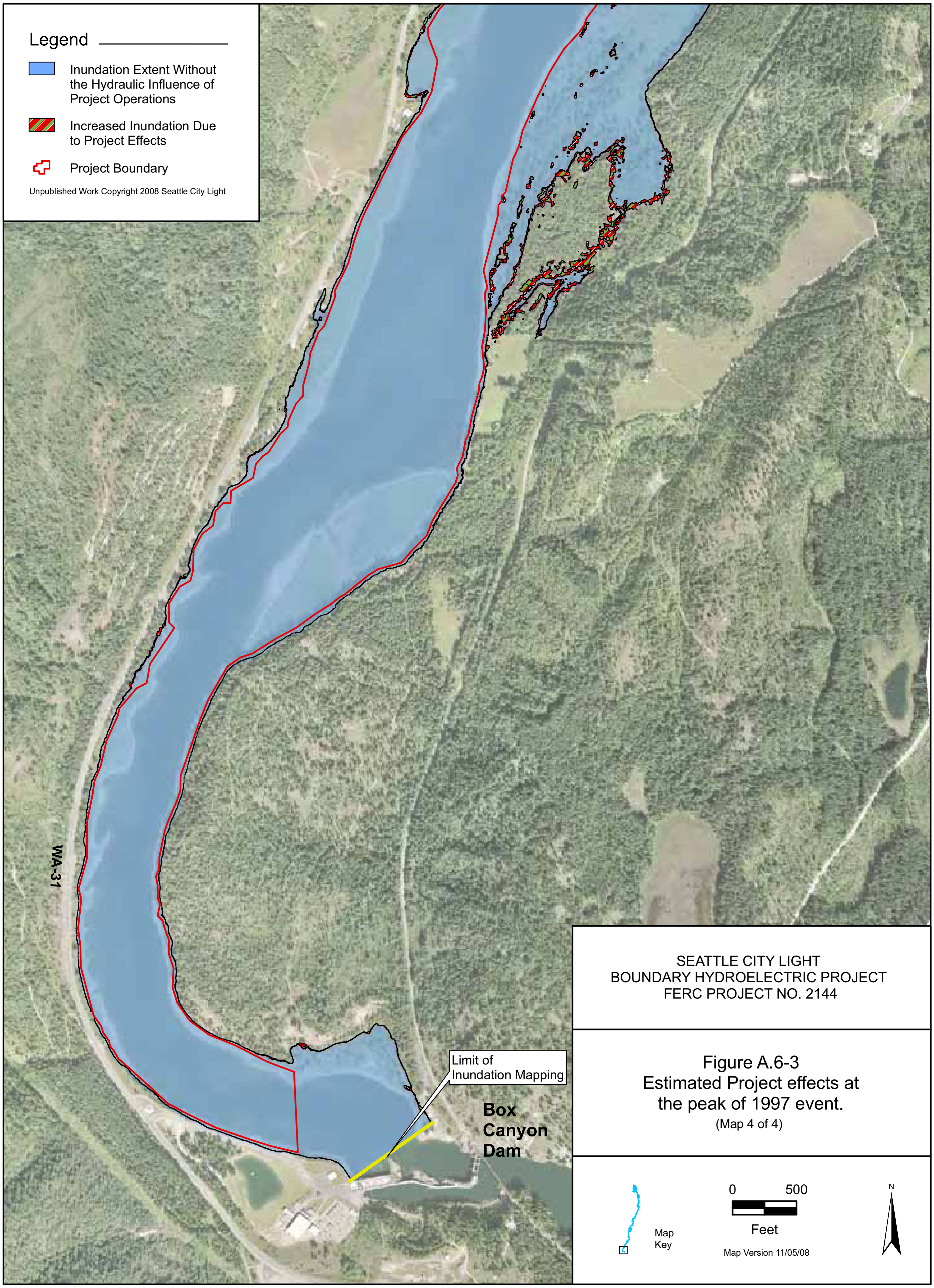
Sand
Creek
SEATTLE CITY LIGHT
BOUNDARY HYDROELECTRIC PROJECT
FERC PROJECT NO. 2144
Figure A.6-3
Estimated Project effects at the peak of 1997 event.
(Map 3 of 4)
Map Key
0
500
Feet
Map Version 11/05/08
N



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FERC PROJECT NO. 2144

Figure A.6-3
Estimated Project effects at the peak of 1997 event.
(Map 4 of 4)