INL/EXT-10-20572

Idaho National Laboratory Materials and Fuels Complex Natural Phenomena Hazards Flood Assessment

Gerald Sehlke
Paul Wichlacz

December 2010

The INL is a U.S. Department of Energy National Laboratory
operated by Battelle Energy Alliance

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Idaho National Laboratory
Idaho Falls, Idaho 83415

http://www.inl.gov

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Approved by

Jennifer Jorge | December 13, 2010
Jennifer Jorge | Date

EXECUTIVE SUMMARY

The purpose of this study is to determine the highest flood elevation (called a Design Basis Flood) and evaluate that flood against Critical Flood Elevations for existing nuclear facilities at the Materials and Fuels Complex (MFC) and the adjacent Transient Reactor Experiment and Test Facility (TREAT) located at the Idaho National Laboratory (INL). The facilities specifically evaluated were: [MFC-704, MFC-752, MFC-765, MFC-767, MFC-771, MCF-774, MFC-775, MFC-776, MFC-784, MFC-785, MFC-786, MFC-787, MFC-792, MFC-792A, MFC-794, MFC-798, MFC-1702, MFC-709, MFC-720, and MFC-723]. This report concludes that all MFC facilities including TREAT (MFC-720 and MFC723), passed the PC-3 (10,000 year recurrence interval) flood. Because of the analytical methodology selected in the preparation of this study, this study can also be used to support the construction of new facilities at MFC.

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ACRONYMS

AEP	Annual Exceedance Probability
AL	Analytical Laboratory
ANL-W	Argonne National Laboratory - West
ASTM	American Society for Testing and Materials
BEA	Battelle Energy Alliance, LLC
BLS	below land surface
CESB	Contaminated Equipment Storage Building
CFA	Central Facilities Area
CFE	Critical Flood Elevation
DBFL	design basis flood
DOE	Department of Energy
DOE-ID	Department of Energy Idaho Operations Office
EBR-II	Experimental Breeder Reactor II
ESRP	Eastern Snake River Plain
ESRPA	Easter Snake River Plain Aquifer
FASB	Fuels and Applied Science Building
FCF	Fuel Conditioning Facility
FEMA	Federal Emergency Management Agency
FIRMs	Flood Insurance Rate Maps
FMF	Fuel Manufacturing Facility
FY	Fiscal Year
GIS	Geographical Information System
GPS	Global Positioning System
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center - River Analysis System
HFEF	Hot Fuel Examination Facility
HUC	Hydrologic Unit Code
IBC	International Building Code
ID	identification
IDEQ	Idaho Department of Environmental Quality
INEEL	Idaho National Engineering and Environmental Laboratory
INL	Idaho National Laboratory
INTEC	Idaho Nuclear Technology and Engineering Center

IWP	Industrial Waste Pond
LiDAR	Light Detection and Ranging
LOFT	Loss-of-Fluid Test Facility
MFC	Materials and Fuels Complex
MLW	mean low water
NOAA	National Oceanic and Atmospheric Administration
NPH	natural phenomena hazard
NRAD	Neutron Radiography Facility
O&M	operations and maintenance
PC	Performance Category
PMF	Probably Maximum Flood
PMP	probably maximum precipitation
PNNL	Pacific Northwest National Laboratory
RCL	Radiochemistry Laboratory
RCRA	Resource Conservation and Recovery Act
RLWTF	Radioactive Liquid Waste Treatment Facility
RSWF	Radioactive Scrap and Waste Facility
SCS	Soil Conservation Service
SMC	Specific Manufacturing Capability
SPRS	space radioactive power source
SSCs	structures, systems, and components
SSPSF	Space and Security Power Systems Facility
STP	Sanitary Sewage Treatment Pond
TAN	Test Area North
TREAT	Transient Reactor Experiment and Test Facility
TREAT-W	Transient Reactor Experiment and Test Facility Warehouse
UH	Unit Hydrograph
USACE	U.S. Army Corps of Engineers
USGS	United States Geological Survey
ZPPR	Zero Power Physics Reactor

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1. INTRODUCTION

Idaho National Laboratory (INL), formerly known as the Idaho National Engineering and Environmental Laboratory (INEEL), is a government-owned reservation located in southeastern Idaho, approximately 40 km (25 mi) west of Idaho Falls, Idaho. The INL Site covers approximately 2,300 km^2 (890 mi^2), extending a maximum of 63 km (39 mi) from north to south and 58 km (36 mi) from east to west. There are nine major operational areas at the INL Site in addition to a number of miscellaneous facilities (Figure 1). Additional support and administrative facilities are located in Idaho Falls.

INL is owned by the U.S. Department of Energy (DOE), and most of INL is managed by the DOE Idaho Operations Office (DOE-ID). Battelle Energy Alliance (BEA), LLC, is the primary Site management and operations contractor; hence, it has primary responsibility for protecting INL facilities and infrastructure from natural phenomena hazards (NPHs).

1.1 Purpose & Scope

This report assesses the potential for flooding at the Materials and Fuels Complex (MFC) and the Transient Reactor Experiment and Test (TREAT) Facility. Chapter IV of DOE O 420.1B (Natural Phenomena Hazards Mitigation) establishes DOE's requirements for facility design, construction, and operations in order to protect the public, workers, and the environment from the impact of all NPH events (e.g., earthquake, wind, flood, and lightning). These requirements are applicable to all DOE facilities and sites. DOE facilities and operations are required to ensure that structures, systems, and components (SSCs) and personnel will be able to perform their intended safety functions effectively under the effects of an NPH. DOE O 420.1B has five supporting standards for implementing the NPH requirements at DOE facilities:

- DOE-STD-1020-2002, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*
- DOE-STD-1021-93, *Natural Phenomena Hazards Performance Categorization Criteria for Structures, Systems, and Components*
- DOE-STD-1022-94, *Natural Phenomena Hazards Site Characterization Criteria*
- DOE-STD-1023-95, *Natural Phenomena Hazards Assessment Criteria*
- DOE-STD-1024-92, *Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites*.

DOE Standards 1020, 1022, and 1023 are appropriate for and were utilized in this assessment.

MFC contains facilities that may be impacted by natural phenomena hazards, including flooding. Therefore, based on the requirements of DOE Order 420.1B, BEA conducted a comprehensive NPH flood assessment to determine the potential for regional and local flood events to inundate nuclear and nuclear-related facilities (SSCs) at MFC and TREAT.

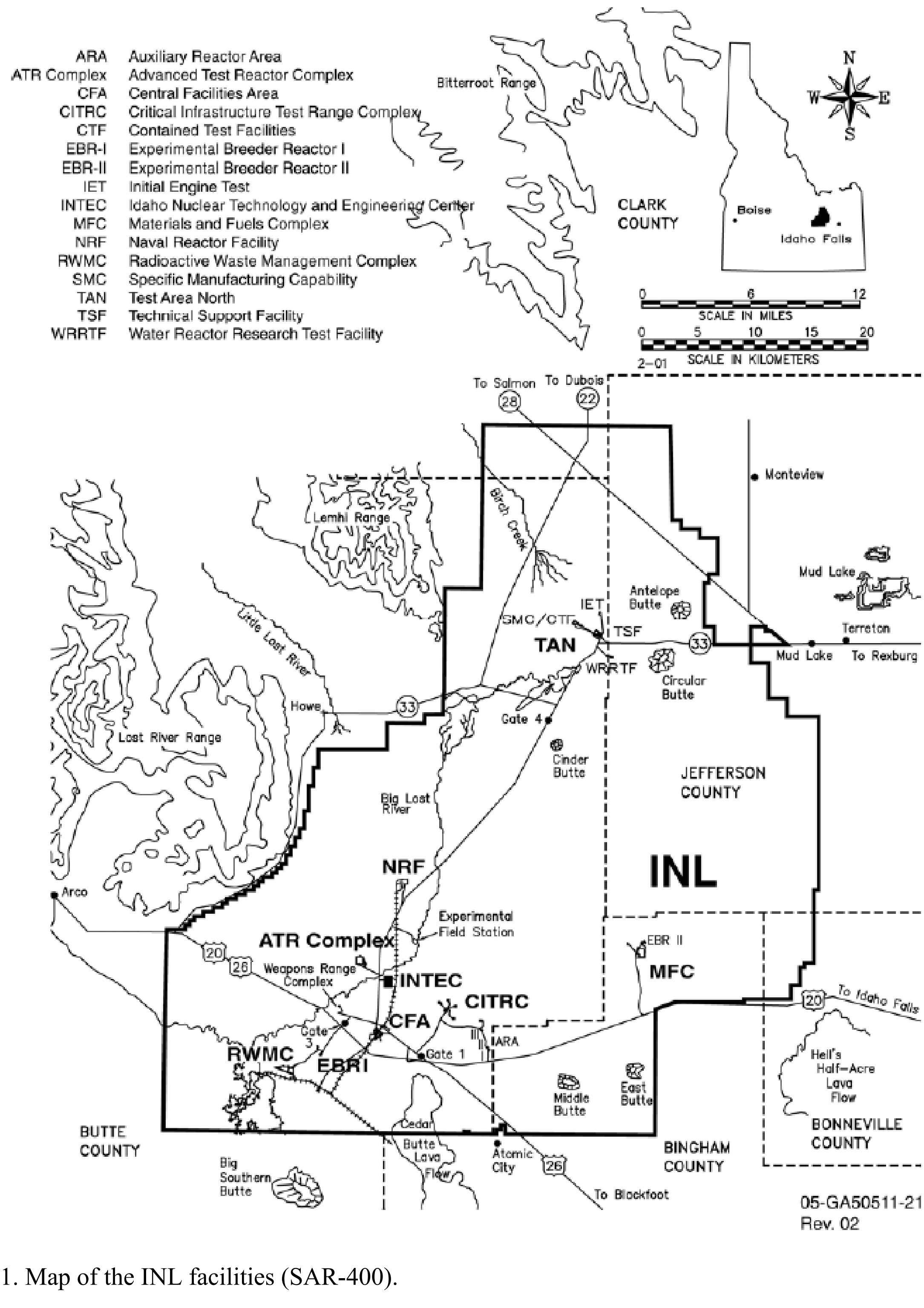

Figure 1. Map of the INL facilities (SAR-400).

1.2 Assessment Criteria

Based on the criteria in DOE-STD-1020-2002, NPH analyses for multiple SSCs are required to be conducted to meet the highest performance category being assessed. MFC contains Performance Category (PC) 0, 1, 2, and 3 SSCs and Transient Reactor Experiment and Test Facility (TREAT) contains PC 0, 1, and 2 facilities. Using a graded approach, MFC and TREAT are analyzed to determine if they meet DOE's flood hazard assessments criteria for a potential probable maximum flood (PMF) and for PC 3 facilities.

According to DOE-STD-1020-2002, PC 3 SSCs are require to conduct a probabilistic flood hazard analysis with a mean hazard annual probability of 1×10^{-4} (10,000-year recurrence) (Table 1), including the potential combinations of floods, as provided in Table 2.

Table 1. Flood criteria summary.[a]

Item	Performance Category			
	1	2	**3**	4
Flood Hazard Input	Flood insurance studies or equivalent input, including the combinations in Table 4-2	Site probabilistic hazard analysis, including the combinations in Table 4-2	Site probabilistic hazard analysis, including the combinations in Table 4-2	Site probabilistic hazard analysis, including the combinations in Table 4-2
Mean Hazard Annual Probability	2×10^{-3}	5×10^{-4}	1×10^{-4}	1×10^{-5}
Design Requirements	Applicable criteria (e.g., governing local regulations, IBC 2000) shall be used for building design for flood loads (e.g., load factors, design allowables), roof design, and site drainage. The design of flood mitigation systems (levees, dams, etc.) shall comply with applicable standards as referred to in these criteria.			
Emergency Operation Plans	Required to evacuate onsite personnel if facility is impacted by the design basis flood (DBFL)	Required to evacuate onsite personnel and to secure vulnerable areas if site is impacted by the DBFL	Required to evacuate onsite personnel not involved in essential operations. Provide for an extended stay for personnel who remain onsite. Procedures must be established to secure the facility during the flood such that operations may continue following the event.	
a. Table 4-1, DOE-STD-1020-2002				

Table 2. Design basis flood events.[a]

Primary Hazard	Case No.	Event Combinations[b]
River Flooding	1	Peak flood elevation. Note that the hazard analysis for river flooding should include all contributors to flooding, including releases from upstream dams, ice jams, etc. Flooding associated with upstream-dam failure is included in the dam failure category.
	2	Wind-waves corresponding, as a minimum to the 2-year wind acting in the most favorable direction (Ref. 4-2), coincident with the peak flood or as determined in a probabilistic analysis that considers the joint occurrence of river flooding and wind generated waves.
	3	Ice forces (Refs. 4-2 and 4-3) and Case 1.
	4	Evaluate the potential for erosion, debris, etc., due to the primary hazard.
Dam Failure	1	All modes of dam failure must be considered (overtopping, seismically induced failure, random structural failures, upstream dam failure, etc.)
	2	Wind-waves corresponding, as a minimum to the 2-year wind acting in the most favorable direction (Ref. 4-2), coincident with the peak floor or as determined in a probabilistic analysis that considers the joint occurrence of river flooding and wind generated waves.
	3	Evaluate the potential for erosion, debris, etc., due to the primary hazard.
Local Precipitation	1	Flood based on the site runoff analysis shall be used to evaluate the site drainage system and flood loads on individual facilities.
	2	Ponding on roof to a maximum depth corresponding to the level of the secondary drainage system.
	3	Rain and snow, as specified in applicable regulations.
Storm Surge or Seiche (due to hurricane, seiche, squall lines, etc.)	1	Tide effects corresponding to the mean high tide above the MLW[c] level (if not included in the hazard analysis).
	2	Wave action and Case 1. Wave action should include static and dynamic effects and potential for erosion (Ref. 4-2).
Levee or Dike Failure	1	Should be evaluated as part of the hazard analysis if overtopping and/or failure occur.
Snow	1	Snow and drift roof loads as specified in applicable regulations.
Tsunami	1	Tide effects corresponding to the mean high tide above the MLW[c] level (if not included in the hazard analysis).

a. Table 4-2, DOE-STD-1020-2002

b. Events are added to the flood level produced by the primary hazard.

c. MLW - Mean Low Water.

DOE-STD-1020-2002 states that an evaluation of the flood design basis for new SSCs consists of:

- Determining the DBFL for each flood hazard as defined by the hazard annual probability of exceedance and applicable combinations of flood hazards
- Evaluating the site storm water management system (e.g., site runoff and drainage, roof drainage)
- Developing a flood design strategy for the DBFL that satisfies the criteria performance goals (e.g., build above the DBFL, harden the facility)
- Designing the civil engineering systems (buildings, buried structures, site drainage, retaining walls, dike slopes, etc.) to the applicable DBFL and design requirements.

However, all facilities assessed at MFC and TREAT are existing facilities. DOE-STD-1020-2002 states that the performance criteria for existing facilities are comparisons of the CFE versus the DBFL. Therefore, this study was designed to address the first criteria above and provide context and information for addressing the following three criteria. It is focused on providing the hydrologic inputs and conducting the analyses necessary for estimating the DBFL and determining whether the DBFL could potentially inundate existing nuclear facilities (SSCs) at MFC and TREAT. DOE-STD-1020-2002 recognizes that existing SSCs may not be situated above the DBFL; therefore, it states that SSCs should be assessed to determine the level of flooding, if any, that can be sustained without exceeding the performance requirements. Furthermore, it states that if the Critical Flood Elevation (CFE) is higher than the DBFL, then the performance goals are satisfied. If the CFE is lower than the DBFL, it is presumed that such flooding will cause damage or disruptions such that the performance goal is not satisfied.

1.3 Assessment Overview

BEA Environmental Stewardship and Water Management assessed and/or provided the necessary data and information for the facility description, data characteristics, flood history, and site hydrology information relative to assessing the potential for flooding due to the potential flooding conditions described in Table 2. Pacific Northwest National Laboratory (PNNL) utilized Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) and Hydrologic Engineering Center– River Assessment System (HEC-RAS) software to convert estimated precipitation into watershed flows, to route those flows through MFC, and to assess the DBFL relative to the CFE at each of the facilities assessed at MFC and TREAT. The results of these assessments are summarized below and discussed in detail in Appendix A.

2. PREVIOUS FLOOD INVESTIGATIONS

Historically, approximately 70 flood or flood-related studies have been conducted at the INL Site. The vast majority of these studies were associated with the Big Lost River and INL facilities located in the Big Lost River Drainage. Several studies have been conducted in the Birch Creek drainage. No assessments are known to have been conducted for assessing potential flooding in the Little Lost River drainage.

Since no perennial surface water features exist within the MFC drainage, few formal flood analyses have been conducted within the MFC watershed. Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRMs) were developed for Bingham County where MFC and TREAT are located. The FIRMs designate MFC and TREAT as being in Zone C (DOE-ID 1995). INL related studies include one to determine if flooding from the Big Lost River could impact MFC (Van Haaften et. al. 1984) and another geotechnical engineering study to assess the MFC flood diversion dam (NTL 1979). A third study was not associated with flooding per se; however, it provides information relative to evaporation and infiltration at MFC, which is useful for assessing potential floods at MFC (ANL 1999).

3. FACILITY DESCRIPTION

MFC (formerly Argonne National Laboratory – West [ANL-W]) is located in the southeastern corner of the INL Site, approximately 35 miles west of Idaho Falls, Idaho and 16 miles east of the Central Facilities Area. The MFC operational area encompasses approximately 60 acres (Figure 2). It was established for reactor research in the mid 1950s and has primarily served as a reactor testing facility. The facility was operated by the University of Chicago for the DOE Chicago Operations Office from the INL Site's inception as the National Reactor Test Station until it was integrated into the DOE-ID's primary site management and operations contract in 2005.

Figure 2. Aerial photograph of MFC.

3.1 Major Operational Areas at MFC

MFC has been redirected to new missions that are primarily devoted to research and development of nuclear technologies, nuclear environmental management, and space radioactive power source (SPRS) development. In addition, MFC is currently INL's center of conducting research and development related to new reactor fuels and related materials, and nuclear nonproliferation methods and technologies. Currently, there are 55 buildings at MFC. Key MFC nuclear or nuclear-related facilities include:

- Fuel Manufacturing Facility (FMF, MFC-704). FMF was originally constructed for manufacturing fuels for EBR-II. Its current mission is fuel processing, research and development on new fuel fabrication methods and storage of nuclear materials.
- Fuel Conditioning Facility (FCF, MFC-709). FCF consists of two hot cells for working with nuclear materials and handling, storage, and assembling and/or disassembling radioactive components. In addition to the hot cells, FCF has an equipment mockup area and areas where contaminated equipment can be decontaminated and repaired.
- Transient Reactor Test Facility (TREAT, MFC-720–724). TREAT contains an air-cooled UO_2-graphite-fueled reactor operated to produce high-power transients of very short duration for reactor safety tests. A permit modification request to allow storage and treatment of mixed waste was approved by the Idaho Department of Environmental Quality (IDEQ) in Fiscal Year (FY) 2002.
- Analytical Laboratory (AL, MFC-752). AL houses shielded hot cells, glove boxes, casting laboratories and facilities used for nuclear fuels and materials characterization, environmental sampling and analysis, and conducting other examinations.
- Experimental Breeder Reactor-II (MFC-767 and -768). EBR-II was a sodium-cooled reactor operated as a fuel and material irradiation facility, which was shut down on September 30, 1994. Defueling of the EBR-II reactor was completed, and the facility was placed in a radiologically and industrially safe configuration. Portions of these facilities have been closed in accordance with RCRA permit closure plans and the facility is currently undergoing decontamination and decommissioning (D&D). These buildings have been received by the Idaho Cleanup Project and its principal contractors.
- Radioactive Scrap and Waste Facility (RSWF, MFC-771). RSWF is a fenced area of about 4 acres located 1/2 mile north of the MFC security perimeter. RSWF consists of 1350 liners, approximately 13 ft long, buried vertically in the ground. It is an interim storage area for radioactive scrap and waste and mixed waste.
- ZPPR Facilities (MFC-774–776, 784, and 792). The ZPPR Building (MFC-776) contained a large air-cooled fast-reactor critical assembly (a reactor core model) used to study reactor physics. The reactor was removed early in 2010. The ZPPR Building currently houses a glovebox for gas generation experiments. The ZPPR Vault and Work Room (MFC-775) is utilized to store nuclear material used at MFC. Directly south of MFC-775 is the Materials Control Building (MFC-784) formerly used to store sodium and lithium hydride used in ZPPR experiments. This building also houses a cleaning room with solvent tanks. Directly south of MFC-784 is the Mock-up Building (MFC-792) formerly used to store nuclear materials, and other ZPPR equipment, it has been modified for use as the SSPSF (see below). Adjacent to MFC-775 is the Support Wing (MFC-774), which houses the Electron Microscopy Laboratory, tool-cleaning facilities, and offices.
- Hot Fuel Examination Facility (HFEF, MFC-785). HFEF contains a two hot-cell complex and radiation-shielded rooms for handling irradiated reactor fuel and structural materials. HFEF was used to examination the performance of fuels and materials irradiated in EBR-II and is now used to examination fuels and materials from other facilities and for conducting transuranic (TRU) waste characterizations in support of the Waste Isolation Pilot Plant (WIPP) in New Mexico.

- Neutron Radiography Reactor (NRAD, MFC-785) Facility. NRAD, is a 250 kW TRIGA reactor housed in the basement underneath the HFEF hot cells. It is currently the only operational reactor at MFC. NRAD is used for conducting neutron radiography irradiation of small test components.
- Space and Security Power Systems Facility (SSPSF, MFC-792). SSPSF is designed for assembling and testing of radioisotope power sources ("batteries") to provide electrical power for use in space applications or remote terrestrial locations.
- Contaminated Equipment Storage Building (CESB, MFC-794). CESB is a pre-engineered metal building consisting of two large rooms for the storage of radioactively contaminated equipment that may be needed in the future, and for repackaging radioactive materials and waste for disposal.
- MFC operates 9 Hazardous Waste Management Act/RCRA-permitted hazardous waste management units in the following facilities: Contaminated Equipment Storage Building (CESB, MFC-794), EBR-II, Outside Radioactive Storage Area (ORSA, MFC-797), Radioactive Scrap and Waste Facility (RSWF, MFC-771), Sodium Process Facility (SPF, MFC-799), Sodium Components Maintenance Shop (SCMP, MFC-793), Sodium Processing Facility (SPF, MFC-799), and Sodium Storage Building (SSB, MFC-703), and TREAT (MFC-720).

Several new facilities have been proposed (e.g., a remote-handled waste treatment facility), others (e.g., MFC-774 and MFC-710) are being upgraded. Fifteen buildings have been identified as candidates for footprint reduction (e.g., Sodium Process Facility and Sodium Components Maintenance Shop) or are in the process of being decontaminated and decommissioned (e.g., EBR-II reactor and Zero Power Physics Reactor) (BEA 2005). MFC will continue to use all other support facilities for current work scope. A list of the facilities assessed in this study is provided in Table 3 and show in Figures 3 and 4.

Table 3. Facilities assessed at MFC and TREAT.

Building No.	Facility Name	Performance Category
MFC-704	Fuel Manufacture Facility (FMF)	3
MFC-720	Transient Reactor Experiment and Test Facility (TREAT) Reactor Bldg.	2
MFC-723	Transient Reactor Experiment and Test Facility (TREAT) Warehouse	2
MFC-752	Analytical Laboratory (AL) and Office Building	2
MFC-765	Fuel Conditioning Facility (FCF)	3
MFC-767	Experimental Breeder Reactor II (EBR-II) Reactor Plant Building	*
MFC-771	Radioactive Scrap Waste Facility (RSWF)	2
MFC-774	Zero Power Physics Reactor (ZPPR) Support Wing	3
MFC-775	Zero Power Physics Reactor (ZPPR) Vault-Workroom Equipment Room	3
MFC-776	Zero Power Physics Reactor (ZPPR) Reactor Cell	3
MFC-784	Zero Power Physics Reactor (ZPPR) Materials Control Building	2
MFC-785	Hot Fuel Examination Facility (HFEF)/ Neutron Radiation (NRAD) Facility	2
MFC-786	Hot Fuel Examination Facility (HFEF) Substation	*
MFC-787	Fuels and Applied Science Building (FASB)	*
MFC-792	Space & Security Power System Facility (SSPSF) Control Room	3
MFC-792A	Space & Security Power System Facility (SSPSF) Annex	3
MFC-794	Contaminated Equipment Storage Building (CESB)	2
MFC-798	Radioactive Liquid Waste Treatment Facility (RLWTF)	*
MFC-1702	Radiochemistry Laboratory (RCL)	2
* This facility's Performance Category has not yet been designated per DOE-STD-1020.		

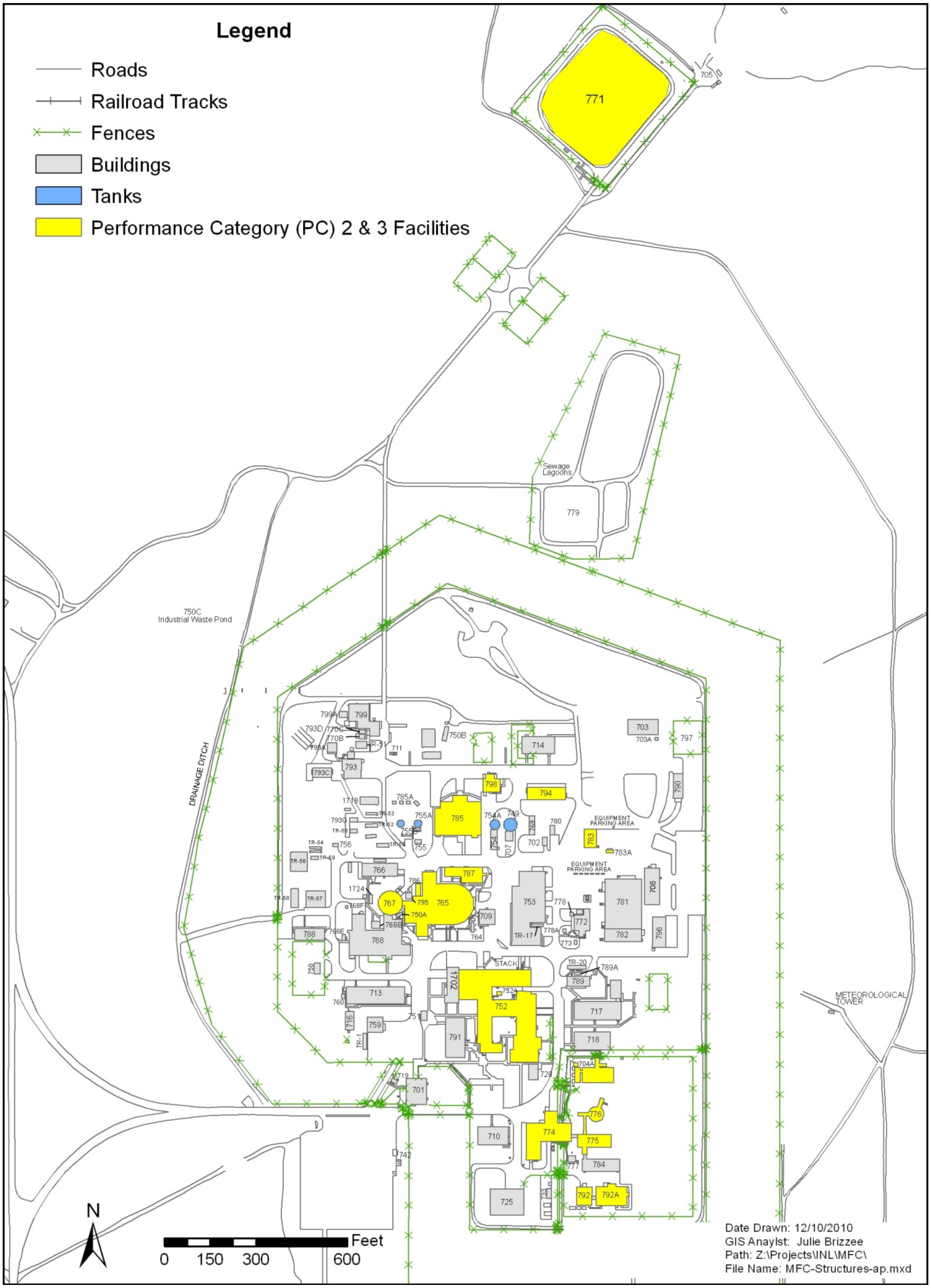

Figure 3. Map of facilities at MFC. Highlighted facilities are the facilities assessed for potential flooding.

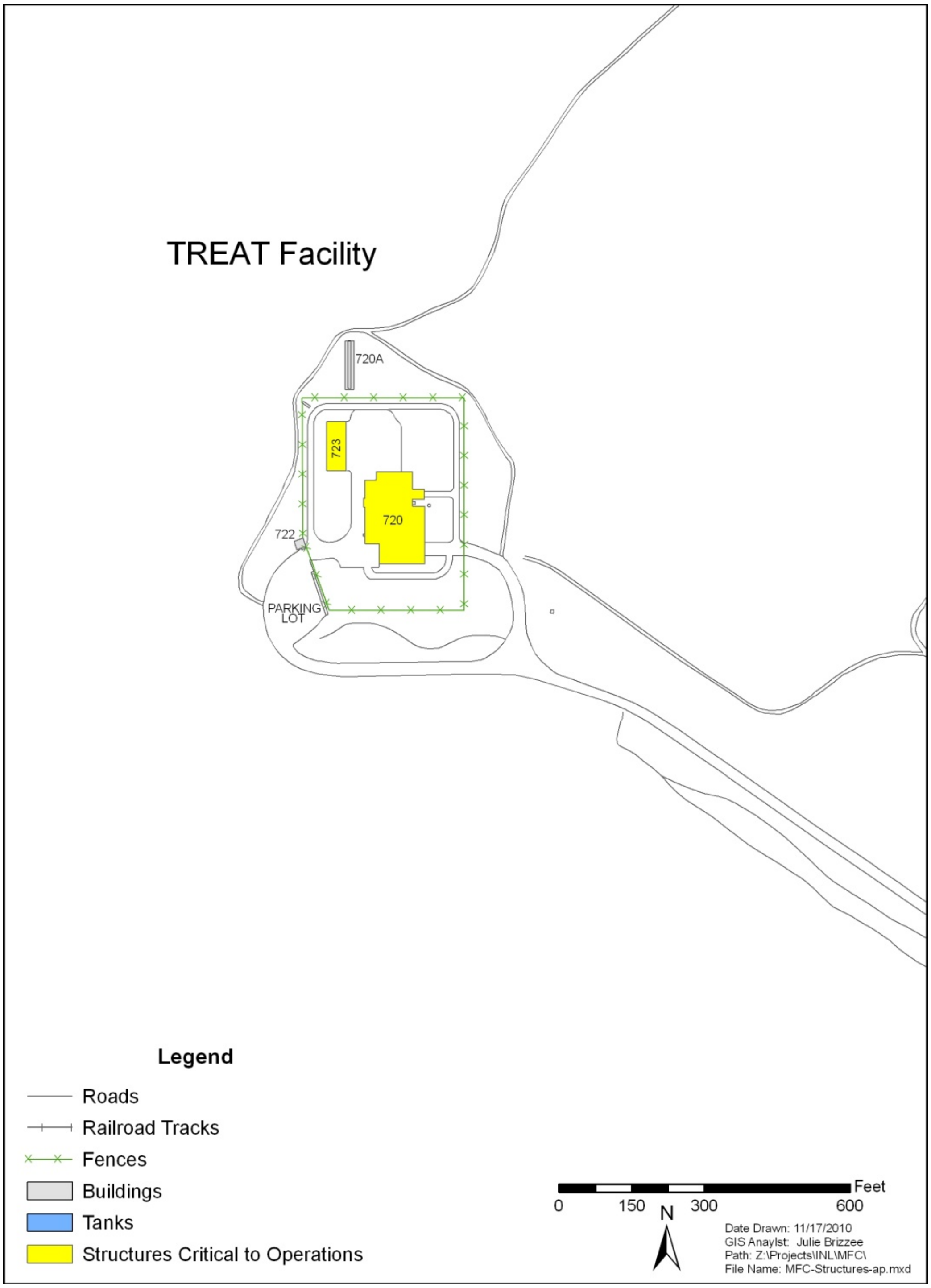

Figure 4. Map of facilities at TREAT. Highlighted facilities are the facilities assessed for potential flooding.

3.2 MFC Flood Control System

Because of historical flooding at MFC, a flood control system was constructed starting in about 1963 and it has been upgraded over time as necessary. Drainage ditches and culverts, an Interceptor Canal, a diversion dam, and the IWP were either constructed or modified to help control and collect water from intermittent surface water runoff events (Figure 5). The focus of the flood control system is to divert water around MFC (e.g., the MFC Diversion Dam) and to drain floodwater from the operational area into the IWP. The system is discussed briefly in DOE-ID (2010) and DOE-ID (1998) and summarized below.

The IWP is the main storm-water collection point at MFC. It is an unlined, approximately 1.2-ha (3-acre) evaporative seepage pond fed by the interceptor canal and site drainage ditches. Excess water in the pond will naturally spill into an overflow channel where it is dispersed north of MFC. The pond was excavated in 1959, and enlarged in 1988 to obtain a maximum water depth of about 4 m (13 ft).

The onsite drainage system consists of a combination of open ditches, various sized pipes, and culverts (ANL 1978), as show in Figure 5. East of the line formed by Buildings 774 and 794, drainage flows in a northerly direction into a large open ditch on the north side of the facility and westerly to the IWP. West of the line formed by Buildings 774 and 794, drainage flows westerly into three channels that flow off the site to the west side of the facility into a ditch, which is parallel to the main Interceptor Canal, and then flows into the IWP.

- The Interceptor Canal transports industrial waste, spring runoff, and other natural waters around MFC to the IWP for flood control. During high precipitation and rapid snowmelt events at MFC, the Interceptor Canal and the IWP have received surface water runoff.
- The Main Cooling Tower Blowdown Ditch runs north on the west side of the Main Cooling Tower and then north between the security fences to the IWP. It is an unlined channel approximately 700 ft in length and 3 to 15 ft wide. From 1962 to 1996, the ditch was utilized to convey industrial wastewater from the Cooling Tower to the Industrial Waste Pond.
- MFC auxiliary Cooling Tower Blow Down Ditch (Ditch A) conveys industrial wastewater from the EBR-II power plant and the fire station (MFC-768 and MFC-759) and storm water runoff into the Main Cooling Tower Blowdown Ditch and ultimately into the IWP.
- Ditch B was also used to convey storm water runoff and wastewater from the EBR-II power plant and the fire station (MFC-768 and MFC-759) to the IWP. However, only a small 125-ft portion of Ditch B is still being used today since the majority of Ditch B, 1,275 ft, was backfilled with clean soil to grade during the installation of a secondary security fence.
- Ditch C was created in 1978 when a portion of Ditch B was backfilled. The discharge water going to Ditch B was rerouted via a culvert under the security fence to Ditch C, which drains to the Main Cooling Tower Blowdown Ditch and ultimately the Industrial Waste Pond. Ditch C's dimensions are approximately 5 × 500 × 2.5 ft deep.
- The industrial waste lift station discharge ditch (the "north ditch"), is located inside the MFC security fences. The ditch is approximately 500 ft long and 3 to 4 ft wide at the bottom. The ditch receives industrial wastewater from various MFC facilities and from storm water runoff.

The MFC Diversion Dam was constructed in 1968 in response to a 1963 flooding event. It is a low earth-fill dam constructed of materials from the immediate vicinity, primarily sandy silt (used for earth fill) and basalt rock (used for riprap) (Portage 2010) located 1/2 mile south of MFC. The diversion dam is approximately 700 ft long and nearly 11 ft high at the center (above the natural runoff channel) and the crest is approximately 18 ft wide. The height diminishes when moving away from the center and toward

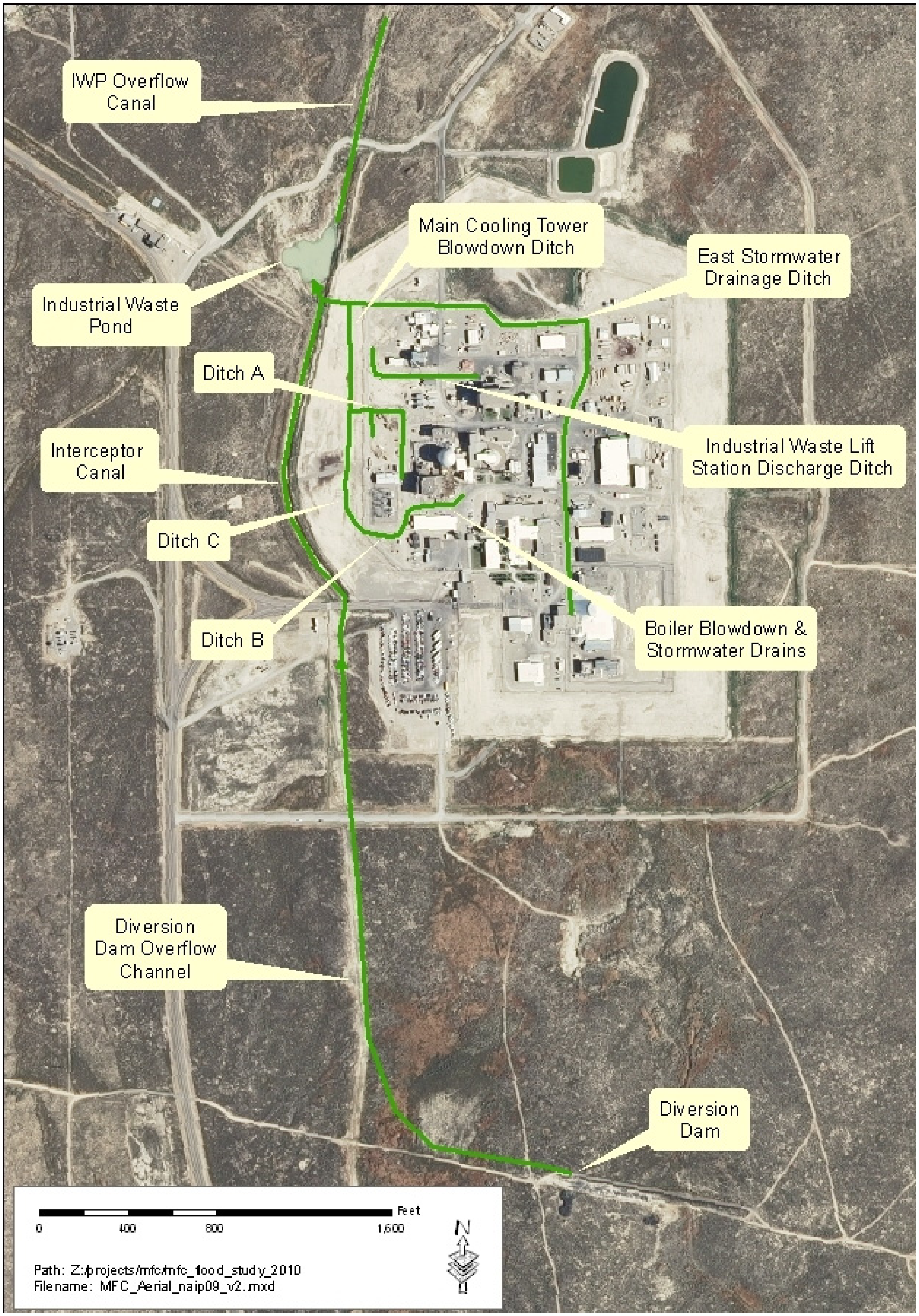

Figure 5. MFC facility storm-water drainage system.

the ends, such that much of the diversion dam resembles a berm or levee. One 36-in. corrugated metal outlet pipe penetrates the bottom center of the structure and is regulated by a single head gate operated from the crest. The head gate is typically kept open, allowing minor flows through the culvert. However, during potential flood events, the head gate can be closed and excess water is diverted over a 40-ft-wide spillway into an adjacent drainage channel, which flows north from the spillway along the west side of the parking lot and into the Interceptor Canal. From Interceptor Canal, water flows directly into the IWP located northwest of all major MFC facilities. If flood flows exceed the capacity of the IWP, water overflows through a channel to open rangeland north of MFC.

3.3 Natural Resources

The INL Site lies along the eastern edge of the Snake River Plain in southeastern Idaho, at the foot of the Lost River, Lemhi, and Bitterroot mountain ranges. The Snake River Plain, which extends across southern Idaho, is a broad, low-relief, sagebrush-covered basin floored with basaltic lava flows and terrestrial sediments, contrasting sharply with mountainous terrain to the north and south. The Snake River Plain is approximately 80 to 97 km (50 to 60 mi) wide and 600 km (375 mi) long, extending in a broad arc from the Yellowstone Plateau in the east to the Idaho-Oregon border in the west. Surface elevations on the Snake River Plain gradually decrease from over 1,980 m (6,500 ft) near the Yellowstone Plateau to approximately 640 m (2,100 ft) near the Idaho-Oregon border.

The Eastern Snake River Plain (ESRP) covers approximately 27,972 km^2 (10,800 mi^2) of southern Idaho. The land surface contains little topographic relief, except for a number of buttes and volcanic scablands. Overall, the surface of the area slopes from Ashton, at approximately 1,603 m (5,259 ft), downward to the west to approximately 975 m (3,200 ft), where the ESRP and Western Snake River Plain meet near King Hill (Figure 6).

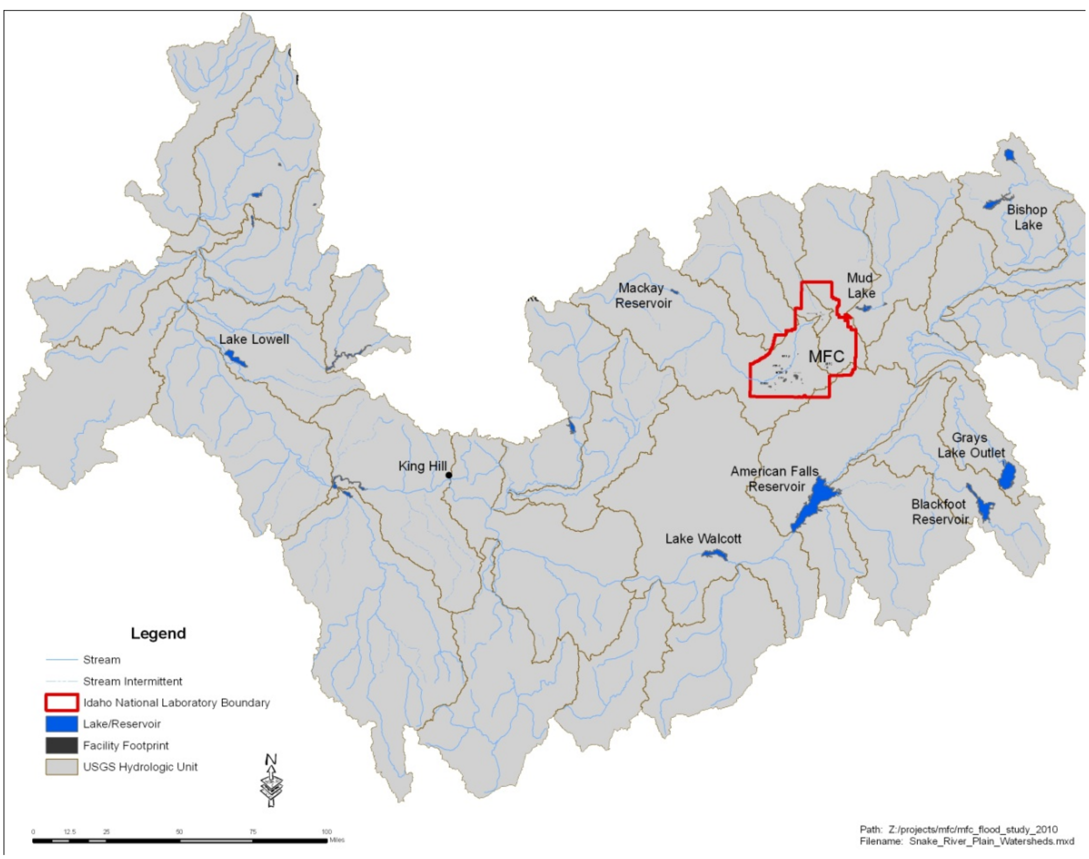

Figure 6. Upper Snake River Basin (USBR) surface water resources.

3.3.1 Climate

INL climatology has been documented by Clawson, Start, and Ricks (1989), and is summarized below. The climate of INL is affected by the surrounding mountains and the Site's location in the ESRP. Because of the northeast-southwest orientation of the ESRP between the surrounding mountains, the prevailing wind is southwesterly. The Centennial and Bitterroot mountain ranges to the north act as a barrier to movement of most of the cold winter air masses passing to the south from Canada. Air masses entering the INL Site are relatively dry because heavy precipitation has most likely occurred while crossing nearby mountain barriers. Therefore, annual rainfall is light, cloud cover is sparse, and the air is relatively dry.

The U.S. Weather Bureau established a weather station at the INL Site in 1949. National Oceanic and Atmospheric Administration (NOAA) meteorologists and technicians are responsible for collecting meteorological observations hourly and daily and for furnishing forecast and emergency support to INL. The information provided below was obtained from Clawson, Start, and Ricks (1989) and updated by Clawson (2010). Wind direction and speed, air temperature, and precipitation have been recorded at CFA since 1949. Weather conditions have also been recorded on a smaller scale at the Test Area North (TAN) and MFC. In addition to recording day-to-day weather data and providing daily operational forecasts for the INL Site, the NOAA staff maintains a research and development program. The program purpose is to improve the reliability of prediction and measurement of meteorological parameters at the INL Site that influence safe conduct of operations. In addition, the NOAA meteorological observation program researches transport, diffusion, and deposition of airborne effluents at the INL Site.

Continuous measurements are made from 35 weather stations in and around the INL Site boundaries. The meteorological stations simultaneously measure the spatial variation of several meteorological parameters, such as air temperature, wind speed, and wind direction. Three of the 35 stations are "tall towers" that range in height from 42 m (150 ft) to 76 m (250 ft). At these towers, winds and temperatures are measured at 10 m (33 ft) and at the tower-top. The remaining stations measure winds and temperatures at 15 m (50 ft). All towers have air temperature and relative humidity sensors at 2 m (6 ft). Telemetered wind measurements, usually at 15 m (50 ft) above ground, are also collected at those stations.

The following sections summarize wind, temperature, and precipitation data obtained from a compilation of INL climate observations made from 1949 through 1988 (Clawson et al. 1989) and updated through December 2006 by Clawson (personal communication, August 2010).

3.3.1.1 Wind

Wind speed and direction have been continuously monitored at a large number of stations on and surrounding the INL Site since 1950. Winds typically blow from the southwest, moving up the ESRP. Winds from the northeast also are common, especially at night when movement of cool air back down the ESRP reverses the daytime flows. The wind directions in the TAN area at (formally at Loss-of-Fluid Test Facility [LOFT], currently at Specific Manufacturing Capability [SMC]) are modified by the broad northwest-to-southeast orientation of the Birch Creek valley that channels strong north-northwest winds into the area.

Based on January 1994–December 2006 data, average monthly near-surface (10-m [33-ft] height) wind speeds are highest in the month of April at SMC with speeds of 16.4 km/h (10.2 mph) and during the month of May at MFC and the Grid 3 facility north of Idaho Nuclear Technology and Engineering Center (INTEC) with speeds of 17.7 km/h (11.0 mph) at both locations. The peak (1-sec) wind gusts at Grid3, MFC, and SMC are 131 km/h (81.6 mph), 119 km/h (73.8 mph), and 115 km/h (71.6 mph), respectively.

3.3.1.2 Temperature

During the summer at the INL Site, days are warm and nights are cool. In winter, days and nights are cold. The limited rainfall, relatively dry air, and infrequent low clouds permit intense solar heating of the surface during the day and rapid radiative cooling at night. These factors combine to produce a large daily temperature range near the ground. The Centennial and Bitterroot mountain ranges to the north keep most of the intensely cold, Canadian winter air masses from intruding into the ESRP. Occasionally, cold air spills over the mountains, producing low temperatures at the INL Site for periods lasting a week or longer.

The longest continuous air temperature record for the INL Site is 56 years at the Central Facilities Area (CFA). Temperatures have been measured from January 1950 through the present, published through December 1988, and updated to 2008 per Clawson (2010). The annual average temperature is 5.7°C (42.3°F), with recorded extremes of -43.9°C (-47°F) and 40.6°C (105°F). Average daily temperatures range from a low of -11°C (13°F) in early January to a high of 21°C (70°F) on several days in late July.

The average frost-free period at CFA is 88 days, and the shortest recorded frost-free period is 40 days. An average of 42% of the days contains a freeze/thaw cycle. Based on data from November 1956 to August 1963, the average maximum depth of freezing temperatures in the soil is approximately 1 m (3 ft). Freezing temperatures have been recorded to a maximum depth of 1.5 m (5 ft).

3.3.1.3 Precipitation

Considerable precipitation variability is characteristic of the INL area. Although the total amount is relatively light, precipitation can be expected in any month of the year. There also have been several months when no precipitation has been recorded. The type of precipitation also varies, depending on the season. The average annual precipitation (water equivalent) between 1950 through 2006 at CFA is 21.6 cm (8.51 in.) (Table 4). The monthly averages for the same periods show a peak in May and June, with 3.09 and 3.00 cm (1.22 and 1.18 in.), respectively. The highest total annual precipitation recorded was 36.6 cm (14.4 in.). Based on data from January 1994 to December 2006, the greatest 1-hr precipitation recorded was 1.80 cm (0.71 in.) in June and the greatest 24-hr precipitation recorded was 4.55 cm (1.79 in.), also in June.

Table 4. Average, highest and lowest total monthly and annual precipitation (water equivalent) and greatest 1-hour and 24-hour precipitation for CFA.[a]

	Annual			Greatest	
	Average (in.)	Highest (in.)	Lowest (in.)	1 Hour (in.)	24 Hour (in.)
January	0.68	2.56	0.00[b]	0.13	1.02
February	0.61	2.40	0.00	0.17	0.49
March	0.62	2.03	0.00	0.21	0.64
April	0.77	2.50	0.00	0.24	0.71
May	1.22	4.42	0.00	0.27	0.77
June	1.18	4.64	0.00	0.71	1.79
July	0.48	2.29	0.00	0.59	0.60
August	0.50	3.27	0.00	0.25	0.43
September	0.64	3.52	0.00	0.37	1.11
October	0.51	1.67	0.00	0.22	1.10
November	0.61	1.74	0.00	0.15	0.44
December	0.71	3.43	0.02	0.16	0.65
ANNUAL	8.51	14.40	4.45		

a. Data period of record spans January 1950 through December 2006.
b. Trace amounts are not considered as precipitation.
Original data from Clawson et al. (1989), updated per Clawson (2010).

The highest monthly average snowfall (measured at CFA; insufficient records are available for TAN) is 15.7 cm (6.2 in.) (Table 5), which occurred during January. For December, the highest average snowfall is 15.5 cm (6.1 in.). The maximum-recorded snowfall during any 24-hr period is 22.9 cm (9.0 in.), which occurred during January. The greatest average monthly snow depth was 63.8 cm (25.1 in.) in February. Snowfall as late as May and as early as September has been recorded at the INL Site.

Historic record for CFA shows that years with precipitation events equal to or greater than 1 in. occurred in 1954, 1963, 1964, 1969, 1979, and 1981 (Table 6).

A statistical analysis of precipitation data from 18 stations in the Upper Snake River Plain indicates 4.3 and 5.6 cm (1.7 and 2.2 in.) for the 25 and 100-yr, 24-hr summertime precipitation events (SAR 400). For wintertime 25 and 100-year precipitation events, the amounts are 2.6 and 3.2 cm (1.0 and 1.3 in.), respectively (INL 2010).

Table 5. Monthly and annual snow fall totals and monthly and daily extreme totals for CFA.[a]

	Average (in.)	Maximum (in.)	Minimum (in.)	Largest Daily Maximum (in.)
January	6.2	18.1	0.0	9.0
February	4.7	16.1	0.0	7.5
March	3.0	10.2	0.0	8.6
April	1.9	16.5	0.0	6.7
May	0.5	8.3	0.0	4.4
June	0.0	0.0	0.0	0.0
July	0.0	0.0	0.0	0.0
August	0.0	0.0	0.0	0.0
September	0.0	1.0	0.0	1.0
October	0.5	7.2	0.0	4.5
November	3.1	12.3	0.0	6.5
December	6.1	22.3	0.0	8.0
ANNUAL	25.9	59.7	6.8	9.0

a. Data period of record spans January 1950 through December 2006.
Original data from Clawson et al. (1989), updated per Clawson (2010).

Table 6. Occurrences of precipitation amounts equal to or greater than 1 in. per day for CFA.[a]

	Year of Occurrence	Amount (in.)
January	None	None
February	None	None
March	None	None
April	1981	1.51
May	None	None
June	1954	1.36
	1963	1.14
	1969	1.64
	1995	1.55
July	1979	1.25
August	None	None
September	1961	1.09
	1961	1.55
	1994	1.10
October	None	None
November	None	None
December	1964	1.07

a. Data period of record spans January 1950 through December 2006. Original data from Clawson et al. (1989), updated per Clawson (2010).

3.3.2 Watershed Topography

Using the United States Geological Survey's (USGS's) surface water Hydrologic Unit Code (HUC) classification scheme, INL is located in the Upper Snake River Basin Accounting Unit (170402). Portions of six "watersheds" (USGS 8-digit "Cataloging Units") either drain surface water to or from the INL Site. These units include American Falls (17040206), Big Lost (17040218), Birch Creek (17040216), Idaho Falls (17040201), Little Lost (17040217), and Medicine Lodge (17040215) (Figure 7). The Big Lost River, Birch Creek, Little Lost River, and Medicine Lodge are all part of the northern "closed basins." Each of these streams flows near or onto the INL Site and terminates on or near the northern portion of the INL boundary by a combination of infiltration and evapotranspiration. The southeastern portion of the INL Site is part of the American Falls hydrologic unit. No perennial surface water bodies are present in this unit near INL. However, given the appropriate conditions (e.g., sufficient surface water to cause overland flow), regional surface water flow in this unit generally would be away from the INL boundary southward toward the Snake River.

MFC and TREAT are both located at the southern portion of the Medicine Lodge cataloging unit (Figure 8). All MFC facilities are within a single "local" (12-digit HUC) topographically closed watershed and TREAT facilities are located in an adjacent watershed, separated by a low-lying topographic ridge (Figure 9). The MFC watershed contains natural drainage channels, which can concentrate overland flow during periods of high precipitation or heavy spring runoff; however, it contains no perennial, natural surface water features. TREAT is located in an adjacent local topographically closed watershed, which also contains no identifiable perennial, natural surface water features. Therefore, flood management at these facilities focuses on controlling intermittent overland flows.

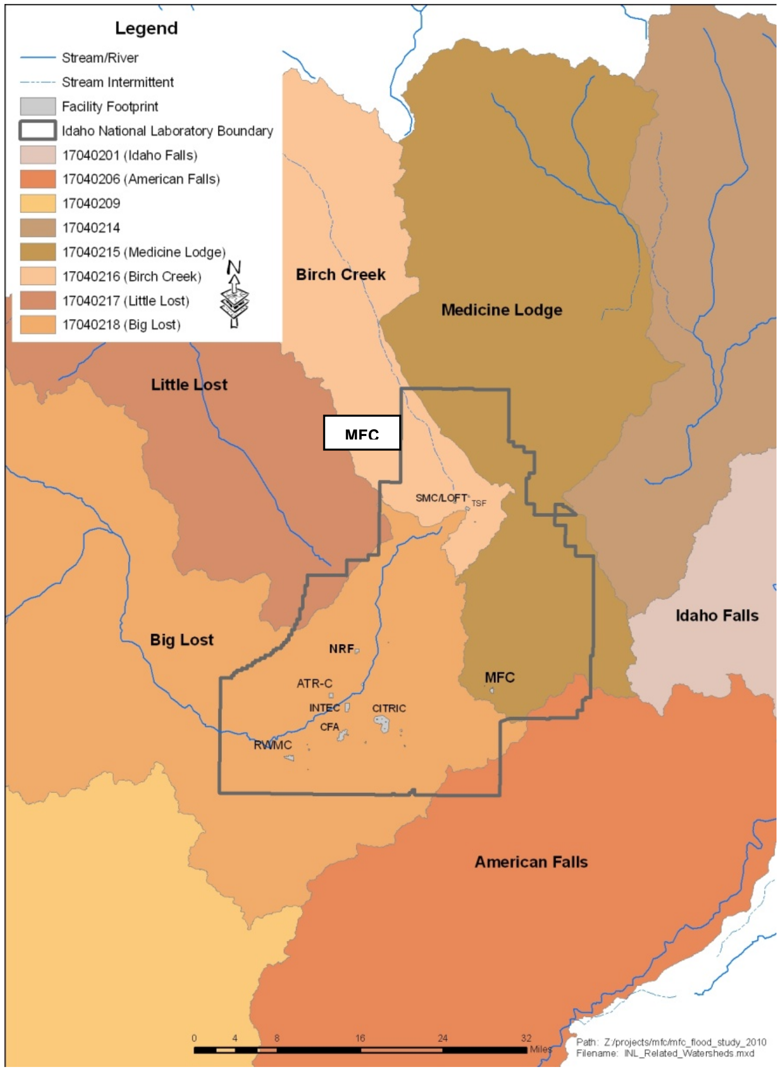

Figure 7. INL related watersheds (USGS 8-digit "cataloging units").

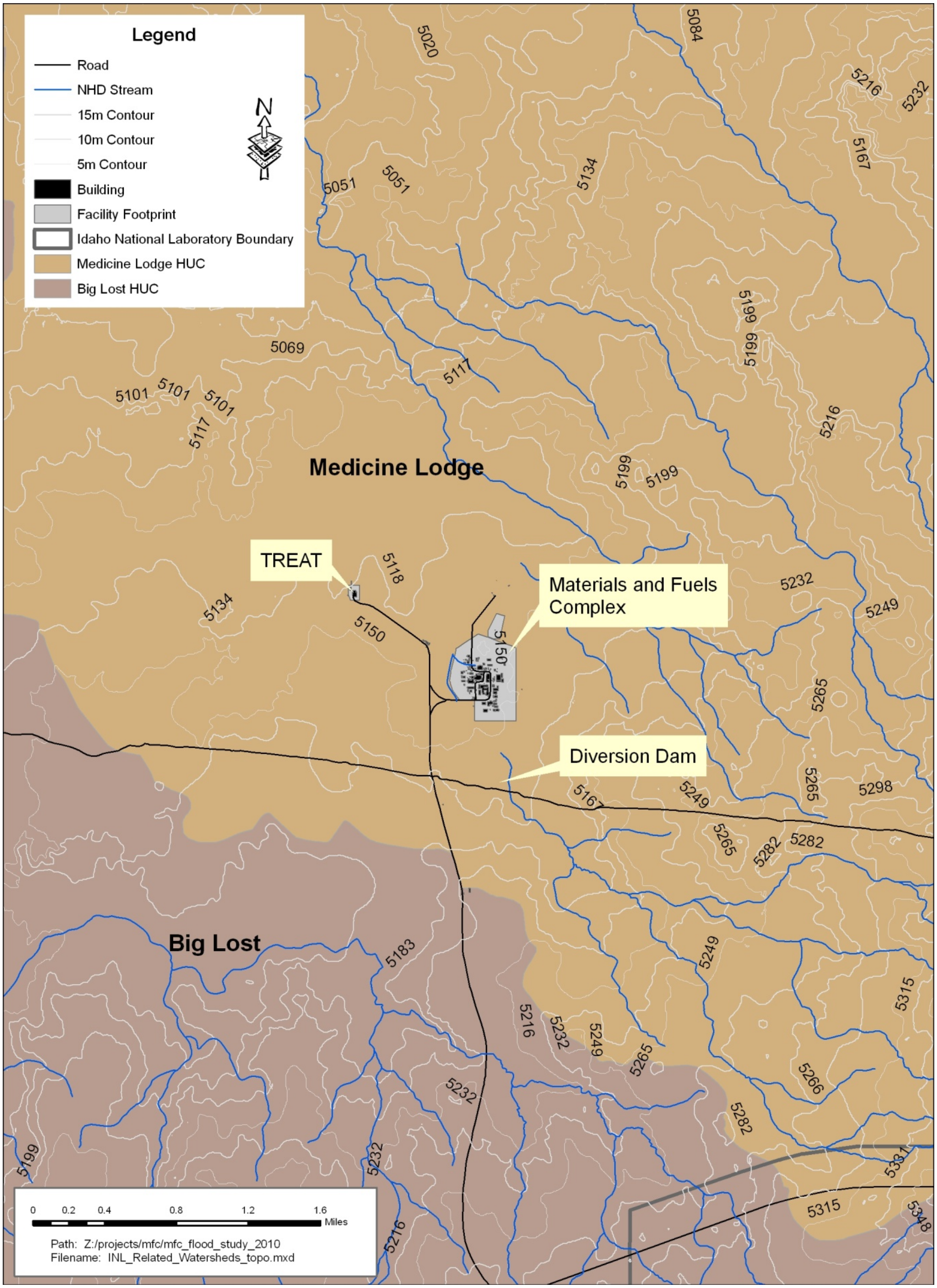

Figure 8. Topographic map of MFC and TREAT in the Medicine Lodge cataloging unit.

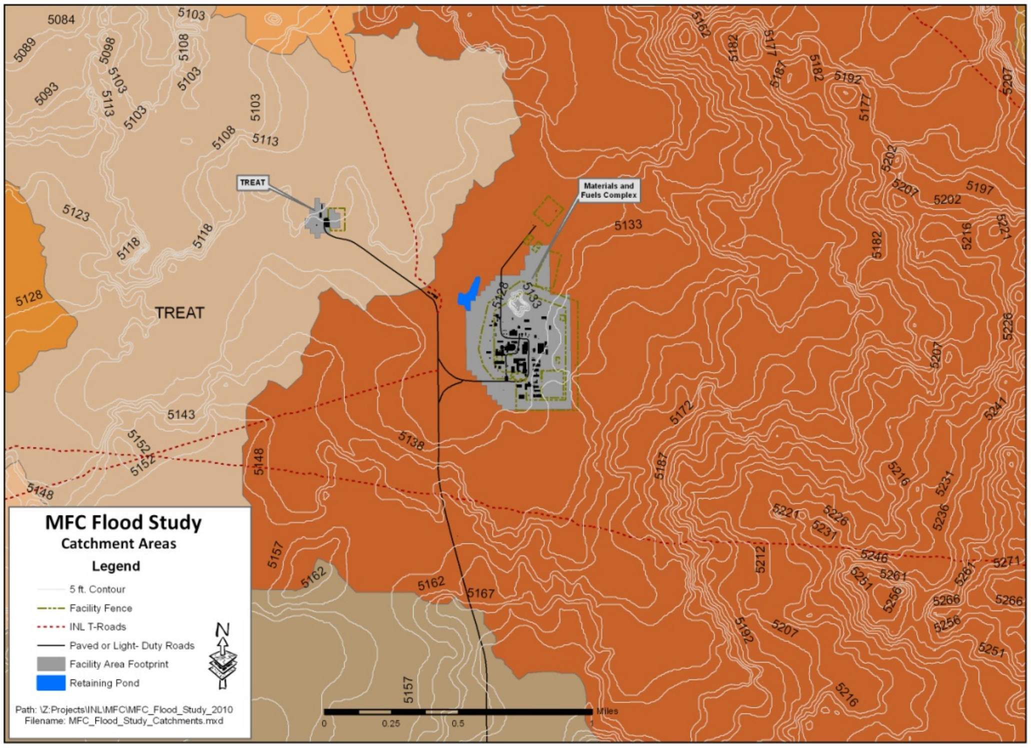

Figure 9. General topography of MFC and TREAT watersheds.

At the 8-digit HUC level, MFC and TREAT are located within the southern portion of the Medicine Lodge cataloging unit. The unit is bounded on the north by the Bitterroot Mountains and on the south by topographic ridges associated with several buttes (East and Middle Buttes). Topographically, the unit consists of gently rolling hills and large expanses of relatively flat land (Figures 10 and 11). In the northern portion of the unit, streams flow from the Bitterroot Mountains south into Mud Lake. In the southern portion of the unit, where MFC and TREAT are located, the land slopes north towards Mud Lake. There are no perennial lakes or streams in the southern portion of the unit on or near the INL Site.

Figure 10. MFC aerial photograph facing north (a) and south (b).

Figure 11. MFC aerial showing diversion dam channel (a) and TREAT (b).

At the 10-digit HUC level, all MFC facilities except TREAT are within a single local topographically closed watershed (the “MFC watershed”); TREAT is located in an adjacent closed watershed (the “TREAT watershed”).

Previous reports (e.g., CH2M Hill 1978) indicated that the maximum topographic relief in the MFC watershed is about 50 ft, ranging from 5,110 ft above mean sea level on the north boundary to 5,160 ft on a basalt ridge to the southeast. Since the watershed was not well described in past studies, the actual area described cannot be discerned. However, Geographical Information System (GIS) evaluations conducted for this study indicate that the MFC watershed slopes from the cinder cone in the southeast to northwest portion of the watershed (Figure 9) approximately 6.3 miles, at a slope of 66 ft per mile. The majority of the MFC watershed north of Highway 20 consists of gentle rolling hills (Figures 10 and 11). MFC facilities lie near the northern portion of the watershed; elevations within the fenced area are between 5,120 and 5,130 ft above sea level (CH2M Hill 1978). The USGS National Hydrography Dataset indicates some stream channels exist within the MFC watershed; however, these natural drainage channels only produce intermittent flows during large precipitation events or large snow melt-off events. The watershed contains no perennial, natural surface water features.

3.3.3 Vegetation

INL lies within the largest sagebrush-steppe region within North America (Kostelnik 2005). The natural vegetation at the INL Site consists primarily of a shrub overstory with an understory of perennial grasses and forbs. Most vegetation communities within the Site boundaries are dominated by various species or subspecies of sagebrush, although some communities dominated by saltbush, juniper, crested wheatgrass, and Indian rice grass are also present and distributed throughout INL. Exotic plant species, including cheatgrass and Russian thistle, are well established, particularly within disturbed areas. Native vegetation is typically removed within most INL operational areas to reduce the uptake and transport of potential contaminants and to improve drainage. The land surface in and around the MFC and TREAT operational areas is either barren soil or gravels, covered with asphalt or concrete (e.g., roads, pads, and sidewalks) or covered with small patches of lawn (Figures 10 and 11).

3.3.4 Geology

The geology of the INL site has been studied extensively and provided in numerous documents; the geology information provided below is summarized from Sehlke and Bickford (1993) and updated as appropriate. The ESRP is floored with basaltic lava flows and terrigenous sediments. It is approximately 80 to 100 km (50 to 62 mi) wide and over 560 km (348 mi) long (Figure 12). The ESRP extends in a broad arc from the Yellowstone Plateau on the east to the Idaho-Oregon border on the west. It transects and sharply contrasts with the mountainous country of the Northern Basin and Range province and the Idaho batholith. Much of the INL Site’s geology consists of Pleistocene and Holocene basalt flows. Three large silicic domes and a number of smaller basalt cinder cones occur on the INL Site and along the southern boundary.

3.3.4.1 Surface Geology

Both MFC and TREAT are within topographically closed watersheds; therefore, surficial materials are the result of upland erosion from the surrounding highlands or windblown loess (Figure 13). Low basalt ridges east of the facility rise as high as 100 ft above the level of the plain. Surficial sediments cover most of the underlying basalt, except where pressure ridges form basalt outcrops. Thickness of these surficial sediments ranges from 0 to 20 ft (Northern Engineering and Testing, Inc. 1988).

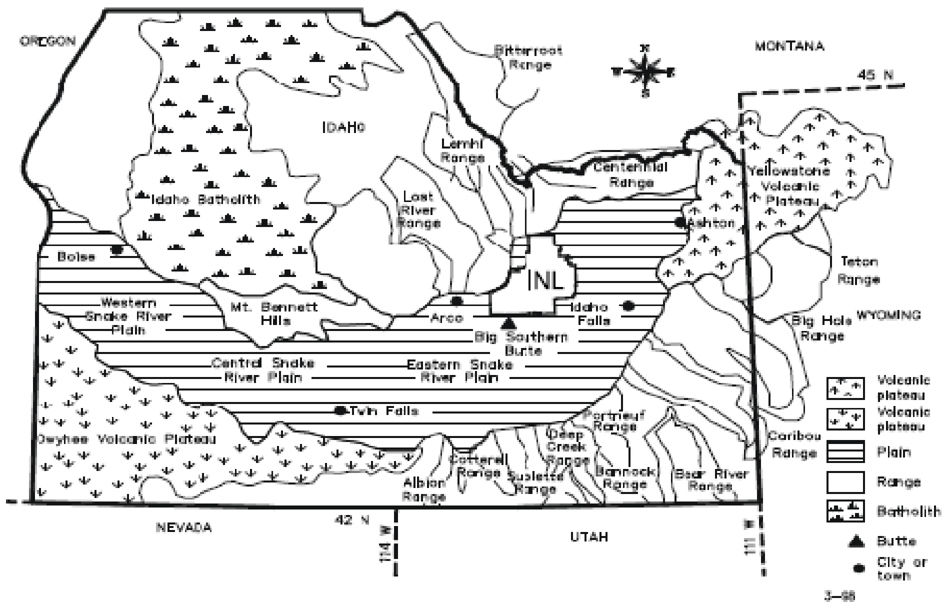

Figure 12. Geologic setting of INL (SAR-400).

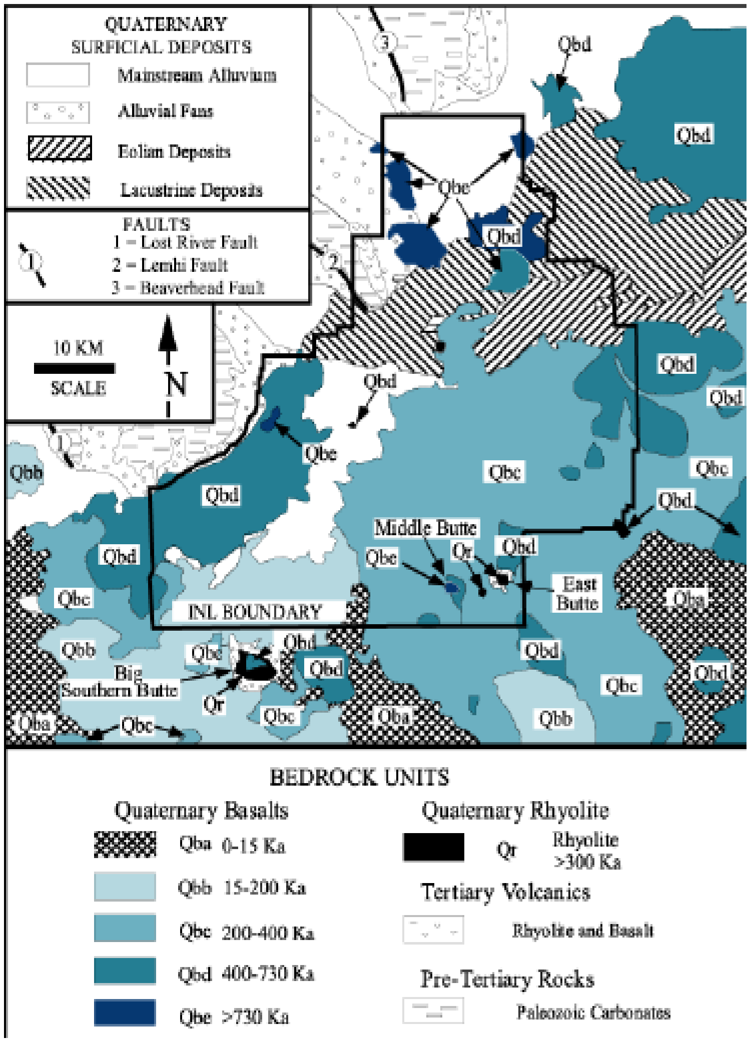

Figure 13. Geologic map of the INL area (SAR-400).

The thickness of surficial sediment near the MFC range from 0, associated with outcroppings at the surface, to 14 ft in depth. In general, the depths of the surface soils tend to increase from approximately 2 ft deep on the east side of the facility to a depth of 14 ft near the west side of the security fence. Test borings at MFC have revealed two distinct layers in the surface sediments. The uppermost layer, from zero to several feet below land surface (BLS), consists of a light brown silty loam. The upper 1 to 2 ft of this silty loam layer contains plant roots. This silty loam layer may also contain basalt fragments in areas where it directly overlies basalt. The lower layer is a sandy-silt (loess) that extends to the underlying basalt. The loess of this layer was probably transported by wind from other parts of the plain. The windblown loess is calcareous and light buff to brown in color. Small discrete lenses of well-sorted sands that occur within the loess are probably the result of reworking by surface runoff into local depressions. The lower portion of this loess layer often contains basalt fragments of gravel to boulder size. The surface of the underlying basalt, whether it is in contact with the upper or lower layer, is highly irregular, weathered, and often very fractured.

The two primary types of soils at MFC are classified as 425-Bondfarm-Rock outcrop-Grassy Butte complex and 432-Maim-Bondfarm-Matheson complex. The 425-Bondfarm-Rock outcrop-Grassy Butte complex is found throughout MFC (DOE-ID 1998). . The permeability of these soils is moderately rapid to rapid. Effective rooting depth is 10 to 60 in. The hazard of erosion is slight or moderate.

3.3.4.2 Subsurface Geology

The subsurface lithology presented in this section is based on information gathered from borings around MFC. Minor discontinuous sedimentary interbeds occur at various depths, overlying the tops of basalt flows. The subsurface geology is similar to that on the rest of the INL Site with the exception of a lack of continuous sedimentary interbeds beneath the facility. The sedimentary interbeds appear to be discontinuous stringers, deposited in low areas on basalt surfaces. They are generally composed of calcareous silt, sand, or cinders. Rubble layers between individual basalt flows are composed of sand and gravel to boulder-sized material. The interbeds range in thickness from less than 1 in. to 15 ft. Aerially extensive interbeds have been identified above the regional water table, at approximately 400, 550, and 600 ft BLS (Northern Engineering and Testing, Inc. 1988). The depth to the SRPA below MFC is approximately 640 ft BLS. The nature of these sedimentary interbeds and rubble zones does not appear to cause perching, but may retard the downward movement of water and produce preferred flow paths.

The thickness and texture of individual basalt lava flows are quite variable and range in thickness from 10 to 100 ft. The upper surfaces of the basalt flows are often irregular and contain many fractures and joints that may be filled with sediment. The rubble zones occur at variable depths and extents. Exposed fractures commonly have silt and clay infilling material. The outer portions of a flow (both top and bottom) tend to be highly vesicular. The middle portions of the flow typically have few vesicles and are dominated by vertical fractures formed during cooling.

3.3.5 Hydrology

This section provides regional and INL-specific hydrological information, including both surface water and groundwater features. The hydrology of the INL site has been studied extensively and provided in numerous documents; the geology information provided below is summarized from Sehlke and Bickford (1993) and updated as appropriate.

3.3.5.1 Surface Water

Three streams, the Big Lost River, Little Lost River, and Birch Creek drain the mountain region to the north and west of the INL Site and currently do or historically have flowed onto INL (Figure 7). However, irrigation and hydropower diversions, and infiltration losses along the channel bed often deplete these streams before they reach INL. These are the only perennial natural water bodies associated with the INL Site; no perennial water bodies exist near MFC. In the northern portion of the Medicine Lodge cataloging unit, streams flow from the Bitterroot Mountains southward into Mud Lake, a 4.3 mi^2 reservoir located north of INL, approximately 22 miles north of MFC. The topography in the southern portion of the cataloging unit, where MFC and TREAT are located, slopes from the MFC/TREAT area towards Mud Lake; however, there are no perennial surface water bodies in this portion of the cataloging unit. The primary surface-water features in the MFC/TREAT area are the anthropogenic features (e.g., drainage canals, ditches, and discharge ponds) constructed for MFC operations and for the collection of intermittent surface runoff (see Section 3.1).

3.3.5.2 Ground Water

Recharge to the SRPA near MFC occurs as snowmelt or rain. During rapid snowmelt in the spring, moderate recharge to the aquifer can occur. However, high evapotranspiration rates during the summer and early fall prevent significant infiltration from rainfall during this period. Because of the distance from the surrounding mountains and permanent surface water features (i.e., the Big Lost River), the SRPA beneath MFC is relatively unaffected hydrologically by underflow or recharge from these sources.

Perched water is defined as a discontinuous saturated lens with unsaturated conditions existing both above and below the lens. Classical conceptualization of a perched water body implies a large, continuous zone of saturation capable of producing some amount of water. At the INL Site, perched zones can occur over dense basalts that exhibit low hydraulic conductivity in addition to sediment interbeds that have low permeability. It is unknown which conceptual model is more prevalent at the INL Site. However, in the subsurface basalts at MFC, the "perched water" appears as small, localized zones of saturated conditions above some interbeds and within basalt fractures, which are incapable of producing any significant amount of water. The depth to the nearest perched water body beneath MFC and TREAT is over 400 ft below land surface.

The Eastern Snake River Plain Aquifer (ESRPA) is a continuous body of groundwater underlying nearly all of the ESRP. Depths to the water table from the INL land surface range from approximately 61 m (200 ft) in the northern part of the INL Site near TAN to more than 274 m (900 ft) in the south near RWMC. Aquifer boundaries are formed by contact of the aquifer with less permeable rocks at the margins of the plain. These boundaries correspond to the mountains on the west and north and to the Snake River on the east. The aquifer is approximately 325 km (200 mi) long, 65 to 95 km (40 to 60 mi) wide, and covers an area of approximately 25,000 km^2 (9,600 mi^2). It extends from Ashton, Idaho, northeast of the INL Site southwest to near Hagerman, Idaho.

The aquifer is composed of numerous relatively thin basaltic flows extending to depths in excess of 1,067 m (3,500 ft) below land surface. Over time, some of these flows have been exposed at the surface long enough to collect sediment. These sedimentary interbeds are sandwiched between basaltic flows at various depths. Estimates of the active thickness of the SRPA near INL are based on direct and indirect information obtained from wells and surface geophysical surveys. Direct evidence of the active aquifer thickness includes temperature gradients, lithologic variations in drill cores, and aquifer pumping tests. The active thickness of the aquifer in eight wells that penetrate the full thickness of the aquifer, as determined by temperature logs, ranges from 102 m (334 ft) to 368 m (1,207 ft).

Depth to the SRPA near MFC is approximately 640 ft BLS, based on 1995 water level measurements. Transmissivity of the SRPA near MFC range from 29,000 to 556,000 ft^2 per day based on aquifer test data from two MFC production wells (DOE-ID 1998).

4. FLOOD SCREENING ANALYSIS

DOE's flood hazard assessment process has two parts: first a flood screening analysis to evaluate the magnitude of flood hazards at a facility and then, if needed, a comprehensive flood analysis (DOE-STD-1023-95). The objective of the flood screening analysis is to conduct a preliminary flood hazard assessment that identifies potential flood hazards, including flood-induced rise in ground water, and to determine whether flooding can take place or whether the site can be considered a "flood-dry site" (ANS 1987). A flood-dry site is defined as "… one where the structures are physically removed from the potential sources of flooding so that safety from flooding is obvious and can be documented with minimal effort." Because there have been local flooding events at MFC, it can be concluded that it is by definition not a flood-dry site.

The screening analysis consists of a preliminary flood hazard assessment that provides an initial estimate of the site design basis as specified by DOE design guidelines (McCann and Boissonnade 1988). The final part of the screening analysis involves a review of the vulnerability of onsite facilities by the site manager. The objective of the screening procedure is to determine whether flood hazards should be considered as part of the INL Site design basis for a site.

The preliminary hazard analysis is based on a systematic review of available information and existing flood studies. This includes data collection, a review of the historical record and the physical environment to determine whether flooding onsite is physically realizable; and if so, to assess the annual probability of flooding based on data produced by flood insurance or similar studies to estimate the probability of flood peaks. The flood assessment is intended to evaluate the potential for onsite flooding at the individual facility level; however, it does not consider localized flooding at the site due to precipitation (e.g., local run-off, storm sewer capacity, roof drainage) (McCann and Boissonnade 1988)

A flood screening analysis includes the following three steps:

- Step 1: Identification of the sources of flooding
- Step 2: Evaluation of flooding potential
- Step 3: Preliminary flood hazard analysis.

4.1.1 Identification of Potential Sources of Flooding at MFC

DOE-STD-1023-95 requires that each facility assess the following potential sources of flooding:

- River flooding
- Dam failures
- River-related levee or dike failures
- Flood runoff/drainage
- Tsunamis
- Seiches
- Storm surges or large waves
- Ground-water
- Mudflows
- Subsidence-induced flooding.

Based on the appropriate topographical and meteorological conditions, DOE-STD-1023-95 requires each potential source of flooding to be screened to determine potential flooding due to multiple sources and other possible chains of events. It states, "For each of the sources of potential flooding, simple criteria (without performing any analysis other than those collected) shall be provided establishing whether the site is affected by potential flooding from this source. These criteria include the applicable physical arguments that certain sources not present are very unlikely or that their consequences on the site are negligible or nil."

INL, including MFC and TREAT, has been well monitored and characterized for approximately 60 years. Based on that information most of the potential sources of flooding described above could be removed from further consideration relative to MFC and TREAT. Both facilities are in topographically closed watersheds within the regional Medicine Lodge cataloging unit. They contain no perennial surface water bodies and they do not receive surface water flow from other cataloging units associated with the INL Site (e.g., Big Lost, Birch Creek, Idaho Falls, Little Lost, or American Falls). Water in the Medicine Lodge cataloging unit would generally drain from the direction of INL to Mud Lake. The surrounding watersheds are topographically lower in elevation and they are isolated from the facilities by relatively large topographic ridges. Therefore, no potential sources of riverine flooding, dam, and dike/levee failures are present at MFC.

Tsunamis, seiches, storm surges, and large waves are associated with oceans and/or large open bodies of water. DOE-STD-1023-95 defines a tsunami as "A long period ocean wave caused by an underwater disturbance such as an underwater earthquake, landslide, or volcanic eruption." MFC and TREAT are located more than 5,100 ft above sea level and approximately 1000 miles from the nearest ocean. DOE-STD-1023-95 defines a seiche as "A cyclic oscillation or sloshing of a lake or large body of water due to the effect of winds, seismic forces, and/or atmospheric pressure." The closest "large" bodies of fresh water are the American Falls Reservoir, an 87.5 mi^2 reservoir on the Snake River approximately 30 miles southwest of MFC, and Mud Lake, a 4.3 mi^2 reservoir located approximately 22 miles north of MFC. Seiches are known to occur at American Falls Reservoir; however, in addition the distance between the reservoir and the facilities, the reservoir is approximately 700 ft lower in elevation (4357 ft versus 5,125 ft above mean sea level). Seiches are not generally known to occur at Mud Lake; however, in addition the distance between the lake and the facilities, the lake is approximately 340 ft lower in elevation (4785 ft versus 5,125 ft above mean sea level). Therefore, no large sources of water are present at or near MFC and TREAT to potentially cause tsunamis, seiches, storm surges, or large waves.

As described above (Section 3.3.5.2) the depth to the ESRP aquifer beneath MFC is approximately 640 ft below land surface, and the depth to the nearest perched water body is over 400 ft below land surface; therefore, the probability of groundwater-related flooding is essentially nil. The skeletal framework for the ESRP aquifer is primarily rigid basalt flows; therefore, the probability of natural or anthropogenic subsidence-induced flooding is essentially nil.

As described above (Section 3.3.4.1), MFC and TREAT are located in a relatively small, closed watersheds with stable geology and soils, which have a slight to moderate potential for erosion with no perennial stream channels. Sharp topographic changes within the watershed are basalt outcrops; there are no steep soil or sediment bluffs, cliffs, or stream banks within the watershed susceptible to mass wasting during intermittent flood events. With relatively low topographic relief within the watershed and without established stream channels, surface water flows will be dispersed and; therefore, unlikely to have sufficient hydraulic energy to support the transport of large quantities of soil material to cause significant mudflows during intermittent overland flow events. Consequently, while some soil erosion would be expected during extreme precipitation events, the potential of significant mudflows impacting MFC is negligible.

Based on the assessment provided above and the requirements of DOE-STD-1023-95, the only potential type of flooding event that must be considered at MFC is from intermittent overland flow.

4.1.2 Evaluation of Flooding Potential at MFC

Because MFC and TREAT are not located near a river, or a large body of surface water that could cause a tsunami, seiche, storm surge, or large waves, detailed flood analyses have not been conducted by DOE, the Corps of Engineers, or other entities with the exception of FEMA.

FEMA FIRMs were developed for Bingham County where MFC is located. FIRMs identify flood zones:

- Zone A indicates areas subject to 100-year floods
- Zone B indicates areas between the limits of 100 and 500-year floods
- Zone C indicates areas of minimal flooding
- Zone D indicates areas of undetermined, but possible flood hazards.

MFC is addressed in Panel 160018 0050B. The footnotes indicate that this panel is not published, but the area is designated Zone C. TREAT, located west of MFC is included on Map Panel No. 25 of 750, Section 11, which is also designated as Zone C (INL 2004). Although FEMA designated the MFC area as "Zone C" (minimal potential for flooding), local flooding is known to have occurred at MFC.

4.1.3 Historical Flooding at MFC and TREAT

Given the appropriate conditions, TREAT can receive overland flow from a potion of a 140-acre watershed and MFC can receive overland flow from six subwatersheds. The total watershed area that could potentially contribute runoff to MFC is 7.8 mi^2; with the subwatersheds ranging in size from 0.11mi^2 to 4.9 mi^2 (Figure 14). Water from the subwatersheds and from the MFC operational area runoff into one or more of four "subbasins" (B1 - B4). During times of high precipitation when the ground is not frozen, surface water will first infiltrate into the soils, then pond in low-lying areas, and then may eventually become overland flow. Under colder conditions, precipitation or snowmelt may be unable to infiltrate into the ground and may immediately become overland flow. Runoff can occur as overland sheet flow and/or become concentrated in normally dry depressions and channels. Excess flows will seek the lowest point in the watershed. Since TREAT is located near the upper topographic ridge of the TREAT watershed (approximately 1,000 ft from the ridge top) there is little land surface upgradient that can contribute overland flow to the facility. However, since MFC is located in the lower northwestern portion of the MFC watershed, overland flows will generally flow towards and/or through MFC, seeking the topographic low point of the watershed, north of the IWP. Each watershed was assessed to determine the cumulative potential impact of overland flood flows on nuclear facilities within the MFC and TREAT operational areas.

The first documented flood at MFC occurred in 1963. Very little information on this event could be found. However, several documents associated with the MFC diversion and drainage system provide some insight into the event (CH2M Hill 1978 and ANL 1993). The correspondence states that approximately 1 in. of rain fell on frozen ground causing some flooding at the INL Site and filling the drainage channel around the west side of MFC. The flows resulted in surrounding some of the facilities and water backing up in front of the "main office building" (Assumed to be MFC-752). No information was provided as to the actual flood depths or location of the high water marks.

Because few contemporary descriptions of the flood event have been found, attempts were made to interview MFC employees to gain insights to the extent of pass flood events at MFC. MFC workers employed at the time have retired from service at INL, and none are available for questioning. At the time of the flood, the EBR-II, the primary nuclear facility at MFC, had been completed, as was the power plant. The sodium boiler plant had been completed just the year before. Dry criticality had been achieved in the fall of 1961, and by the end of 1962, the reactor would have been fueled and sodium coolant was circulating. Histories of the reactor indicate that it achieved "wet" criticality in November 1963.

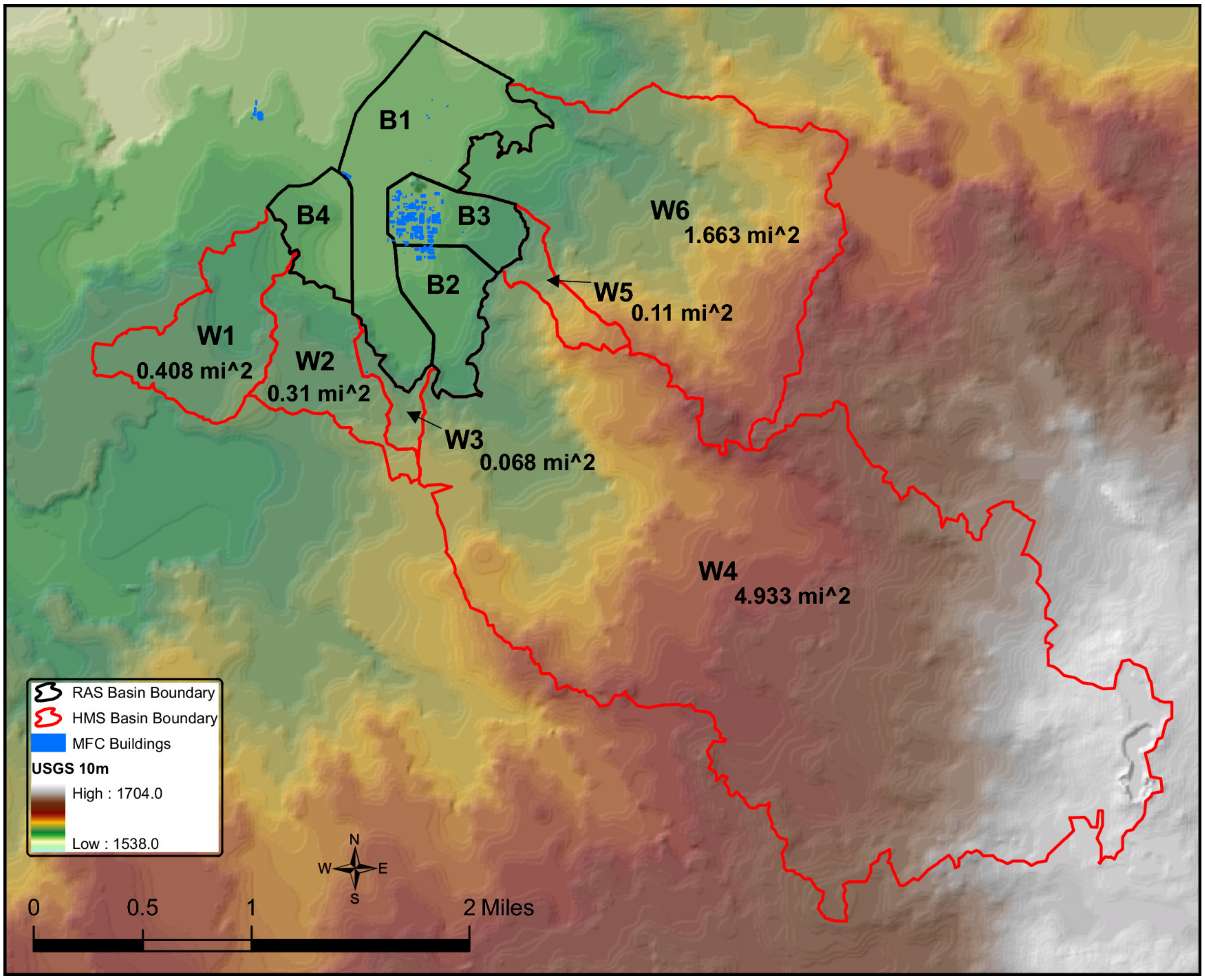

Figure 14. Subwatersheds and basins which could potentially contribute flood-flows to MFC, simulated with HEC-HMS (W), and HEC-RAS (B).

However, the formal history of the reactor does not mention this flood event, which may well have occurred in the spring of that year, before the approval of wet criticality experiments.

As a result of the 1963 flood, the grade in the Interceptor Canal around the west side of MFC and the outlet of the IWP was lowered to an elevation of 5114.6 ft. This elevation is as low as the pond could be excavated without removing basalt outcrops downstream of the pond. In addition, in 1968 a diversion dam, with head gate and culvert was constructed about 1/2-mile south of the facility. A spillway was designed to divert excess water from the dam through the overflow channel to the Interceptor Canal. In addition, other drainage ditches and drainage pipes were constructed within MFC operations area to improve onsite drainage.

In January1969, a similar precipitation and snow-melt runoff over frozen ground event occurred (Note: NOAA recorded a 1.64-in. precipitation event in June, but no precipitation events greater than or equal to 1 in. in January. Clawson [2010] stated that local precipitation events can vary widely and that such an event could have occurred at MFC without being recorded at CFA). Water levels specific to the MFC site were not recorded, but they were sufficient to reach and overtop U.S. Highway 20, south of the facility. Photographs indicate the level of inundation at MFC (Appendix B), including the northern half of a road, labeled U.S. Highway 20 by a sign visible a photo (No. 9880). During this event, the MFC Diversion Dike failed around the culvert and floodwater again flooded around MFC (ANL 1993). No photographs were taken of MFC or TREAT facilities at the time of this flood, suggesting that the flooding did not significantly affect any facilities. The CH2M Hill report (1978), indicates that the drainage channels at MFC were observed backing up into the main channels during periods of high spring runoff, and high water marks showed that water had been as high as 5118 ft above mean sea level, 3.5 ft higher than the retention pond outlet, possibly due to ice or other obstruction of culverts. Repairs were made to the diversion dam and the fill material compacted appropriately.

One long-time site worker, now retired, remembers a long, hard rain preceding the flooding. He also remembers that no buildings on MFC were flooded. This can be confirmed anecdotally by contemporary newspaper weather reports from the Idaho Falls *Post Register*, which indicate that in the 2 weeks prior to the event, temperatures rose to a range between 38 and 43°F for 5 days, during which 1.63 in. of precipitation fell. The precipitation had ended by January 22 when, the *Post Register* reported flooding in local creeks. This rainfall was followed by a hard freeze, and then a second thaw with lesser precipitation of 0.14 in. over 2 days. The *Post Register* provides no coverage of the events at MFC, but it does confirm the retirees' memories of hard rains over several days (in January, when rain would be little expected) and shows that flooding was occurring in other areas nearby. In addition, it suggests a complex series of thawing and freezing that may have resulted in ice accumulation in areas where thermal transfer is high (i.e., around the galvanized pipe of the culvert) while a large preponderance of water remained in a liquid state, rising behind the diversion dam.

The most recent overland flow event occurred during the spring of 1995 when warm air conditions caused snowmelt over frozen ground, precluding subsurface infiltration. During this event, overland flow from the surrounding watershed flowed into the IWP for approximately 4 days. The pond had sufficient capacity to hold the entire flow; no water was observed discharging from the pond to the overflow channel.

4.1.4 Preliminary Flood Hazard Analysis

DOE-STD-1023-95 recommends that each facility identify potential sources of flooding and conduct a preliminary flood hazard analysis to assess flood potential utilizing conservative assumptions; existing climate, geological, and facilities information and, where available, existing flood assessments. Based on the results of this assessment, if the facilities are located outside of areas of inundation, then no future studies would be required under DOE's graded approach. If it is determined that the facilities are located inside of areas of potential inundation, then additional assessments would be necessary to refine the data and reevaluate the potential flood based on the NPH criteria for the given facilities.

Because a formal flood hazard assessment has not been conducted at MFC, little information could be gained from facility records and flooding is known to have occurred at the facility, it was determined that investing more time and funding to conduct a preliminary flood hazard analysis would produce highly uncertain results and would not be cost effective. Therefore, a detailed flood hazard assessment was conducted for MFC. The detailed flood hazard assessment is discussed below.

5. DETAILED FLOOD HAZARD ASSESSMENT

This section provides a summary of the Comprehensive Flood Hazards assessment. According to DOE-STD-1023-95, the need to perform a site comprehensive hazard assessment depends on the potential DBFL impact on the facilities for the flood hazard exceedance probabilities. Guidelines to evaluate these impacts are provided in DOE-STD-1020-2002. These guidelines recommend the design basis for DOE facilities based on the following factors: (1) types of potential flood hazard, (2) performance category, (3) reliability of flood protection devices, and (4) acceptable level of risk. It also states that the flood hazard will be defined in terms of the annual probable frequency of exceeding specified elevations and that all uncertainties in estimating flood levels shall be propagated in the flood hazard analysis. Factors 1 and 2 are addressed in Section 1.2; Factor 3 is addressed in Sections 3.1 and 5.2.8; and Factor 4 is addressed in Section 5.2.7.

DOE-STD-1023-95 states that a comprehensive flood hazard assessment shall consider detailed meteorologic, hydrologic, and hydraulic assessments of the potential flood hazards determined by the flood screening and an evaluation of the reliability of flood protection systems (e.g., dams, levees), if present. This includes:

- Estimating rainfall and snowfall frequency in watersheds.
- Assessing overland flow due to precipitation (Crawford and Kinsley 1966).
- Hydrologic modeling of watershed responses using validated models (IACWD 1986) and (U.S. Army Corps of Engineers 1986).
- Assessing discharge (flow rates) and flood elevations using detailed hydraulic modeling techniques (e.g., HEC) (1986).
- Estimating joint natural hazard events frequency. For example, a joint probability analysis shall be performed to assess surge level frequencies (Ho, et al. 1987).
- Assessing the likelihood of upstream dams and levees failures. All causes of dam failures should be accounted for (McCann and Boissonnade 1988b).
- Assessing the uncertainty due to the limited data for estimating model parameters, the modeling of physical processes, and interpretation of the available data.

Based on DOE's NPH flood criteria and the preliminary assessment discussed above, the only type of potential flood hazard, which could be a potential threat to MFC and TREAT, is from overland flow during large precipitation or snowmelt events. MFC contains PC 0, 1, 2, and 3 facilities and TREAT contains PC 0, 1, and 2 facilities. However, this assessment was conducted conservatively to accommodate potential for future construction. This study assessed MFC and TREAT utilizing the methods and criteria required for assessing PC 3 facilities. The highest ranked existing facilities at MFC and TREAT are PC 3, which DOE established an acceptable level of risk level of 1×10^{-4}. In addition, this study was conducted a PMF assessment. The reliability of the MFC flood control system was assessed based on existing information; a detailed engineering investigation was not conducted as part of this study.

This study utilized a graded approach, which included first conducting a PMF assessment utilizing the most conservative assumptions (using Probably Maximum Precipitation [e.g., no infiltration or evaporative losses, and considering the storm water management system to be non-functional]). Because the modeling results indicated some facilities were inundated under these conditions, the modeling parameters were relaxed to those associated with a 1×10^{-4} (10,000 year), as required for PC 3 SSCs. Other conservative assumptions (e.g., Manning's coefficient) were iteratively assessed for sensitivity and were relaxed as appropriate. The assessment is summarized below and discussed in detail in Appendix A.

5.1 Critical Flood Elevations (CFEs)

This assessment was conducted to determine the potential for flooding at existing MFC and TREAT facilities, rather than the construction of new facilities. DOE-STD-1020-2002 states that for existing facilities, SSCs should be reviewed to determine the level of flooding, if any, that can be sustained, without exceeding the performance goal requirements. For each SSC there is a critical elevation, which if exceeded, causes damage or disruption such that the performance goal is not satisfied. This is referred to as the Critical Flood Elevation (CFE). If the CFE is higher than the DBFL, then the performance goals are satisfied. Due to questions concerning the elevations provided on the facility drawings, extensive surveying was conducted at MFC (Figure 15). To conduct the surveys, the nominal floor level was located at each facility assessed and surveyed at the elevation of utilities that support the facility or at the actual base elevation of the facility (Figures 16a and 16b). The locations and elevations surveyed were selected based on best professional judgment. Surveys were conducted, adjusted, and logged using a Trimble 4800 GPS unit (vertical resolution $\pm$ 0.5 cm). A Topcon total station was utilized to measure the building corners. All points were surveyed using NAVD88 and NAD83 coordinates.

Horizontal and vertical information was obtained for at least two points for each facility with the exception of Buildings 775, 776, and 704. No building corner information could be obtained for these buildings due to their bunker-type nature of construction. Finished floor elevation information for these buildings was obtained by either by opening outside doors and surveying through the doors or by obtaining an elevation at the door threshold directly outside of the facility. The elevations that were determined at the building corners were not necessarily representative of the finished floor elevations, so additional surveying was conducted to better determine the actual finished floor elevations.

5.2 Preliminary Natural Phenomenon Hazard Flood Assessment

Pacific Northwest National Laboratory (PNNL) modeled potential flood hazards at MFC and TREAT (see Appendix A) (PNNL 2010). The PNNL assessments were conducted in accordance with DOE Standard DOE-STD-1020-2002 and DOE Standard DOE-STD-1020-2023-95, including:

- A determination of the DBFL for the flood hazard
- An evaluation of the critical flood elevations for each facility (SSC) assessed relative to the DBFL.

This assessment evaluated the flood potential at MFC and TREAT utilizing a graded approach; initially utilizing conservative assumptions and relaxing those assumptions as prudent and appropriate. This study utilized existing and updated climate, geological and facilities information, as discussed above, and GIS digital elevation models and Light Detection and Ranging (LiDAR) data to assess topographic relief. Global Positioning System (GPS) surveys were utilized to estimate CFEs. The results of these assessments are summarized below and discussed in more detail in Appendix A.

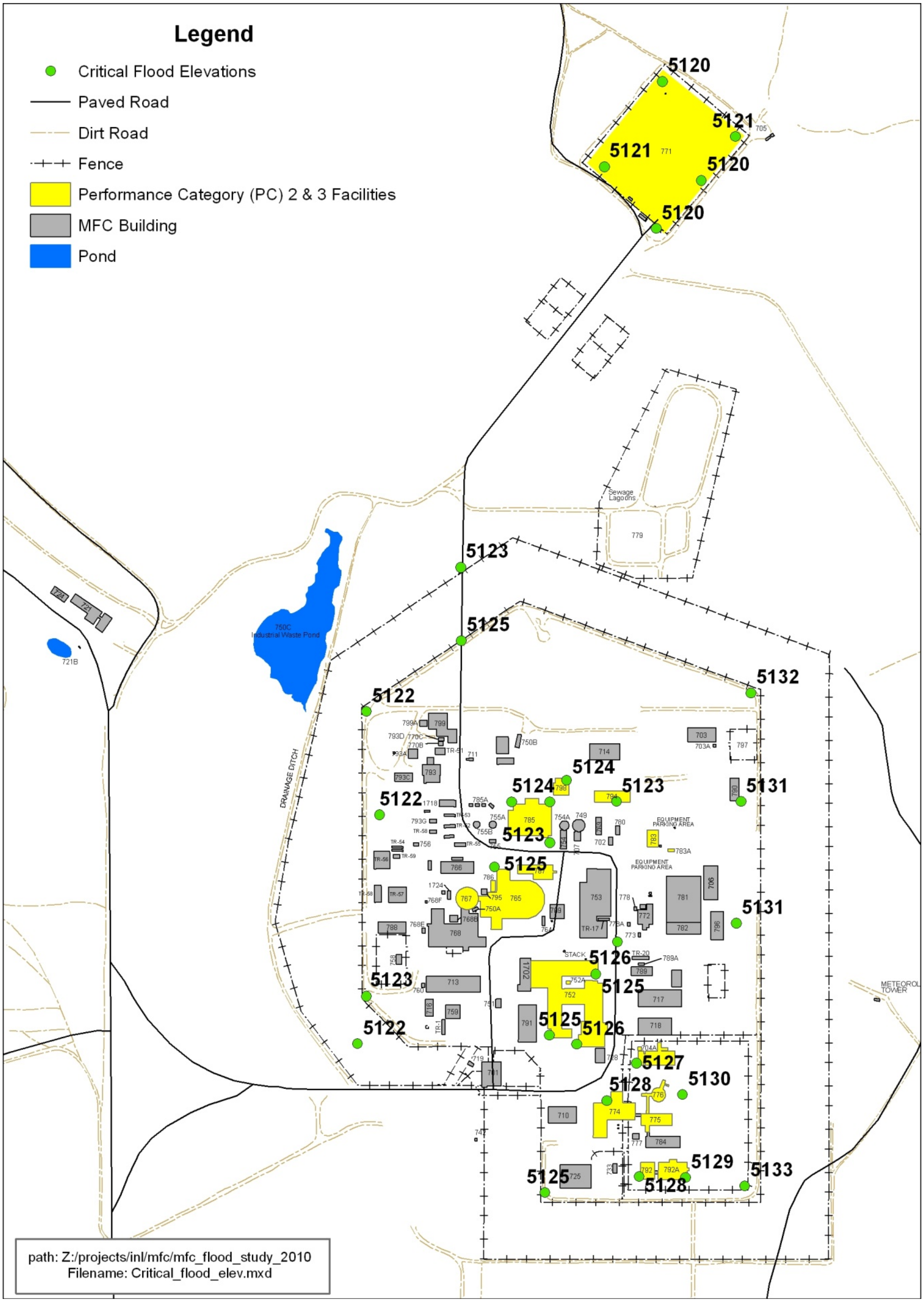

Figure 15. Selected locations and elevations surveyed at MFC.

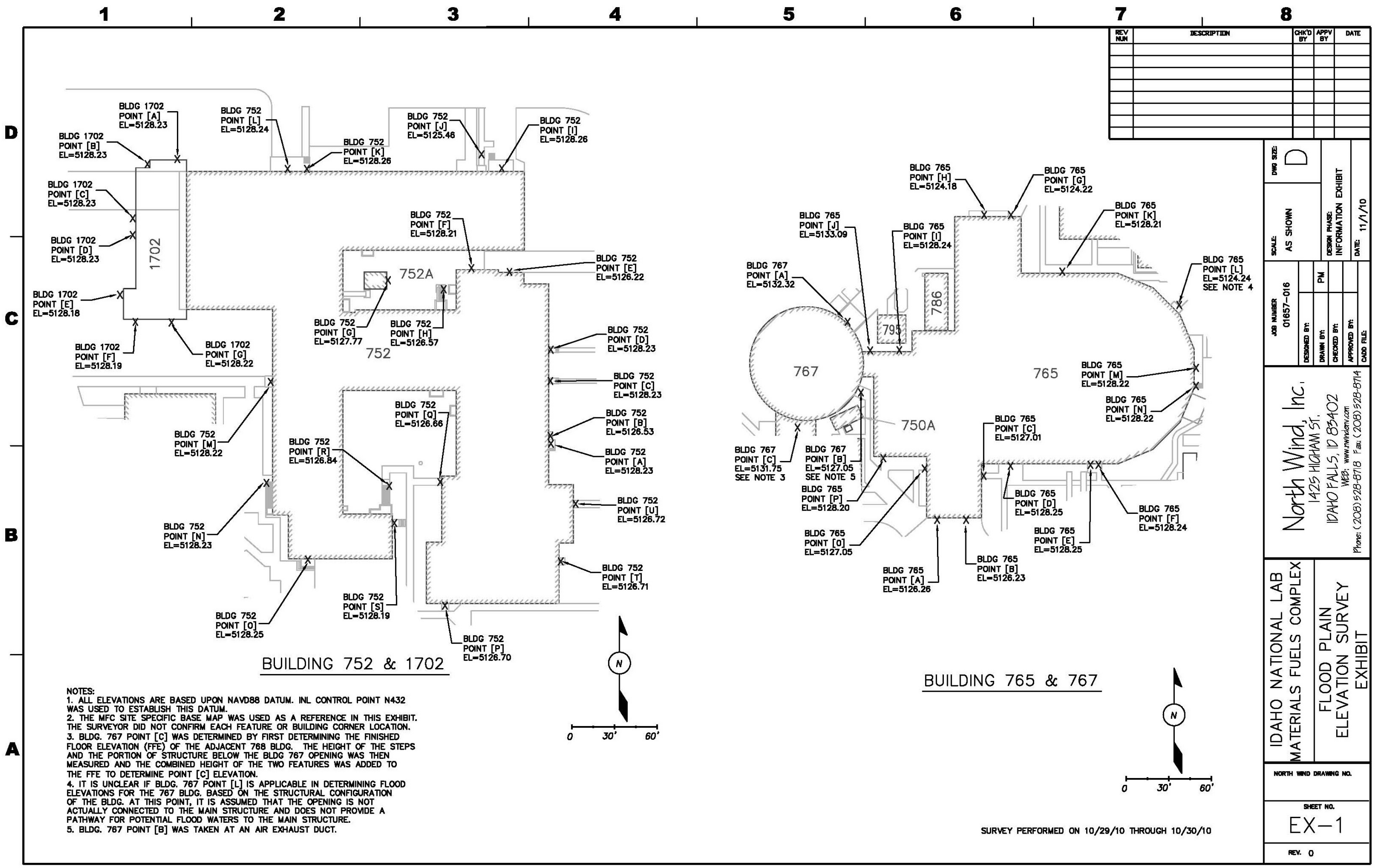

Figure 16a. CFEs for selected facilities at MFC.

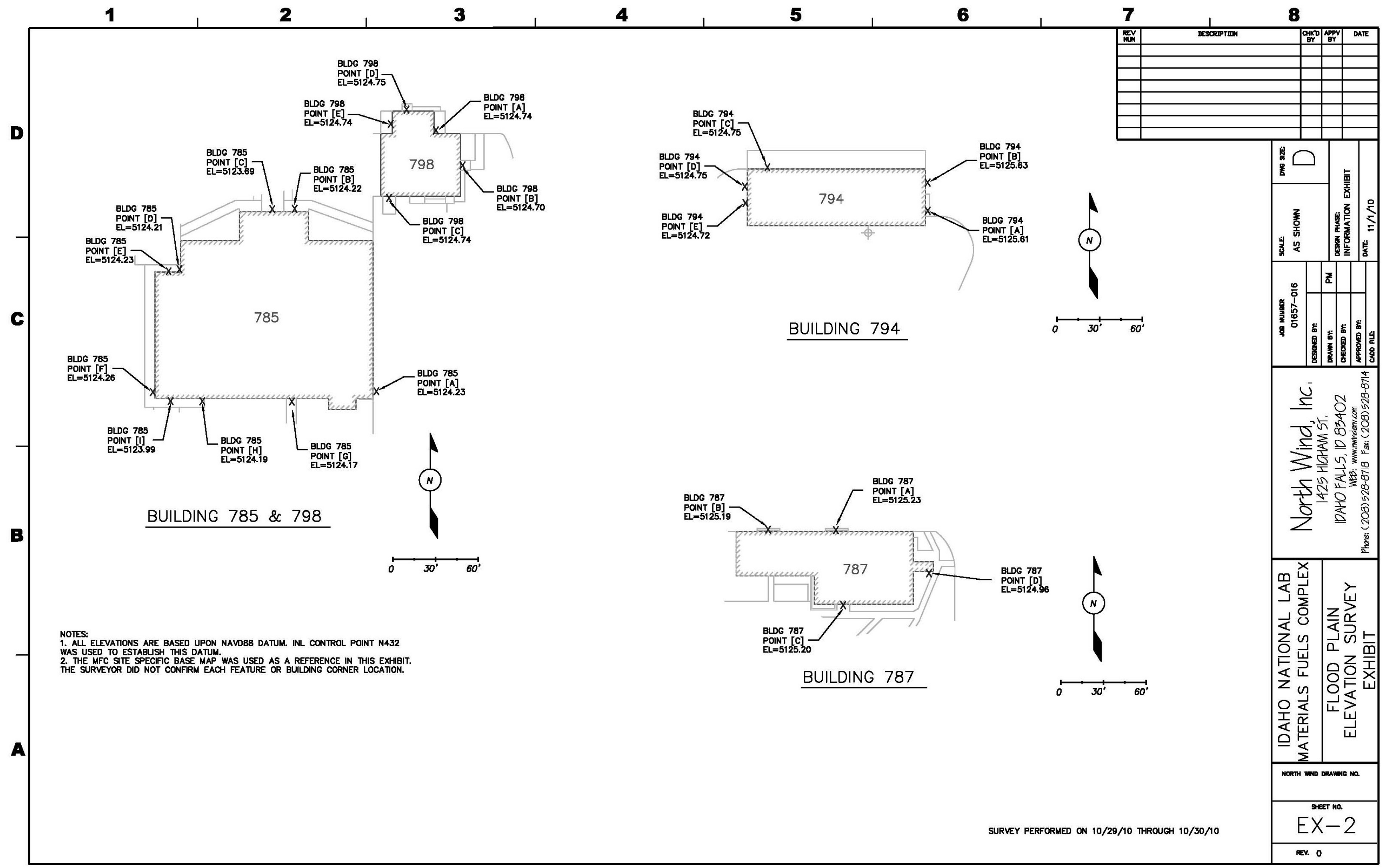

Figure 16b. CFEs for selected facilities at MFC.

5.2.1 Estimation of Rainfall and Snowfall Frequency in Watersheds

INL-specific historical precipitation and snowfall data are summarized above in Section 4.2.1. The probable maximum precipitation (PMP), which is the basis for the PMF, was estimated using Hydrometeorological Report No. 57 (NWS 1994). The resulting design storm from this analysis is composed of a 6-hour total rainfall of 9.12 in., with approximately 3.80 in. falling in the first 15 minutes (Figure 17). Based on discussions with NOAA's Field Research Division, located in Idaho Falls, it was determined that a standard 10,000-year precipitation event has not been estimated for INL (Clausen 2010). However, a 10,000-year precipitation event was previously estimated for a flood analysis conducted at the RWMC (Dames and Moore 1993). Based on a review by NOAA (Clausen 2010), it was determined that no extreme precipitation events have occurred that would significantly change the events estimated in the 1993 analysis. The 10,000-year event storm consisted of 2.2 in. of total rainfall over the first hour, with approximately 4.65 in. falling in 24 hours (Table 7).

It should be noted that the PMP described above is approximately equal to the total annual average rainfall at INL (8.51 in. per year) based on the 56-year period of record at the CFA (Table 4). For comparison to the PMP, a statistical analysis of meteorological data from the CFA for the period of 1950 through 1995 provides estimates of 1.7 in. of precipitation for a 25-year (3.9% Annual Exceedance Probability [AEP]), 24-hour storm event and 2.2 in. of precipitation for a 100-year (1% AEP), 24-hour storm event (INL 2010 and Sagendorf 1996). Therefore, the 9 in., 24-hour local PMP is much less likely than 1% AEP.

5.2.2 Overland Flow Assessment due to Precipitation

The PMP and 10,000-year precipitation events were transformed into watershed outflows using the USACE HEC-HMS software. For each analysis, the synthetic, the Soil Conservation Service (SCS) unit hydrograph (UH) option within HEC-HMS was used as the transformation method to convert the design storm hyetographs to stream flow hydrographs. Losses due to surface ponding, infiltration, and evaporation were assumed to be zero to provide the most conservative (highest) runoff for the PMF analysis. Therefore, the runoff volumes equaled the precipitation over the watersheds.

The 7.8 mi^2 MFC watershed was subdivided into six contributing subwatersheds ranging from 0.11 mi^2 to 4.9 mi^2. The culverts and other drainage systems at MFC were assessed under both "blocked" (plugged) and "open" (unplugged) conditions for both PMP and 10,000-year precipitation events. The estimated maximum runoff was input to HEC-RAS model for each subwatershed. Each subwatershed was assessed individually and then the combined impacts were assessed.

The 0.22 mi^2 (140 acre) TREAT was treated as a single watershed. A preliminary assessment of the TREAT site suggested that, given the relatively small contributing area and the height of the facility floor above the channel, the flood hazard was low. TREAT was analyzed for PMF to conservatively estimate overland flows in the watershed, by transforming the PMP to runoff using the rational method with no surface ponding, infiltration, or evaporative losses. The estimated maximum runoff was input to HEC-RAS model at the upstream end of the watershed as a constant flow for the steady-state hydraulic modeling analysis.

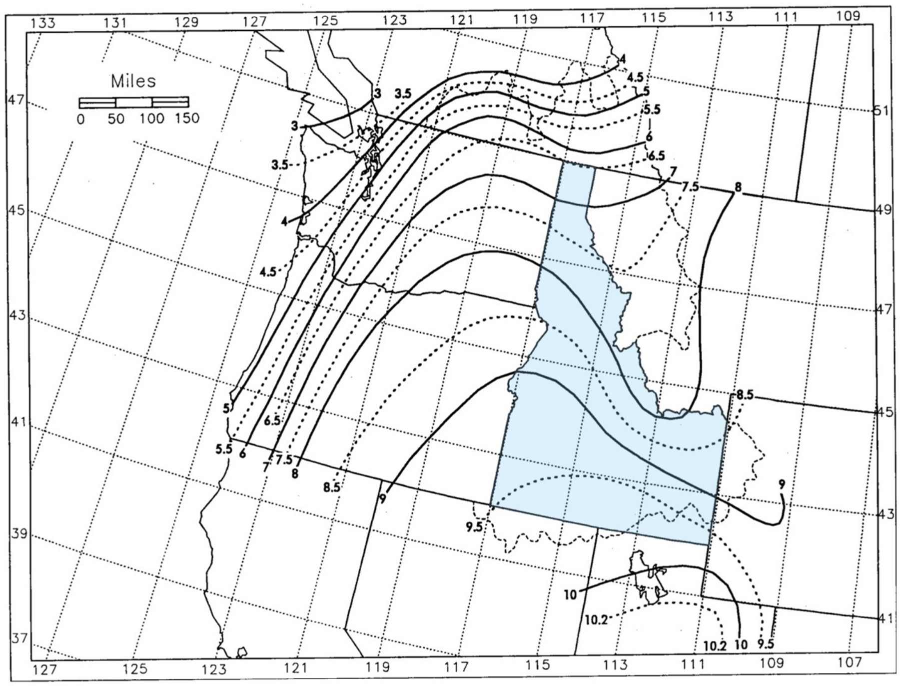

Figure 17. One hour, 1-mi^2 local storm probable maximum precipitation in inches for elevations to 6,000 ft. (From Hydrometeorological Report No. 57, Hansen et al. 1994).

Table 7. Temporal distribution of precipitation depths (Dames & Moore 1993).

	Cumulative Precipitation Depth (Inches)						
Duration	2-Year	10-Year	20-Year	100-Year	500-Year	1,000-Year	10,000-Year
5 Minute	0.13	0.23	0.27	0.35	0.44	0.49	0.64
10 Minute	0.20	0.36	0.40	0.55	0.69	0.76	0.99
15 Minute	0.25	0.46	0.52	0.69	0.87	0.96	1.25
30 Minute	0.35	0.63	0.73	0.96	1.21	1.33	1.74
1 Hour	0.44	0.80	0.92	1.21	1.53	1.68	2.20
2 Hour	0.54	0.91	1.07	1.37	1.69	1.83	2.32
3 Hour	0.62	1.00	1.19	1.51	1.86	2.01	2.56
6 Hour	0.80	1.20	1.45	1.80	2.22	2.40	3.05
12 Hour	1.02	1.48	1.75	2.25	2.82	3.07	3.95
24 Hour	1.20	1.70	2.00	2.65	3.32	3.62	4.65

5.2.3 Hydrologic Modeling of Watershed Responses using Validated Models

The DBFLs for MFC and TREAT were based on estimating the PMP and 10,000-year flood events and then using them to force a watershed runoff model for the surrounding upland basins as well as the MFC site itself. Runoff (outflows) was modeled using the USACE HEC-HMS and HEC-RAS software. Outflows modeled with HEC-HMS were input to the HEC-RAS hydrodynamic flood routing model-to-model flood flows and elevations for both MFC and TREAT.

The approach was to use the hydraulic model HEC-RAS to route flow through the INL Site considering backwater effects that could occur due to diversion dams, roads, and blocked culvert.

HEC-RAS is a one-dimensional model and its setup requires thoughtful assembly of cross-sectional and control structure characteristics of the system. Cross-section data input into HEC-RAS was obtained from LiDAR data. The geometric representation of the MFC site and surrounding areas includes cross sections that characterize the presence of drainage channels, roads, and buildings.

5.2.4 Assessment of Discharge (Flow Rates) and Flood Elevations using Detailed Hydraulic Modeling Techniques

PNNL (2010) estimated the PMF maximum water surface elevations in MFC site. PNNL analyzed MFC and TREAT for the PMF and 10,000-year flood events and the delta between the DBFL and CFE for 13 facilities. However, four additional facility assessments were requested during the review process. Sufficient information had been collected and analyzed to conduct these additional assessments. The additional information for MFC-771, MFC-784, MFC-792, and MFC-744 was provided via personal communication (Skaggs 2010) and was incorporated into this document.

5.2.4.1 MFC Flow Rates and Flood Elevations

The cumulative peak discharges for the PMF and 10,000-year events at the MFC site were 10,200 cfs and 2,880 cfs, respectively. The peak discharges associated with the six MFC subwatershed outflows for the PMF and 10,000-year flood events, under Blocked and Open conditions, are provided in Figures 18 and 19.

The maximum water-level profiles are summarized below and in Table 8, and described in detail in Appendix A. The maximum water level profiles for the PMP and 10,000-year flood events under blocked and open conditions show the hydraulic relationships between the subbasins. The combined profiles of the maximum water level modeled for the PMP is shown in Figure 2.17 of Appendix A. The individual profiles of the maximum water levels for each individual subbasin modeled for the PMP is shown in Figure 2.18 of Appendix A. The combined profiles of the maximum water level modeled for the 10,000-year flood event are shown in Figure 2.25 of Appendix A. The individual profiles of the maximum water levels for each individual basin modeled for the 10,000-year flood events are shown in Figure 2.26.

The maximum water level elevations for the PMF and 10,000-year flood events for each facility assessed at MFC are provided in Table 8.

5.2.4.2 TREAT flow rates and flood elevations

The PMF estimated for runoff at TREAT (MFC-720 and MFC-723) is 2,161 cfs; however, the flow was bracketed (200 to 5,000 cfs) for modeling to provide a conservative estimate. At 5,000 cfs, which is more than double the runoff rate estimated to be generated by the PMP, the maximum surface water elevation at TREAT (MFC-720 and 723) is 5114.82 ft. The TREAT Control Room (MFC-724) was not considered in this study because it is not considered a nuclear facility at this time.

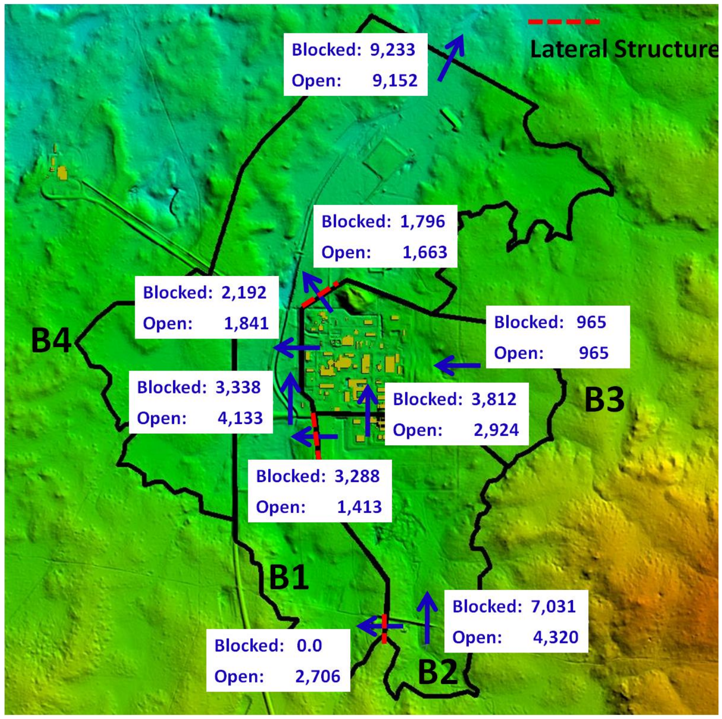

Figure 18. Flows paths and maximum flows (cfs) for MFC PMF; Blocked and Open cases.

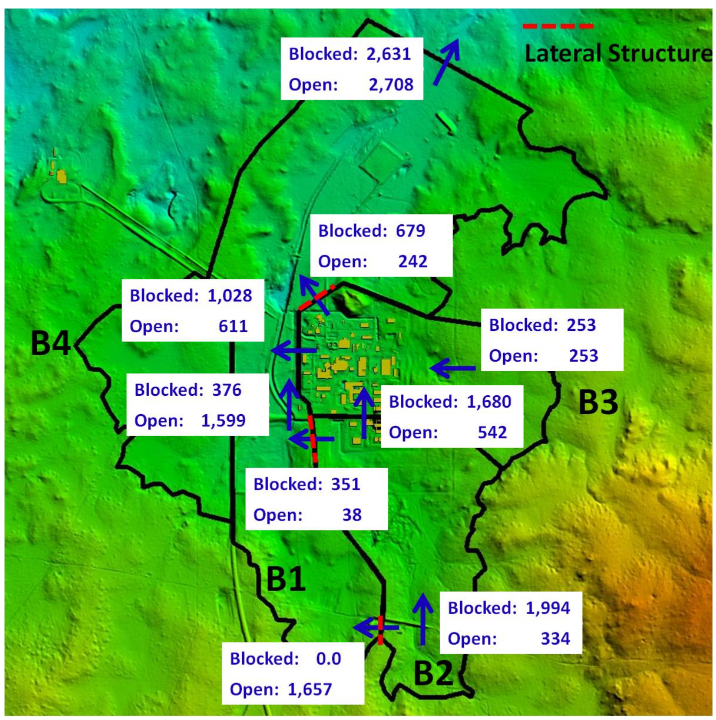

Figure 19. Flows paths and maximum flows (cfs) for the MFC 10,000-year return period event for Blocked and Open cases.

Table 8. Comparison of the floor elevations of buildings at the MFC site and the maximum water-surface elevations obtained from the HEC-RAS Model. Maximum water-surface elevations highlighted indicate the value exceeds the flood elevation.

Building Number and Name	Location	Finished Floor Elevation[1]	Station ID and Basin	PMF[1]		10,000-yr Return Period[1]		Sensitivity Run (MFC Cross Sections Manning's n = 0.013)[1]
				Blocked	Open	Blocked	Open	10,000-yr Return Period Open
MFC-704 Fuel Manufacturing Facility (FMF)	N. Door	5127.59	291 Basin 3	5126.14	5126.25	5125.53	5125.59	5125.70
MFC-752 Analytical Laboratory and Office Building (AL&OB)	W. Dock	5128.23	288 Basin 3	5125.63	5125.37	5124.91	5124.28	5124.15
	E. Dock	5128.24	289 Basin 3	5125.63	5125.40	5124.91	5124.37	5124.43
	N. Door East Side	5128.23	289 Basin 3	5125.63	5125.40	5124.91	5124.37	5124.43
	S. Door East Side	5126.73	289 Basin 3	5125.63	5125.40	5124.91	5124.38	5124.43
MFC-765 Fuel Conditioning Facility (FCF)	SE Door	5128.23	287 Basin 3	5125.63	5125.37	5124.91	5124.28	5124.15
	W. Bay Door	5126.21	286 Basin 3	5125.55	5125.30	5124.86	5124.26	5124.14

Table 8. (continued).

Building Number and Name	Location	Finished Floor Elevation[1]	Station ID and Basin	PMF[1] Blocked	PMF[1] Open	10,000-yr Return Period[1] Blocked	10,000-yr Return Period[1] Open	Sensitivity Run (MFC Cross Sections Manning's n = 0.013)[1] 10,000-yr Return Period Open
MFC-767 EBR-II Reactor Plant Building	Adj. to Building North	5126.94	285 Basin 3	5125.27	5125.01	5124.61	5124.02	5123.62
	N. Truck Dock	5128.01	285 Basin 3	5125.27	5125.01	5124.61	5124.02	5123.62
	Adj. to Building West	5125.17	285 Basin 3	5125.27	5125.01	5124.61	5124.02	5123.62
MFC-771 Radioactive Scrap Waste Facility (RSWF)	S. corner	5120. 91	484 Basin 1	5122.54	5122.50	5119.93	5119.83	5119.83
	W. corner	5121.64	484 Basin 1	5122.54	5122.50	5119.93	5119.83	5119.83
	N. corner	5120.92	484 Basin 1	5122.54	5122.50	5119.93	5119.83	5119.83
	E. corner	5121.13	484 Basin 1	5122.54	5122.50	5119.93	5119.83	5119.83
	E. side	5120.78	484 Basin 1	5122.54	5122.50	5119.93	5119.83	5119.83

Table 8. (continued).

Building Number and Name	Location	Finished Floor Elevation[1]	Station ID and Basin	PMF[1]		10,000-yr Return Period[1]		Sensitivity Run (MFC Cross Sections Manning's n = 0.013)[1]
				Blocked	Open	Blocked	Open	10,000-yr Return Period Open
MFC-774 ZPPR Support Wing	N. Door	5128.22	390 Basin 2	5131.42	5130.49	5128.91	5127.91	5127.90
MFC-775 ZPPR Vault-Workroom Equipment Room	Inside S. Security Door	5128.74	390 Basin 2	5131.42	5130.49	5128.91	5127.91	5127.90
MFC-776 ZPPR Reactor Cell	Based on 775 El. and Design Drawings	5128.74	389 Basin 2	5131.47	5130.52	5128.90	5127.87	5127.88
MFC-784 Zero Power Physics Reactor (ZPPR) Materials Control Building	E. Side	5128.01	390 Basin 2	5131.42	5130.49	5128.91	5127.91	5127.90
	SW Corner	5128.41	390 Basin 2	5131.42	5130.49	5128.91	5127.91	5127.90
MFC-785 Hot Fuel Examination Facility (HFEF)	S. Main door	5124.18	287 Basin 3	5125.63	5125.40	5124.91	5124.28	5124.15
MFC-786 HFEF Substation	N. Door	5124.23	286 Basin 3	5125.55	5125.30	5124.86	5124.26	5124.14
MFC-787 Fuels and Applied Science Building (FASB)	N. Door	5125.23	287 Basin 3	5125.63	5125.37	5124.91	5124.28	5124.15

Table 8. (continued).

Building Number and Name	Location	Finished Floor Elevation[1]	Station ID and Basin	PMF[1] Blocked	PMF[1] Open	10,000-yr Return Period[1] Blocked	10,000-yr Return Period[1] Open	Sensitivity Run (MFC Cross Sections Manning's n = 0.013)[1] 10,000-yr Return Period Open
MFC-792 Space & Security Power System Facility (SSPSF) Control Room	S. Side	5128.99	392 Basin 2	5131.43	5130.52	5128.98	5127.93	5127.91
	SE Corner	5129.07	392 Basin 2	5131.43	5130.52	5128.98	5127.93	5127.91
MFC-792A Space & Security Power System Facility Annex	N. Side	5130.26	392 Basin 2	5131.43	5130.52	5128.98	5127.93	5127.91
MFC-794 Contaminated Equipment Storage Building (CESB)	NE Corner	5124.76	289 Basin 3	5125.64	5125.40	5124.91	5124.37	5124.43
	SE Corner	5124.51	289 Basin 3	5125.64	5125.40	5124.91	5124.37	5124.43
MFC-798 Radioactive Liquid Waste Treatment Facility (RLWTF)	N. Bay Door	5124.71	288 Basin 3	5125.63	5125.37	5124.91	5124.28	5124.15
MFC-1702 Radiochemistry Laboratory (RCL)	NW Door	5128.20	288 Basin 3	5125.63	5125.37	5124.91	5124.28	5124.15

1. Elevations are in ft. above Mean Sea Level, NAVD88 coordinates.

5.2.5 Estimation of Joint Natural Hazard Events Frequency

A joint hazard event assessment is an assessment of the combination effects of multiple flood events (e.g., riverine flooding, run-on/run-off flooding, and tsunamis). Because overland flooding is the only potential source of flooding that is of concern to MFC and TREAT, a joint hazard event assessment is not required for this assessment.

5.2.6 Comparison of Design Basis Flood (DBFL) versus Critical Flood Elevation (CFE)

Comparison of the DBFL (maximum water-surface elevations) and CFE (floor elevations) of each facility assessed at MFC are provided in Table 8. The HEC-RAS identification (ID) numbers are the cross-section stations that are at or just upstream of the building. Maximum water-surface elevations highlighted indicate the DBFL exceeds the CFE. The Open cases have the Basin 3 culverts and Basin 2 diversion channel open, while the Blocked cases have all culverts and the diversion channel blocked.

5.2.6.1 MFC DBFL versus CFE Assessment

A total of 17 facilities were assessed at MFC. The CFE was higher than the DBFL under all PMF and 10,000-year recurrence flood scenarios at four facilities assessed (MFC-740, FMF; MFC-752, AL&OB; MFC-765, FCF; and MFC-1702, RCL). Modeling at the remaining MFC indicates that 13 facilities may be at risk of potential flooding under one or more circumstances (Table 8). The DBFL exceeds the CFE at some portion of all the remaining 13 facilities for the PMP in the blocked scenario. The DBFL exceeds the CFE at some portion of 12 facilities for the PMP in the open scenario; the exception is MFC-767 (EBR-II Reactor Plant Building). The DBFL exceeds the CFE at some portion of eight facilities for the 10,000-year event in the blocked scenario. These include MFC-774 (ZPPR Support Wing); MFC-775 (ZPPR Vault-Workroom Equipment Room); MFC-776 (ZPPR Reactor Cell); MFC-784 (ZPPR) Materials Control Building); MFC-785 (HFEF); MFC-786 (HFEF Substation); MFC-794 (CESB); and MFC-798 (RLWTF). The DBFL exceeds the CFE at some portion of two facilities for the 10,000-year flood event in the open scenario. These include MFC-785 (HFEF) and MFC-786 (HFEF Substation).

A sensitivity test was conducted to assess the potential causes of the elevations modeled at MFC. The Manning's coefficient, a measure of surface roughness, is considered to be one of the most sensitive factors associated with modeling flood flows. The initial modeling runs, described above, utilized a Manning's coefficient of 0.035, which reflects the sagebrush community surrounding MFC and TREAT. The sensitivity test used a Manning's coefficient of 0.013 within the MFC operational area, which more closely reflects the conditions mixed gravel, asphalt, and lawn conditions within the MFC operational area. The results of this sensitivity test showed a decrease in water levels within the MFC operational area to the extent that the CFE was higher than the DBFL at all facilities assessed at MFC (Table 8).

The delta between the CFE and the DBFL for the PMF, the 10,000-year flood event, and the sensitivity test is provided in Table 9.

The Safety Equipment Building (MFC-709) is a nuclear facility, but was not assessed in this study. A survey point was found approximately 50 ft away from MFC-709 and is 5125.69 ft. Buildings near MFC-709 that were analyzed (MFC-752, MFC-765, MFC-787) all had a PMF height under the closed scenario of 5125.63 ft. The buildings were 146.0, 20.5, and 82.3 ft distant from MFC-709, respectively. These near buildings had a PC-3 flood height under reasonable conditions of 5124.15-5124.43 ft. Inferring from the above information MFC-709 is above the PMF and PC-3 flood heights.

5.2.6.2 TREAT DBFL versus CFE Assessment

Modeling at TREAT (MFC-720 and MFC-723) using the 5,000 cfs upper-bracketed flow indicates that DBFL for the TREAT facilities analyzed is 5114.82 ft above sea level. The CFE elevation for

TREAT is 5121.85 ft, more than 7 ft above the DBFL. Therefore, MFC-720 and MFC-723 at TREAT are not at risk of flooding even under the most conservative assumptions modeled.

Table 9. Delta between Design Basis Floods and Critical Flood Elevations at various MFC facilities. Maximum water-surface elevations highlighted indicate the DBFL exceeds the CFE.

			CFE Higher Than or Lower Than the DBFL (Feet)				
			PMF		10 K		Sensitivity 10 K Open
Building	Survey Location	Building Elevation	Blocked	Open	Blocked	Open	
MFC-767 EBR-II Reactor Plant Building	Building West	5125.17	-0.1	0.16	0.56	1.15	1.55
MFC-771 Radioactive Scrap Waste Facility (RSWF)	S. Corner	5120. 91	-1.63	-1.59	0.98	1.08	1.08
MFC-774 ZPPR Support Wing	N. Door	5128.22	-3.2	-2.27	-0.69	0.31	0.32
MFC-775 ZPPR Vault-Workroom Equipment Room	Inside S. Security Door	5128.74	-2.68	-1.75	-0.17	0.83	0.84
MFC-776 ZPPR Reactor Cell	Drawing Elevations	5128.74	-2.73	-1.78	-0.16	0.87	0.86
MFC-784 Zero Power Physics Reactor (ZPPR) Materials Control Building	E. Side	5128.01	-3.41	-2.48	-0.90	0.10	0.11
MFC-785 Hot Fuel Examination Facility (HFEF)	S. Main door	5124.18	-1.45	-1.22	-0.73	-0.1	0.03
MFC-786 HFEF Substation	N. Door	5124.23	-1.32	-1.07	-0.63	-0.03	0.09
MFC-787 Fuels and Applied Science Building (FASB)	N. Door	5125.23	-0.4	-0.14	0.32	0.95	1.08
MFC-792 Space & Security Power System Facility (SSPSF) Control Room	S. Side	5128.99	-2.44	-1.53	0.01	1.06	1.08
MFC-792A Space & Security Power System Facility (SSPSF) Annex	N. Side	5130.26	-1.17	-0.26	1.28	2.33	2.35
MFC-794 Contaminated Equipment Storage Building (CESB)	SE Corner	5124.51	-1.13	-0.89	-0.4	0.14	0.08
MFC-798 Radioactive Liquid Waste Treatment Facility (RLWTF)	N. Bay Door	5124.71	-0.92	-0.66	-0.2	0.43	0.56

5.2.7 Assessment of the Uncertainty due to the Limited Data

The lack of sufficient spatial and temporal data and the lack of adequate data density are always a source of uncertainty for flood hazard assessments; however, the INL Site has longer and more robust datasets available than most comparable facilities. INL has collected real-time regional climatologic data, which dates back to 1950. The precipitation data collected by NOAA are accurate to $\pm$0.01 in. (Clawson 2010). However, there are un-quantified spatial uncertainties associated with extrapolating point data from one location (CFA) to another (MFC/TREAT) and from point data to spatially extensive data (e.g., from a weather monitoring station to a watershed). It addition, there is uncertainty associated with extrapolating approximately 60 years of local precipitation data into 10,000-year precipitation event and regional data into a PMP event. Clearly this is one of the largest areas of uncertainty associated with this study; therefore, caution should be exercised relative to ascribing too much certainty to the estimated surface water elevations because of the large uncertainty associated with extrapolating these extreme precipitation events.

The GPS surveys were conducted, adjusted, and logged using a Trimble 4800 GPS unit and a Topcon total station. The units were calibrated against 1st Order US Geodetic Survey markers at INL. According to manufacturer's information, the vertical resolution of a properly calibrated Trimble 4800 GPS unit is $\pm$0.5 cm.

Regional surface drainage data were developed using USGS 10-meter vertical resolution digital elevation models and local drainage patterns were refined using LiDAR (Light Detection and Ranging) 1-meter horizontal spatial resolution data. LiDAR is an optical remote sensing technology that measures properties of scattered light to find range and other information of a distant target. Ten-meter digital elevation models are commonly used for regional assessments; and the level granularity was greatly reduced by utilizing 1-meter horizontal spatial resolution LiDAR data. However, even 1-meter LiDAR data introduces a level of uncertainty relative to predicting DBFL elevations.

The LiDAR data was evaluated against the 185 GPS survey points collected as part of this project to assess the level of uncertainty between them. Three obvious outliers were removed from further assessment; therefore, 182 survey points were assessed. Each GPS survey point was compared to the closest LiDAR data point and an average of LiDAR data point clusters around the given survey point. The average delta between the GPS survey points and the closest LiDAR data point was 0.01 ft, and the average delta between the between each GPS survey point and the average of the LiDAR data points clustered around the given survey point was 0.06 ft. The number of LiDAR data points per cluster ranged from 2 to 29 data points per survey point. The average distance between the 182 GPS points and the LiDAR points clustered around each point was 6.65 ft.

Uncertainty associated with the HEC models was assessed utilizing limited sensitivity analyses. Hydraulically, one of the most sensitive parameters to model is surface roughness. Therefore, a sensitivity analysis was conducted to evaluate the maximum water surface elevations as a function of the assumed roughness coefficients used in the hydraulic analysis. The response of the water-surface elevations at MFC was examined for two parameters. First, the downstream boundary condition slope was adjusted to determine if there was a backwater effect produced by the assumed boundary friction value. The results indicated that the MFC stations were insensitive to a change in boundary condition value indicating that the channel properties in the lower end of Basin 1 govern the conveyance of floodwaters through the system.

Second, Manning's roughness coefficient for the overbank portions of the cross section within the MFC site was adjusted. Manning's roughness coefficient provides an estimate for the potential friction or impedance of flow across a given surface. Potentially applicable Manning's coefficient for the MFC operational area and surrounding watershed range from approximately 0.013 for asphalt; 0.25 for bare earth; 0.025, 0.030, and 0.035 gravelly and weedy earth channels, respectively; and 0.035, 0.050, and 0.075 for pasture, light brush, and heavy brush floodplains, respectively (Chow 1959). The coefficient's used for the overbank portions of the cross section within the MFC site were 0.035 and 0.013. These results showed that by reducing the coefficient from 0.035 (representative of lightly vegetated sagebrush) to 0.013 (representative of asphalt) at the MFC cross sections for the 10,000-year precipitation event decreased the maximum water-surface elevations to levels below all floor elevations.

Finally, the assumption that the IWP would be filled by modeling it as part of the conveyance area was tested. The results for this assessment were also found to be insensitive to their treatment in the model; specifically, the pond is adequate for collecting the volumes of runoff modeled without causing significant backwater effects.

5.2.8 Reliability of Flood Protection Devices

The primary storm water and flood management structures are the MFC Diversion Dam, ditches, and canals. This study is not intended to provide an engineering analysis of these structures. However, because of the structural failure of the diversion dam during a 1979 flooding event, it is critical to ensure to the extent possible that the structure was properly repaired and capable of performing properly during future flooding events.

The MFC Diversion Dam was constructed in 1968 in response to the 1963 flood event at MFC (see above). The purpose was "to improve runoff water provisions, during to the marginal capacity of the existing channel and the sharp bend at the bridge over Buchanan Boulevard, which restricted flow and caused ice to accumulate" (ANL 1979). The structure was constructed using native rock and soil from the surrounding area. The earthen structure was originally constructed without an outlet; the structure was later excavated and the outlet was constructed. However, during a flood event in 1969, the soil around the outlet culvert eroded allowing floodwaters to flow around the culvert and directly into the MFC facility. Correspondence stated that "Fortunately, the quantity of water and rate of release were not sufficient to result in significant damage to ANL-W" (ANL 1979). Repairs were later made to the structure after the 1969 failure; however, questions were raised later relative to the adequacy of those repairs (ANL 1979).

In 1979, engineering evaluations were conducted and recommendations were made for repairing the structure (NTL 1979). The recommendations included removing the soil surrounding the culvert from the basalt upward on a 1:1 slope from 4 ft on either side of the culvert to the crest; constructing cutoff collars around the culvert, properly backfilling and compacting the excavation under proper engineering supervision in accordance with American Society for Testing and Materials (ASTM) D698, keying in the cut to reduce seepage planes, and placing 12 in. of 12 minus riprap on the upstream side of the dam. In 1980, construction was authorized (EG&G 1980) to remove the soil surrounding the culvert; constructing a concrete head wall (wing wall) around the face of the culvert; properly backfilling and compacting the soil; and riprapping the upstream side of the dam. The upgrade project was implemented May–June 1980 under the supervision of a licensed engineer (ANL 1980).

The most recent inspections of the Diversion Dam were conducted in July and August 2010 (Portage 2010). The inspection report indicates that in general the diversion dam is in good condition. There were no signs of embankment instability were found during this inspection. Much of the upstream embankment riprap is sound; however, a bench was noted near the left (west) end of the embankment. There is no indication of recent erosion, sloughing, or settlement on the downstream side of the structure. The diversion crest shows some minor rutting from vehicle traffic, but otherwise shows no evidence of potential areas for ponding. Rodent and other burrows were found on the embankment. The embankments were generally covered with sparse shallow-rooted plants and grasses, although some deeper rooting woody plant species (i.e., sagebrush and rabbit brush) were observed on both embankments.

No obstructions were observed at the spillway entrance, and the spillway shows no evidence of erosion. The mechanical sluice gate was well maintained and operated freely within its full range of operations. Debris was observed in the wet well next to the gate. The outlet pipe was generally in good condition, with no evidence of corrosion and few deformities. Some sediment was noted to be present at the bottom of the pipe throughout its entire length. The galvanized steel trash rack on the upstream side showed very little evidence of corrosion; however, grass and brush were present at the mouth of the trash rack, somewhat obscuring the entrance.

5.3 Historical Flood Check

MFC's historical records were reviewed and it was determined that the largest known historical flood to occur during MFC operations was approximately a 1-in. precipitation event that occurred in 1963. This flood was caused by precipitation runoff over frozen ground. Its primary impact was to site access roads. A 1969 flood event damaged the MFC Diversion Dam which was designed to contain floodwaters, but it apparently had negligible impact on the facilities assessed at MFC. The PMF (6-hour total rainfall of 9.12 in., with approximately 3.80 in falling in the first 15 minutes) and 10,000-year flood evaluations (2.2 in. of total rainfall falling over the first hour, with approximately 4.65 in. falling in the 24 hours) conducted herein utilized more conservative data and assumptions than the flood conditions experienced during those flood events. Therefore, this assessment is sufficiently conservative for estimating a natural phenomenon hazard flood at MFC.

5.4 Quality Assurance and Peer Review

A formal quality level determination was conducted with INL QA personnel per LWP-13014, "Determining Quality Levels." It was determined that this is a Quality Level 3 project, as documented in Quality Level Determination (QLD) Number MFC-001013.

The modeling and hydrologic/hydraulic analyses were conducted in accordance with the applicable standards of DOE Order 414.1C, "Quality Assurance," and 10 CFR 830, Subpart A, "Quality Assurance Requirements." USACE's HEC-HMS and HEC-RAS models are industry standards for conducting flood risk analyses, both nationally and internationally. Modeling was conducted in accordance with the appropriate HEC user's guidelines (e.g., "Hydrologic Modeling System Validation Guide, Version 3.5, August 2010; http://www.hec.usace.army.mil/software/hec-hms/documentation.html).

Independent Peer reviews of this assessment were conducted by qualified INL and PNNL hydrologists who were not members of modeling, hydrologic/hydraulic analysis, or report writing teams. Their comments were incorporated as appropriate.

6. CONCLUSIONS

Based on the performance criteria in DOE-STD-1020-2002, MFC and TREAT were analyzed to determine if they meet DOE's NPH flood hazard criteria for PC 3 SSCs (mean hazard annual probability of 1×10^{-4} or a 10,000-year recurrence). However, this study was conducted using a graded approach; therefore, MFC and TREAT were also assessed for the potential impacts of a PMF event.

Using the graded approach, each facility was first assessed using the largest potential flood event (PMF) and the most conservative assumptions (the MFC flood control system was blocked or non-operational and a Manning's coefficient of 0.035) and then conducting more rigorous evaluations using an iterative process to more rigorously assess the facilities, thereby reducing the conservative assumptions. If the facilities assessed within the given facility did not meet the appropriate performance criteria, the assumptions were relaxed using hydrologically reasonable and prudent, but less-conservative assumptions. It is assumed that if a facility meets DOE's flood hazard performance criteria for a larger flood event (e.g., a PMF), it will also meet the criteria for a smaller (e.g., 10,000-year) event. Therefore, both facilities were assessed until all the facilities assessed within them were estimated to meet DOE's NPH flood hazard criteria and then assessments were discontinued at that facility.

The DBFLs were estimated for the PMF at 17 facilities (SSCs) within the MFC operational area and for TREAT, and for the 10,000-year flood event at the 17 facilities within the MFC operational area. Because of its relatively small size, TREAT was assessed as a single SSC. Modeling was conducted using the USACE HEC-HMS and HEC-RAS software. The DBFLs were compared with the CFEs for at both facilities. The results of the MFC and TREAT flood hazard assessments are discussed below.

6.1 Flood Hazard Assessment Results for MFC

The PMF and 10,000-year (1×10^{-4} exceedance) flood events were analyzed for MFC. The peak discharges for the PMF and 10,000-year events at the MFC site were 10,200 cfs and 2,880 cfs, respectively. Comparison of the maximum water-surface elevation results from the HEC-RAS model (DBFL) to floor elevations (CFE) at nuclear facilities is discussed below.

6.1.1 MFC PMF Results

Modeling MFC using the most conservative PMF assumptions (i.e., the diversion channel and culverts were blocked and a Manning's coefficient of 0.035) indicates that the CFE is higher than the DBFL (meets DOE's flood hazard performance criteria) for a PMF under both blocked and open conditions at four facilities: MFC-740, FMF; MFC-752, AL&OB; MFC-765, FCF; and MFC-1702, RCL (Table 8). The DBFL would exceed the CFE at some portion of the remaining 13 facilities assessed. The DBFL exceeded the CFE at these facilities under the blocked scenario by a range of 0.1 to 3.2 ft (Table 9). The delta is greater than 1 ft at MFC-771 (1.63 ft), MCF-774 (3.20 ft), MFC-775 (2.86 ft), MFC-776 (2.73 ft), MFC-785 (1.45 ft), MFC-786 (1.32 ft), MFC-792 (2.44 ft) and MFC-792A (1.17 ft). The delta is less than 1 ft at MFC-767 (0.10 ft), MFC-787 (0.40 ft), and MFC-798 (0.92 ft).

At facilities where the DBFL was greater than the CFE, the facilities were modeled again using less conservative PMF assumptions (i.e., the diversion channel and culverts open and a Manning's coefficient of 0.035). The modeling indicates that the DBFL exceeds the CFE at some portion of 11 facilities for the PMP. The exception is MFC-767 where the CFE is 0.16 ft above the DBFL. The DBFL exceeded the CFE at the remaining facilities by a range of 0.26 to 2.48 ft (Table 9). The DBFL exceeds the CFE by more than 1 ft at MFC-771 (1.59 ft), MCF-774 (2.27 ft), MFC-775 (1.75 ft), MFC-776 (1.78 ft), MFC-784 (2.48 ft), MFC-785 (1.22 ft), MFC-786 (1.07 ft) and MFC-792 (1.53 ft). The delta is less than 1 ft for MFC-787 (0.14 ft), MFC-792A (0.26 ft), MFC-794 (0.89 ft) and MFC-798 (0.66 ft).

6.1.2 MFC 10,000-year Flood Event Results

At facilities where the DBFL was greater than the CFE for the less conservative PMF, the facilities were modeled again using the most conservative 10,000-year event assumptions (i.e., the diversion channel and the culverts blocked and a Manning's coefficient of 0.035). The modeling indicates that the CFE is higher than the DBFL (meets DOE's flood hazard performance criteria) for a 10,000-year flood event at five additional facilities: MFC-767, MFC771, MFC-787, MFC-792, and MFC-792A (Table 9). The delta between the DBFL and the CFE for the remaining facilities ranges from 0.16 to 0.90 ft. The DBFL exceeds the CFE at some portion of eight facilities by less than 1 ft: MFC-774 (0.69 ft), MFC-775 (0.17 ft), MFC-776 (0.16 ft), MFC-784 (0.90 ft), MFC-785 (0.73 ft), MFC-786 (0.63 ft), MFC-794 (0.40), and MFC-798 (0.20 ft).

At facilities where the DBFL was greater than the CFE for the most conservative 10,000-year event, the facilities were modeled again using less conservative 10,000-year event assumptions (i.e., the diversion channel and the culverts open and a Manning's coefficient of 0.035). The modeling indicates that the CFE is higher than the DBFL (meets DOE's flood hazard performance criteria) for a 10,000-year flood event at six additional facilities: MFC-774, MFC-775, MFC-776, MFC-784, MFC-794, and MFC-798 (Table 9). The DBFL exceeds the CFE at some portion of two facilities: MFC-785 (0.1 ft) and MFC-786 (0.03 ft).

Modeling MFC using the even less conservative, but reasonable and prudent 10,000-year event assumptions (i.e., the diversion channel and the culverts open and a Manning's coefficient of 0.013) indicates that the CFE is higher than the DBFL (meets DOE's flood hazard performance criteria) at all facilities assessed at MFC.

6.1.3 Controlling Factors

It should be noted that the changes in maximum water-surface elevations at these two locations for the two events and different assumptions were relatively small. For example, for the PMF with the diversion channel and culverts blocked the maximum water-surface elevation at MFC-785 was 5125.63 ft above mean sea level. For the 10,000-year event with the diversion channel and culverts open, the maximum water-surface elevation was only reduced 1.35 ft. Similarly, at MFC-786 the reduction was only 1.29 ft. This strongly suggests that maximum water-surface elevations are primarily determined by backwater effects from downstream hydraulic controls. An example is the road in the northwest quadrant of the MFC site that is overtopped and produces backwater effects at the MFC buildings during both the PMF and 10,000-year-events.

6.2 Flood Hazard Assessment Results for TREAT

The peak discharge for the PMF at TREAT (MFC-720 and MFC-723) was 2,161 cfs; however, the flow was conservatively bracketed between 200 to 5,000 cfs. Using the most conservative assumptions (5,000 cfs is more than double the estimated discharge) the PMF DBFL estimated for TREAT was 5114.82 ft above mean sea level, nearly 7 ft below the CFE elevation for TREAT (5121.85 ft). Therefore, based on this assessment, TREAT (MFC-720 and 723) is not at risk of flooding during a PMF event.

6.3 Conclusions

Seventeen nuclear or nuclear related facilities at MFC and 2 facilities at TREAT (modeled as one facility) were assessed to determine if they meet DOE NPH flood hazard performance criteria per DOE-STD-1020-2002 and DOE-STD-1023-1995). The results indicate that:

- Four MFC facilities (MFC-704, MFC-752, MFC-765, and MFC-1702) meet DOE's flood hazard performance criteria for a PMF under both scenarios modeled; one additional facility (MFC-767) meets the PMF criteria under the open scenario; and 12 facilities (MFC-771, MCF-774, MFC-775,

MFC-776, MFC-784, MFC-785, MFC-786, MFC-787, MFC-792, MFC-792A, MFC-794, and MFC-798) do not meet the PMF criteria under either the blocked or open scenarios.

- TREAT (MFC_720 and MFC-723) meets DOE's flood hazard performance criteria for a PMF and is not at risk of flooding during a PMF event.
- Of the 12 facilities assessed at MFC did not meet DOE's flood hazard performance criteria for a PMF, four meet the 10,000-year criteria under the most conservative assumptions (MFC-771, MFC-787, MFC-792, and MFC-792A); six additional facilities that did not meet DOE's flood hazard performance criteria for the most conservative 10,000-year scenario met the 10,000-year criteria under less-conservative assumptions (MCF-774, MFC-775, MFC-776, MFC-784, MFC-794, and MFC-798); and all facilities assessed at MFC met the DOE's flood hazard performance criteria for a 10,000-year flood event under the more realistic, but reasonable and prudent assumptions.

Therefore, it can be concluded that all facilities assessed at MFC and TREAT meet the appropriate DOE flood hazard performance criteria (i.e., a mean hazard annual probability of 1×10^{-4} or a 10,000-year reoccurrence).

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Appendix A

Analysis of Flood Hazards for the Materials and Fuels Complex at the Idaho National Laboratory Site

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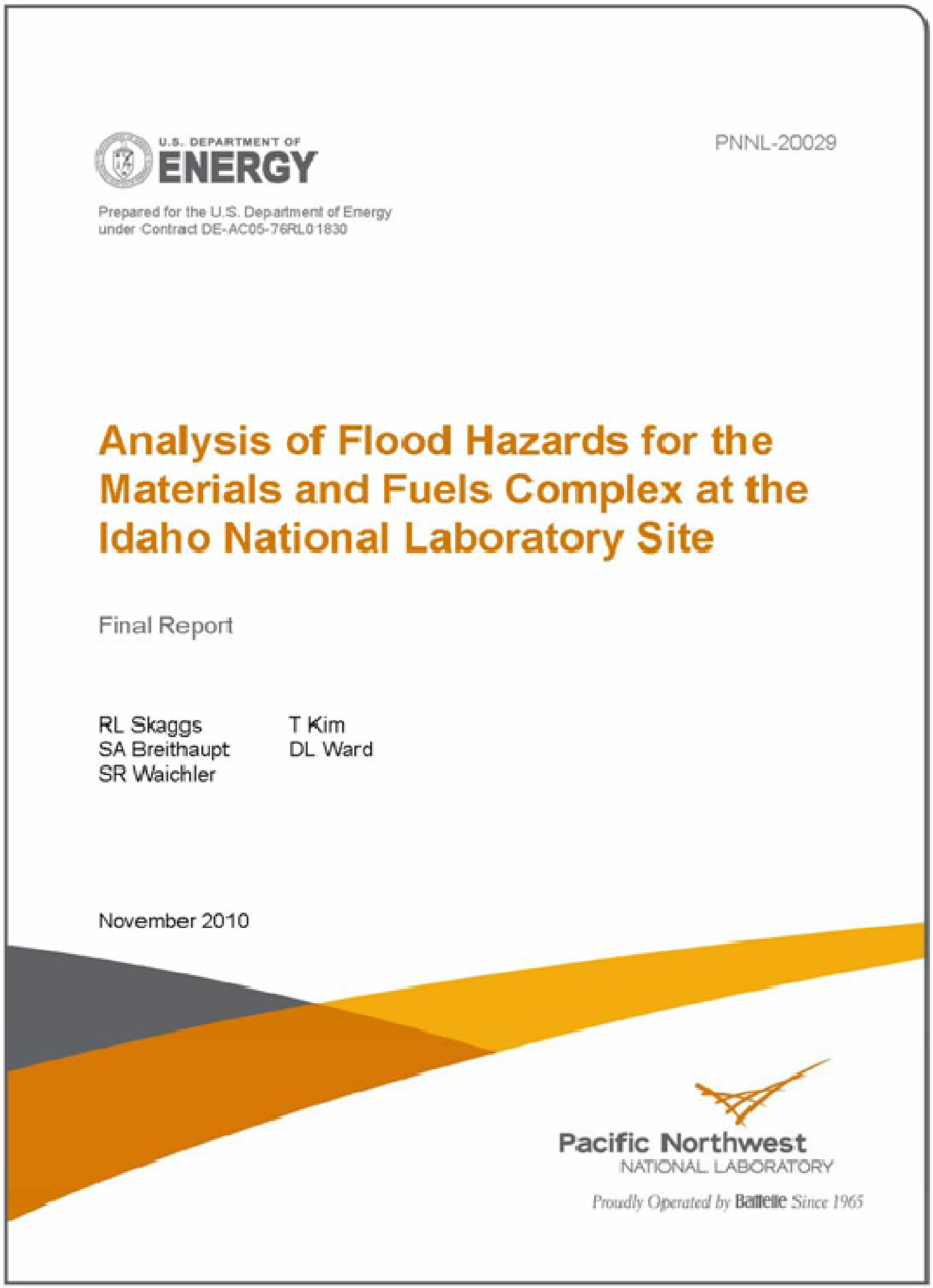
Analysis of Flood Hazards for the Materials and Fuels Complex at the Idaho National Laboratory Site



U.S. DEPARTMENT OF ENERGY

PNNL-20029

Prepared for the U.S. Department of Energy
under Contract DE-AC05-76RL01830

Analysis of Flood Hazards for the Materials and Fuels Complex at the Idaho National Laboratory Site

Final Report

RL Skaggs
SA Breithaupt
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November 2010

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Pacific Northwest National Laboratory
Richland, Washington 99352

Summary

Researchers at Pacific Northwest National Laboratory conducted a flood hazard analysis for the Materials and Fuels Complex (MFC) site located at the Idaho National Laboratory (INL) site in southeastern Idaho. The general approach for the analysis was to determine the maximum water elevation levels associated with the design-basis flood (DBFL) and compare them to the floor elevations at critical building locations. Two DBFLs for the MFC site were developed using different precipitation inputs: probable maximum precipitation (PMP) and 10,000-year recurrence interval precipitation. Both precipitation inputs were used to drive a watershed runoff model for the surrounding upland basins and the MFC site. Outflows modeled with the Hydrologic Engineering Centers Hydrologic Modeling System were input to the Hydrologic Engineering Centers River Analysis System hydrodynamic flood routing model.

Using the most conservative assumptions for the PMF (i.e., all culverts at the MFC and the diversion ditch located upstream of the MFC are blocked) produced flood levels exceeding floor elevations at eight locations ranging from 3.20 ft at MFC Building 774 (ZPPR Support Wing) to 0.1 ft at MFC Building 767 (EBR-II Reactor Plant Building). The flood resulting from the 10,000-year precipitation event, assuming the culverts and the diversion ditch were open (i.e., unblocked), exceeded floor elevations at two locations—the MFC Building 785 (Hot Fuel Examination Facility) by 0.1 ft. and MFC Building 786 (Hot Fuel Examination Facility substation) by 0.03 ft.

To provide additional perspective on the relative significance of the results obtained, a limited sensitivity analysis was conducted on the hydraulic analysis evaluating the change in maximum water surface as a function of the assumed roughness coefficients used in the hydraulic analysis. These results showed that reducing the Manning's roughness coefficient from 0.035 (representative of lightly vegetated sagebrush) to 0.013 (representative of asphalt) at the MFC cross sections for the 10,000-year precipitation event decreased the maximum water-surface elevations to levels below all floor elevations.

An analysis was also conducted for the Transient Reactor Experiment and Test (TREAT) Facility site, located in a separate drainage approximately 4700 ft northwest of the MFC. Results indicate that flows generated by the PMP will produce a maximum water-surface elevation at the TREAT site of only 5114.82 ft, approximately 7 ft below the floor elevation of the TREAT Warehouse (MFC Building 723) and over 9 ft below the floor elevation of the TREAT reactor building (MFC Building 720).

Acronyms and Abbreviations

ANL-W	Argonne National Laboratory – West (previous designation of the MFC site)
B	basin
CFA	Central Facilities Area
CFE	critical flood elevation
cfs	cubic feet per second
DBFL	design-basis flood
DEM	Digital Elevation Model
DOE	Department of Energy
EPA	Environmental Protection Agency
ESRP	Eastern Snake River Plain
ft	foot(feet)
GIS	geographic information system
HEC-HMS	Hydrologic Engineering Centers Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Centers River Analysis System
HMR	Hydrometeorological Report
hr	hour(s)
in.	inch(es)
INL	Idaho National Laboratory
LiDAR	Light Detection and Ranging
MFC	Materials and Fuels Complex
mi	mile(s)
mi^2	square mile(s)
min	minute(s)
NAVD	North American Vertical Datum
NGVD	National Geodetic Vertical Datum
NOAA	National Ocean Atmospheric Administration
PC	Performance Category
PMF	probable maximum flood
PMP	probable maximum precipitation
PNNL	Pacific Northwest National Laboratory
SCS	Soil Conservation Service
SSCs	structures, systems, and components
STP	Sewage Treatment Plant
TREAT	Transient Reactor Experiment and Test Facility
UH	unit hydrograph
USACE	U.S. Army Corps of Engineers

USGS	U.S. Geological Survey
W	watershed
yr	year(s)

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1.0 Introduction

U.S. Department of Energy (DOE) Order 420.1B (DOE 2005) establishes facility and programmatic safety requirements for nuclear and explosives safety design criteria, fire protection, criticality safety, the mitigation of natural phenomena hazards at nuclear facilities, including the Idaho National Laboratory (INL). DOE's INL, in southeastern Idaho, encompasses nine major operational areas, including the Materials and Fuels Complex (MFC). Located on 60 acres in the southeastern corner of the INL site, the MFC is largely devoted to research and development of nuclear technologies and nuclear environmental management. In partial fulfillment of the requirements of DOE Order 420.1 B, INL directed Pacific Northwest National Laboratory (PNNL) to conduct an assessment of the potential for flooding at the MFC.

1.1 Purpose and Scope

An INL review of the MFC operations and facilities in accordance with DOE Order 420.1B determined the highest Performance Category (PC) for existing facilities at MFC is PC-3. Further, MFC contains several facilities that could be affected by natural phenomena hazards, including flooding. Consequently, in accordance with DOE-STD-1020-2002, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* (DOE 2002), an evaluation of the flood design basis for the structures, systems, and components (SSCs) at the MFC is required. This consists of the following:

- determining the design-basis flood (DBFL) for each flood hazard as defined by the hazard annual probability of exceedance and applicable combinations of flood hazards
- evaluating the site stormwater management system (e.g., site runoff and drainage, roof drainage)
- developing a flood design strategy for the DBFL that satisfies the criteria performance goals (e.g., build above the DBFL, harden the facility)
- designing civil engineering systems (e.g., buildings, buried structures, site drainage, retaining walls, dike slopes, etc.) to the applicable DBFL and design requirements.

In partial fulfillment of these requirements, the study presented here is limited to providing the hydrologic inputs necessary for estimating the probability of flood water inundating structures, systems, and components (SSCs) at MFC (i.e., peak flood elevations exceeding critical flood elevations (CFEs).

1.2 Facility Description

Most of the INL site is located in Pioneer Basin—a closed topographic depression. Portions of six watersheds either drain surface water to or from the site (Figures 1.1 and 1.2). The MFC (or, as it was once known, ANL-W) is in a closed basin, located in the American Falls watershed, which generally drains from the INL site to the Snake River. The MFC includes the Transient Reactor Experimental and Test (TREAT) Facility located approximately 4700 ft from the primary MFC site, but within a separate subwatershed. In 1963, the ANL-W reached an unanticipated flood level, which may have prompted the construction of a diversion dam that still functions at the MFC today. A second flood event in January 1969 involved rain and snow melt runoff over frozen ground; water levels reached and overtopped US Highway 20, south of the MFC.

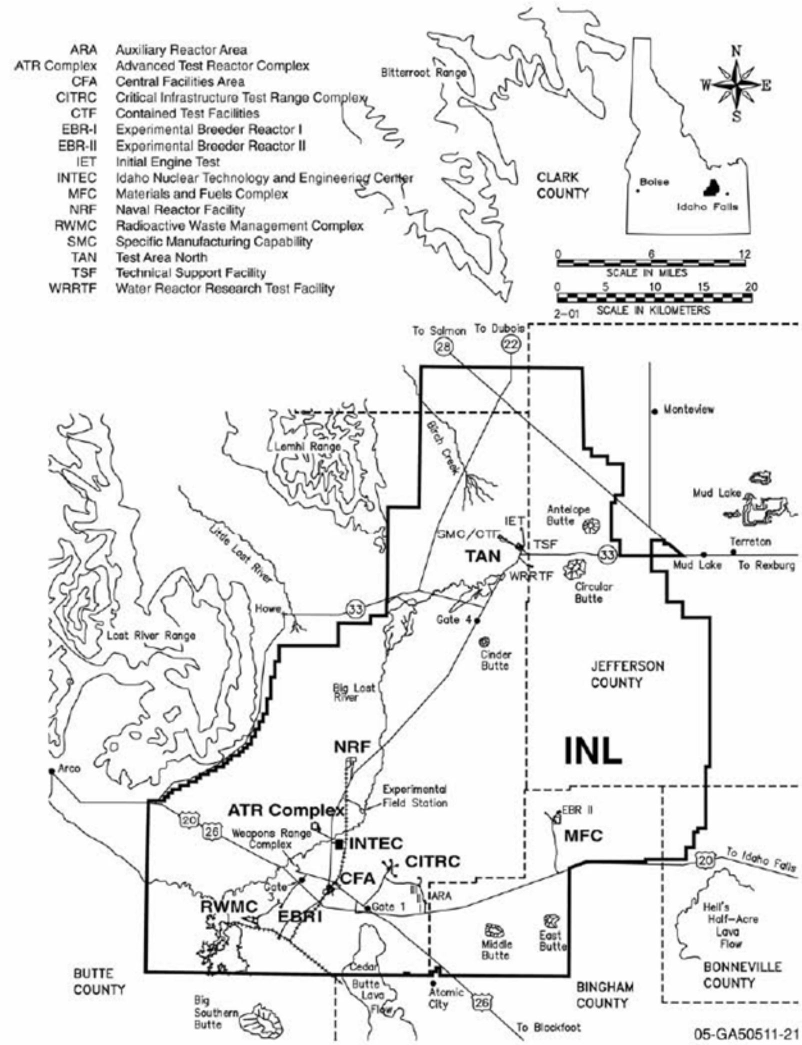

Figure 1.1. Map of the INL Facilities

1.2

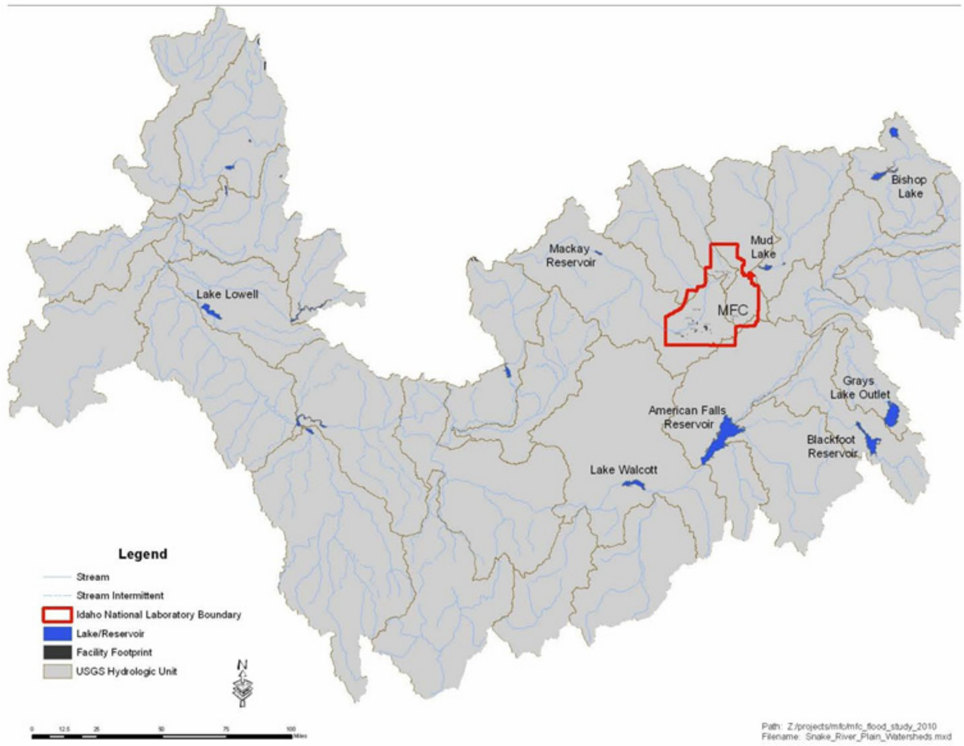

Figure 1.2. Watersheds at the INL Site

Although the MFC site is not affected by flows from other major watersheds in the region, it can be subject to runoff from six upgradient subbasins and from overland flow, ponding, and localized runoff from roofs and other impermeable surfaces located at the facility. The total upgradient contributing watershed area is 7.8 mi^2. During times of high precipitation, runoff can occur as sheet flow or channelized flow flowing downgradient toward the lowest point in the basin. Based on the DOE 420.1B requirements, runoff/drainage (overland flow) is the only potential type of flooding event that needs to be considered at MFC. So, each of the subbasins that can contribute potential flood flows to MFC must be assessed to determine the potential for overland flow flooding to affect SSCs and human health and safety at MFC. In response to INL's request, PNNL researchers conducted a comprehensive flood hazard assessment to evaluate the potential flood risks to MFC SSCs and site personnel. The SSCs are listed in Table 1.1.

1.3 Report Contents and Organization

The ensuing sections of this report describe the flood hazards analysis performed to evaluate the flood design basis for SSCs at the MFC as specified in DOE Order and Standards. Section 2.0 describes the methodology and results of the flood hazard and sensitivity analyses for the MFC and the flood hazard analysis for the nearby TREAT Facility. Section 3.0 presents conclusions. Appendixes A and B contain supplemental climate data plots and terrain data processing information, respectively.

Table 1.1. Example Structures, Systems, and Components at MFC

MFC-704	Fuel Manufacture Facility (FMF)
MFC-719	Vehicle Entry Post
MFC-720	TREAT Reactor Building
MFC-723	TREAT Warehouse
MFC-752	Laboratory and Office Building
MFC-752	Laboratory and Office Building
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MFC-752	Laboratory and Office Building
MFC-765	Fuel Conditioning Facility (FCF)
MFC-765	Fuel Conditioning Facility (FCF)
MFC-767	EBR-II Reactor Plant Building
MFC-767	EBR-II Reactor Plant Building
MFC-767	EBR-II Reactor Plant Building
MFC-774	ZPPR Support Wing
MFC-775	ZPPR Vault-Workroom Equipment Room
MFC-776	ZPPR Reactor Cell
MFC-785	Hot Fuel Examination Facility (HFEF)
MFC-786	HFEF Substation
MFC-787	Fuels and Applied Science Building (FASB)
MFC-792	SSPSF Control Room
MFC-792A	Space and Security Power System Facility Annex
MFC-794	Contaminated Equipment Storage Building
MFC-794	Contaminated Equipment Storage Building
MFC-798	Radioactive Liquid Waste Treatment Facility
MFC-1702	Radiochemistry Laboratory (RCL)

2.0 Analysis and Results

The MFC contains PC-0, 1, 2 and 3 facilities. Therefore, according to DOE-STD-1020-2002, the site stormwater and flood management system must ensure protection against a mean flood hazard probability equal to or less than 1×10^{-4}. The DBFLs for the MFC were based on estimating the probable maximum precipitation (PMP) and 10,000-year precipitation events and then using them as input to a watershed runoff model for the surrounding upland basins as well as the MFC site itself. Runoff, or outflows, were modeled using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Centers Hydrologic Modeling System (HEC-HMS) and Hydrologic Engineering Centers River Analysis System (HEC-RAS) software. Simulated hydrographs for the two events from HEC-HMS were input to the HEC-RAS hydraulic flood routing model to determine peak water-surface elevations at the MFC SSCs.

The details of the flood hazard analysis for the MFC using the described approach is presented in the following sections in three parts:

- determination of the PMP and 10-000-year storm
- determination of the probable maximum flood (PMF) and 10,000-year flood
- hydraulic analysis and determination of the maximum water-surface elevation.

2.1 Determination of the Probable Maximum Precipitation and 10,000-Year Storm

Two DBFLs for the MFC site were developed using different precipitation inputs: PMP and 10,000-year precipitation. Both precipitation events were used as input to a watershed runoff model for the surrounding upland basins and the MFC site itself. Runoff, or outflows, from watersheds W1–W6 (Figure 2.1 and Table 2.1) were modeled using the USACE's HEC-HMS software. Runoff from subbasins designated as B1 through B4 (as shown in Figure 2.1) was estimated using the Rational Method within HEC-RAS. Outflows modeled with HEC-HMS were input to the HEC-RAS hydrodynamic flood routing model. Each of the major elements of the analyses and associated results were reviewed by the project team and technical reviewers for reasonableness and accuracy.

The PMP was estimated using the guidance provided in Hydrometeorological Report (HMR) No. 57 *Probable Maximum Precipitation – Pacific Northwest States* (NWS 1994). PMP is defined as "theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year." The PMP is the potential rainfall that could result from optimal atmospheric conditions and circumstances; it represents an upper limit for a particular duration and area, and is "not a quantity that is expected to be observed." HMR 57 provides background and methods for both general and local storms. General storms are defined as major synoptic events that produce precipitation over areas of at least 500 mi^2 and for durations that often exceed 6 hours. Local storms are defined as having areas of up to 500 mi^2 and durations up to 6 hours. Climate data indicate that both types of storms can occur during any season in the Pacific Northwest, but general storms are less common during the summer months, and local storms primarily occur from April through October. For watersheds less than 10 mi^2 in area, HMR 57 recommends that both general and local PMPs be considered for use. The watershed containing the MFC site is 7.8 mi^2 in area.

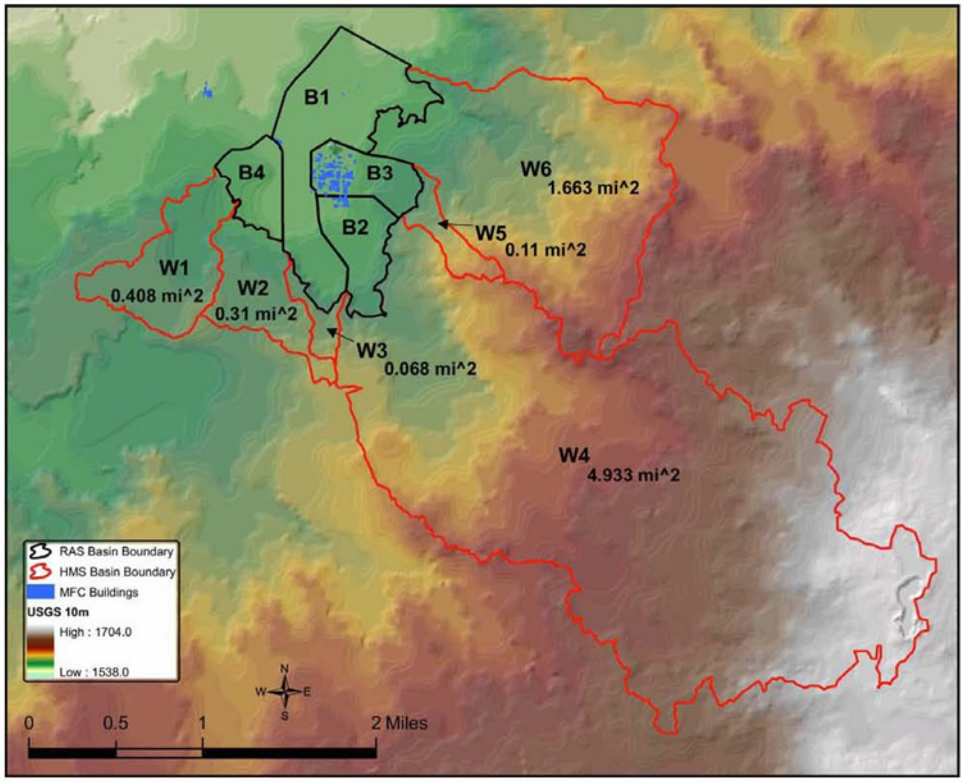


Figure 2.1. Watersheds (W) Simulated with HEC-HMS and Subbasins (B) Simulated with HEC-RAS

The starting point for the general storm PMP is the all-season 10-mi^2, 24-hour PMP index value. The index value is multiplied by factors that account for season, storm duration, and basin size. The index value was 8.5 in. for the INL location. Scripts written in the R language were used to implement the HMR 57 PMP computations. The general storm PMP values for the unique seasons and for durations of 1, 6, 24, 48, and 72 hours are plotted as points in Figure 2.2. The all-season PMP is by definition the maximum of all the seasons, which for much of the Pacific Northwest east of the Cascades occurs in June. For convenience in deriving incremental precipitation for the design storm, a semi-log model was fit to the points using simple linear regression: PMP = a * log(duration) + b, where a and b are the slope and intercept, respectively. The differences between successive hourly cumulative precipitation values from the curve were calculated and arranged in a pattern like that of Figure 2.3 for the design storm. The total

72-hour, all-season precipitation was 10.97 in., and the maximum hourly increment was 1.54 in. The winter months, November to March, have a somewhat lower general storm PMP, 8.78 in. over 72 hours, and a maximum hourly increment of 1.23 in.

Table 2.1. Watershed and Unit Hydrograph Parameters

	W1	W2	W3	W4	W5	W6
Area (mi^2)	0.408	0.310	0.068	4.933	0.110	1.663
Volume in acre-feet of 1 in. rainfall over basin	21.760	16.533	3.627	263.093	5.867	88.693
Mean basin slope (%)	1.664	2.016	2.867	2.762	2.868	2.911
Main channel length (mi)	1.580	0.978	0.363	7.702	0.762	2.199
Mean main channel slope (%)	0.498	1.104	2.067	1.337	2.171	1.283
Weighted main channel slope (%)	0.415	0.845	1.918	0.716	2.029	1.127
Main channel length to centroid (mi)	0.801	0.376	0.054	2.589	0.313	1.008
SCS lag (hr)	0.982	0.608	0.231	2.708	0.418	0.968
SCS time base (hr), 1 hr rain, triangular	3.958	2.959	1.951	8.565	2.450	3.919
SCS time base (hr), 15 min rain, triangular	2.957	1.958	0.950	7.563	1.449	2.918
SCS time of rise (hr), 1 hr rain	1.482	1.108	0.731	3.208	0.918	1.468
SCS peak flow (cfs), 1 hr rain, triangular	133.0	135.2	45.0	743.4	57.9	547.7
SCS peak flow (cfs), 1 hr rain, curved	170.8	167.9	43.8	860.7	66.8	702.8
SCS time of rise (hr), 15 min rain	1.107	0.733	0.356	2.833	0.543	1.093
SCS peak flow (cfs), 15 min rain, triangular	178.1	204.4	92.4	841.8	98.0	735.7
SCS peak flow (cfs), 15 min rain, curved	197.5	237.4	116.4	875.2	118.4	816.2

The local storm PMP process was similar, with a starting index value of 8.7 in. for a 1-mi^2 area below 6000-ft elevation over 1 hour. The PMP values 0.25, 0.5, 0.75, 1, 2, 3, 4, 5, and 6 hours are shown in Figure 2.4. An exponential model of the form PMP = a * duration b + c, where a, b, c are coefficients, fit using nonlinear least-squares regression, was used to derive the 15-minute incremental precipitation over 6 hours. The lower part of Figure 2.4 shows the 15-minute intervals in the design storm sequence from high to low, as recommended by HMR 57. The total 6-hour precipitation in the local storm was 9.12 in., with the first 15-minute increment being the maximum at 3.80 in.

The local storm PMP was selected as the primary scenario for the PMF analysis because of its much higher intensity and likely greater peak flows compared to the general storm. However, the watershed runoff from the general storm, with and without a hypothetical snowmelt event occurring concurrently, was also simulated for comparison purposes.

Climate data from the Central Facilities Area (CFA; labeled in Figure 1.1) and MFC meteorological stations were obtained from the National Ocean Atmospheric Administration (NOAA) and evaluated for this report. The CFA period of record was from March 1, 1950 to April 30, 2010—60 years—and contained daily records of precipitation (in.), snowfall (in.), and snowpack thickness (in.). The maximum daily precipitation was 1.64 in. The MFC period of record was from April 1, 1993 to May 24, 2010 (17 years) and contained daily precipitation (in.); the maximum was 1.48 in. The rainfall maximums at these two stations occurred during June and July, respectively. Plots of precipitation at both stations, and snowfall and snow depth at CFA, can be found in Appendix A of this report.

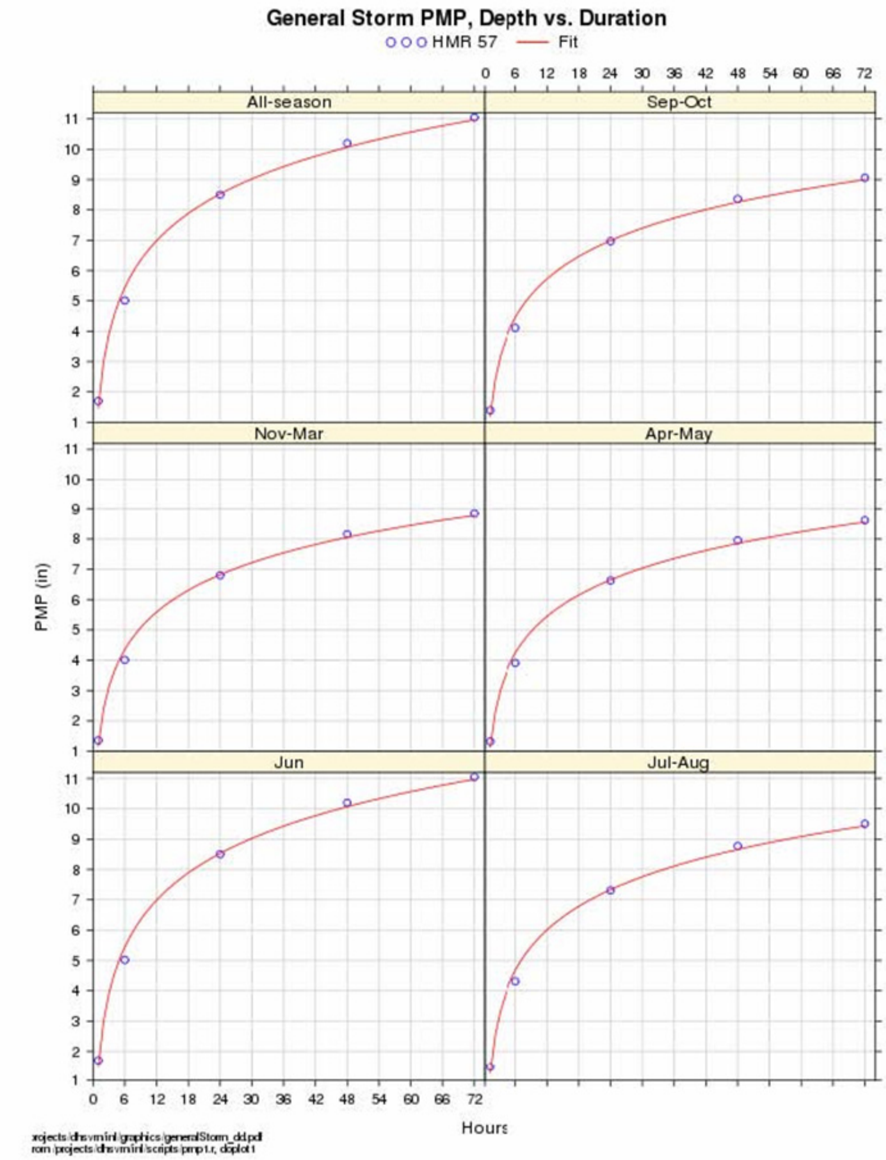


Figure 2.2. General Storm PMP by Season

2.4

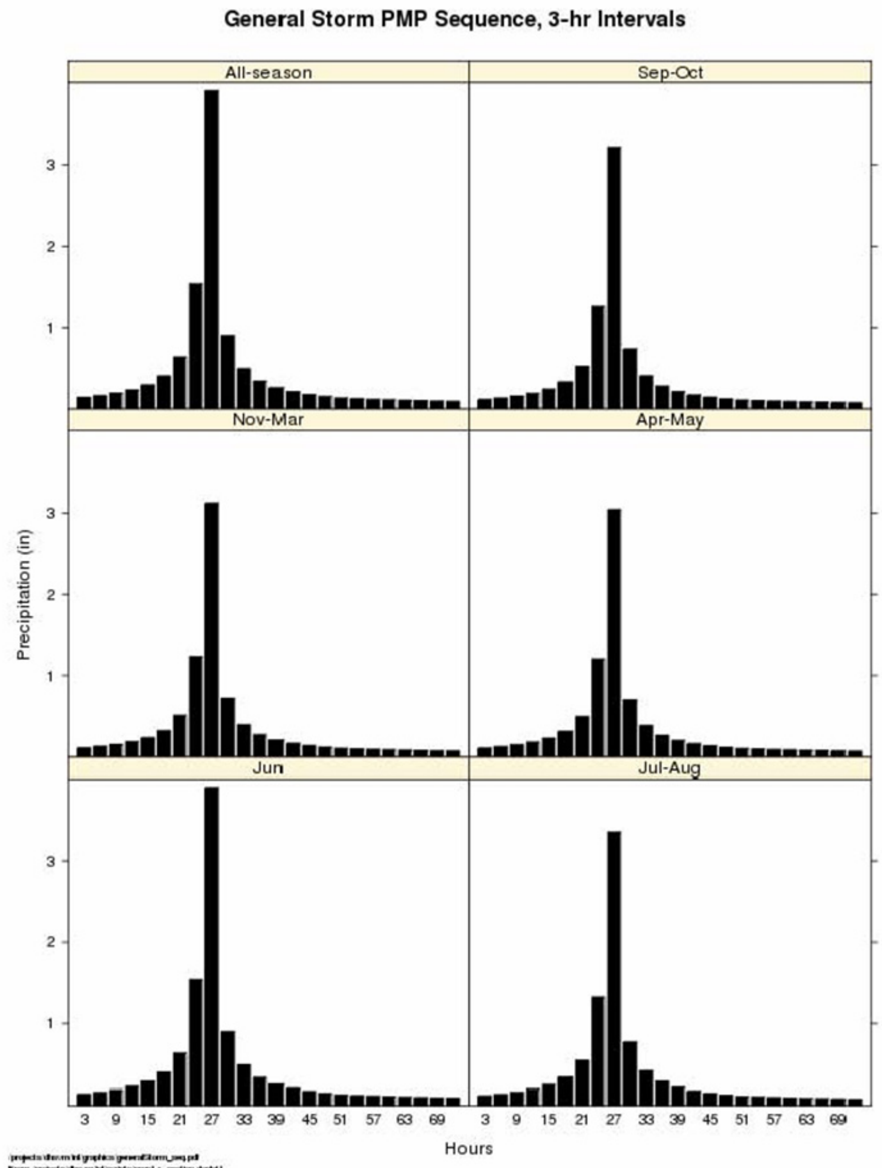


Figure 2.3. Design Storm Hyetographs Based on the General Storm PMP. Six-hour precipitation increments are shown; the hyetograph used as input to HEC-HMS was based on 1-hour increments.

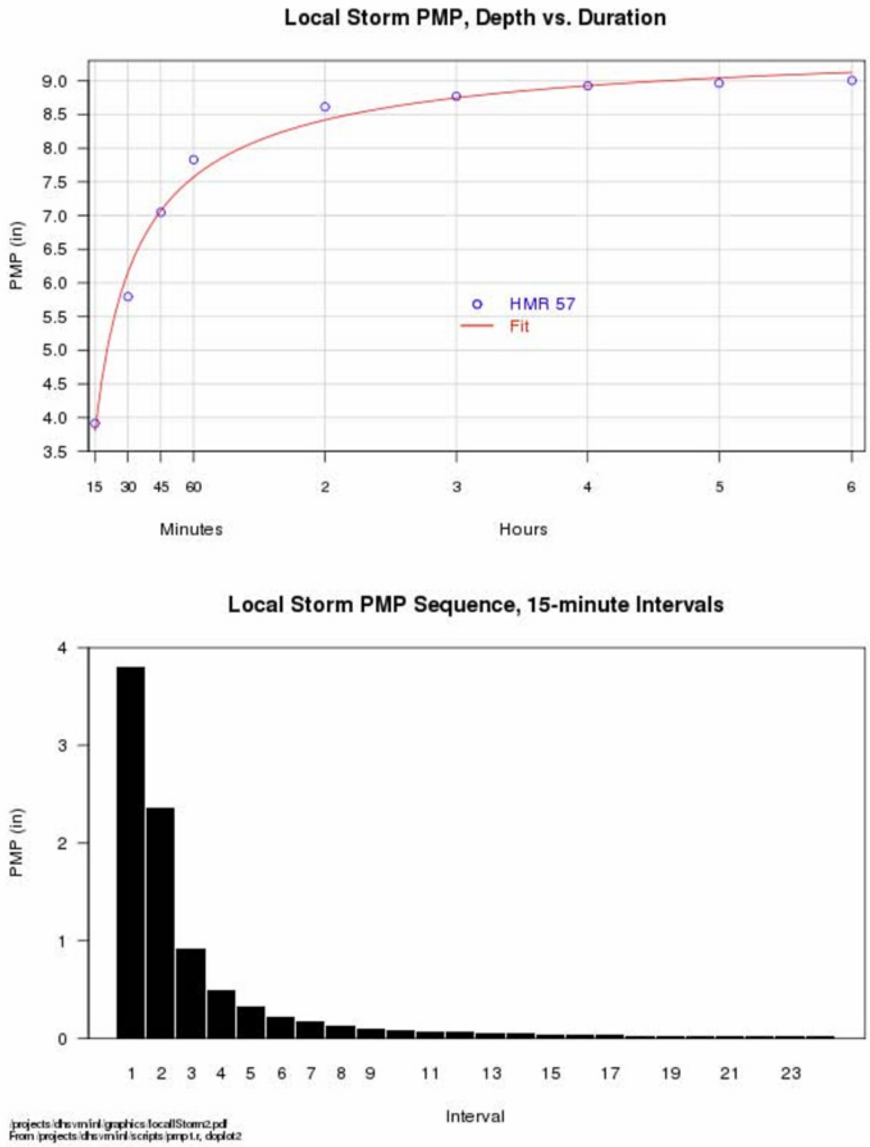


Figure 2.4. Local Storm PMP and Hyetograph with 15-Minute Intervals for the Design Storm

The CFA meteorological station is located at latitude 43.533 degrees north and longitude 112.948 degrees west at an elevation of 4950 ft. Its tower is north of CFA building CF-690. Daily precipitation values are from a manually measured rain gage. The MFC meteorological station (tower code EBR) is located at latitude 43.594 degrees north and longitude 112.652 degrees west at an elevation of 5143 ft. Its tower is near the MFC. A tipping bucket rain gage is used to measure precipitation.

Historical observations at INL have indicated that a rain-on-snow event might provide the largest flood at MFC. In such an event, the amount of water content in the antecedent snowpack is a critical factor for total runoff, along with the amount of rainfall in the storm. To estimate a worst-case, highest-water-content snowpack, data from the snowiest year at CFA were used. The maximum observed snow depth of 30 in. occurred during 1993 and lasted for 11 days (Figure 2.5). It was preceded by a long accumulation season with relatively little mid-winter melting. The precipitation that fell during this time period totaled 4.12 in., which was assumed to be the maximum possible water content of the snowpack. The actual water content of the snowpack was probably less because of loss by infiltration of liquid water into the ground and sublimation. The corresponding water content of 0.137 is somewhat more than typical new-fallen snow in temperate climates. This observed snow condition was assumed to be suitable for the antecedent condition in the PMF analysis.

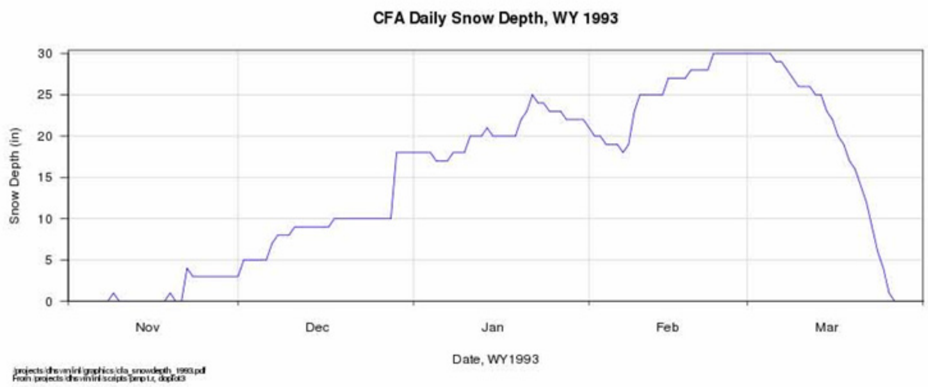
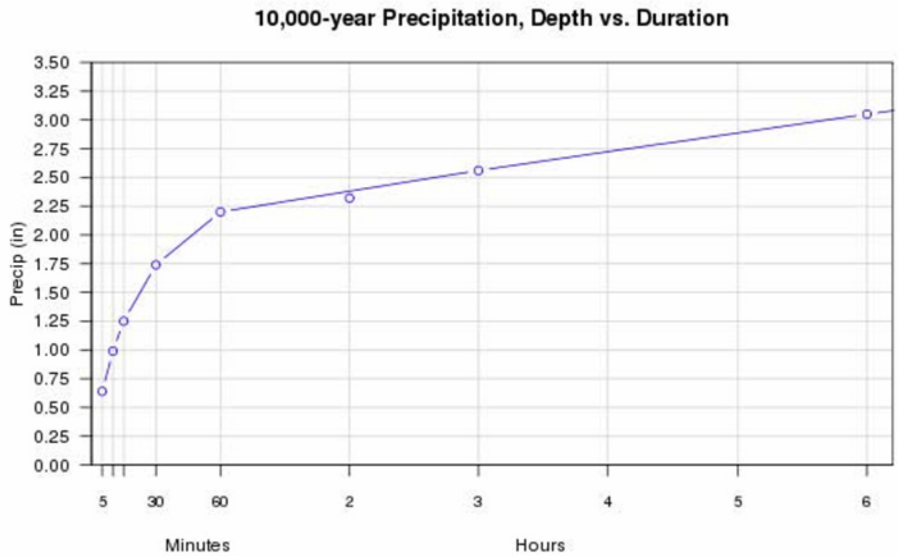


Figure 2.5. Snow Depth at the CFA Meteorological Station During Water Year 1993

The second design storm method was based on the 10,000-year precipitation previously estimated by Dames & Moore (D&M 1993). The rainfall amounts at 5, 10, 15, 30 minutes and 1, 2, 3, 6 hours from that report are shown in Figure 2.6. Not shown here for the sake of plot clarity but included in the 24-hour design storm are values for 12 and 24 hours. Five-minute increments of rainfall were derived by taking the difference between sequential cumulative amounts along the line shown in Figure 2.6. Linear interpolation was used so that the original point values were used as is; however, the value at 2 hours was omitted because it does not fit the general pattern of increase shown by the other Dames & Moore values. The first 6 hours of the 5-minute increments used to drive the watershed model are shown in Figure 2.7.



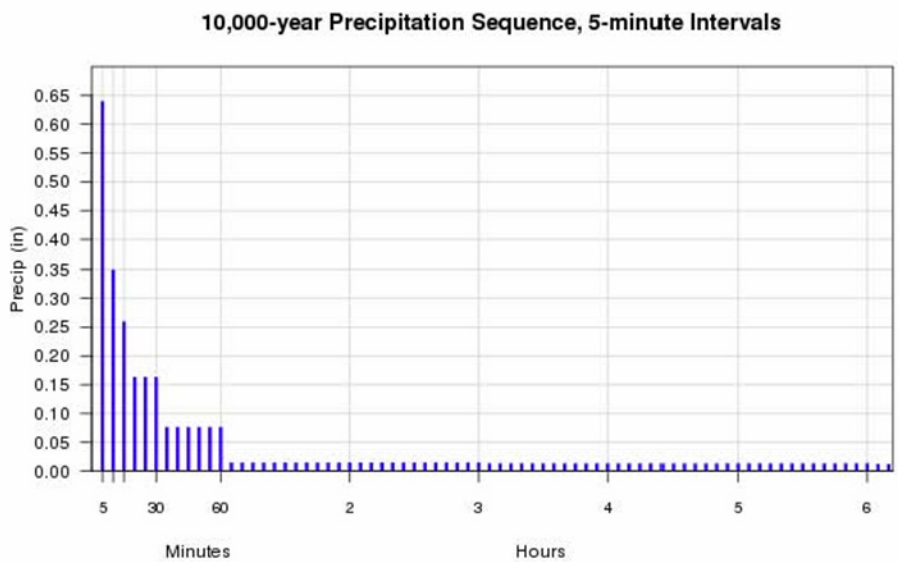


Figure 2.6. 10,000-Year Precipitation Event Depth vs. Duration and 5-Minute Interval Storm Sequence

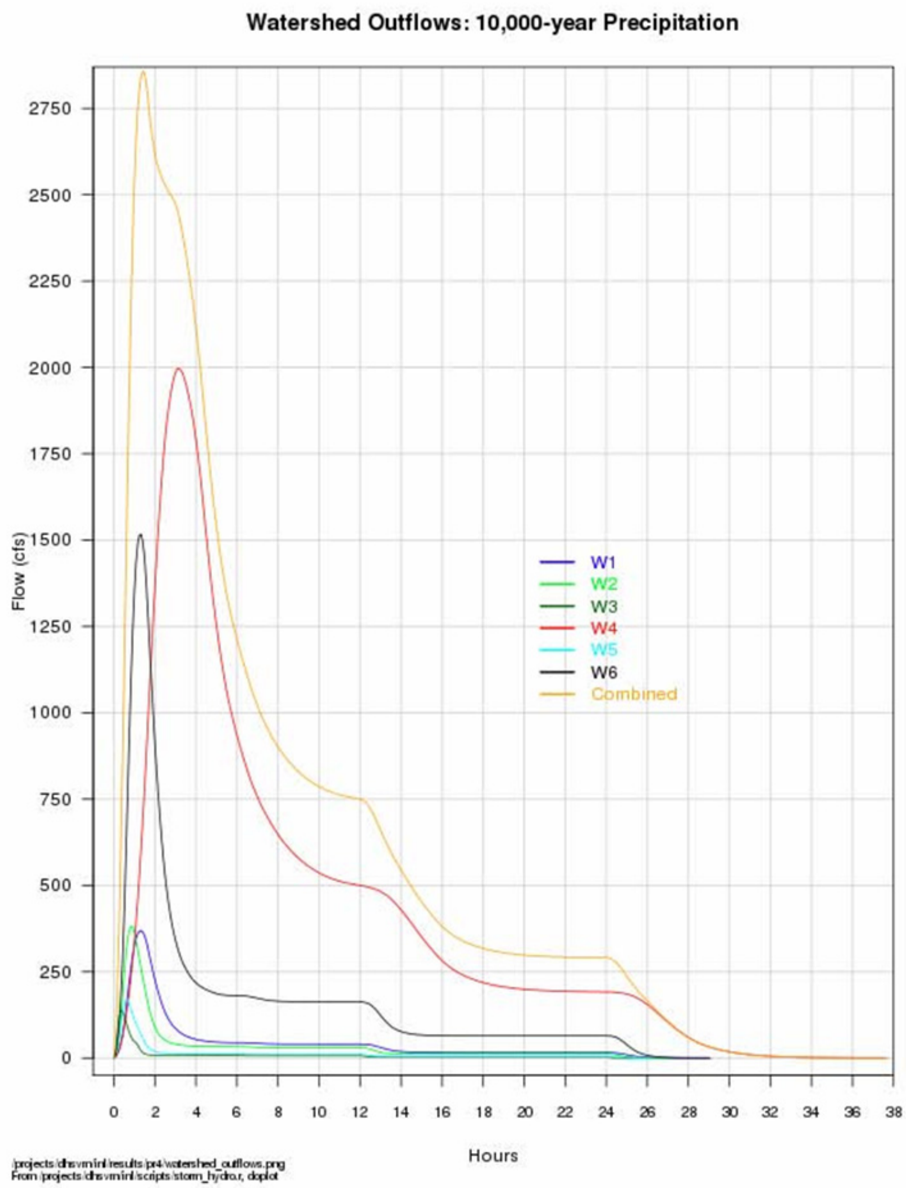


Figure 2.7. Hydrographs for the 10,000-Year Event

2.2 Determination of the Probable Maximum Flood and 10,000-Year Flood

The watershed model HEC-HMS was used to transform the design storms into watershed outflows, which were used as inflows for HEC-RAS hydraulic model analysis. As stated previously, losses due to surface ponding, infiltration, and evaporation were assumed to be zero, which provides a conservative (highest) estimate of runoff for the PMF analysis. Therefore, the runoff volumes equaled the precipitation over the watersheds. Given that historical streamflow records were not available for the MFC site, synthetic unit hydrographs (UHs) were used as the transform method to convert the design storm hyetographs to streamflow hydrographs.

2.2.1 Unit Hydrographs

The UH is conceptually the direct runoff response of one unit of excess precipitation occurring uniformly over the watershed during a specified time period. The key assumptions of UHs are that they are linear and time-invariant, so that runoff from greater or less than one unit of precipitation is simply a multiple of the UH ordinates, and that this relationship holds regardless of antecedent conditions or other circumstances. In the absence of historical rainfall-runoff records, the best available method for estimating steamflows is the synthetic UH. Synthetic UHs are parameterized entirely from watershed properties such as topography and land cover, rather than analysis of rainfall and runoff data, and as such are highly uncertain.

Methods in the following resources were used to develop the synthetic UHs: *Handbook of Hydrology* (Maidment 1993), *Hydrology for Engineers* (Linsley et al. 1982), and *Hydrology and Floodplain Analysis* (Bedient and Huber 1992). The simplest approach is the Soil Conservation Service (SCS) unit hydrograph method, which requires knowing only the duration of excess rainfall and the time lag, or the time from the midpoint of the precipitation period to the time of peak flow. Periods of 15 minutes and 1 hour were both used for the duration of 1 in. of excess rainfall. The SCS time lag was computed as follows (Bedient and Huber 1992):

$$t_p = (L\text{^}0.8 * (S + 1)\text{^}0.7) / (1900 * s\text{^}0.5) \tag{2.1}$$

where t_p = time lag (hr)
L = length of main channel from outlet to basin divide (ft)
S = storage coefficient
s = mean watershed slope (%).

$$S = (1000/CN) - 10 \tag{2.2}$$

where CN is the SCS curve number for the particular soil type and land cover.

Peak flow is then computed as

$$Qp = 483.4 * A / T_r \tag{2.3}$$

where Q_p = peak flow rate (cfs)
A = basin area (mi^2),
T_r = time of rise = time from start of rainfall (and runoff) to time of peak (hr). Time of rise is

$$T_r = 0.5*D + t_p \tag{2.4}$$

where D is the duration of rainfall excess (hr).

The resulting triangular UHs based on 1 in. over 15 minutes and 1 in. over 1 hour for W4 are shown in Figure 2.8. For comparison, UHs based on other methods were also developed. UH theory does not consider the nonlinear watershed response to high-intensity rainfall associated with PMPs (i.e., higher peak flows and shorter times of rise). To account for this phenomenon, the peak was increased by 20% and the time of rise decreased by 33%. The modified SCS UHs in Figure 2.8 do that, with an added property of maintaining the time base of runoff instead of having it decrease to preserve runoff volume in a triangle. This was done by adding an ordinate between the peak flow and the end of the hydrograph, such that the falling limb has an indentation. The time for this ordinate was set at 25% of the falling limb time base, and the flow at that time calculated such that runoff volume was preserved. As shown in Figure 2.8, the SCS method produces the largest peak flow and provides the most conservative estimate.

HEC-HMS alters the SCS UH so that it is a curve rather than a triangle. The HMS time of rise is the same as that derived by the manual triangle method, but the peak flow is somewhat higher (Figure 2.9).

HEC-HMS also offers a Snyder UH option, with a time lag peaking coefficient, C_p, required as input. Two different equations for time lag were used:

$$t_p = Ct * (L * L_c) \wedge 0.3 \text{ (Bedient and Huber 1992)} \tag{2.5}$$

where Ct is a coefficient, L has units of miles, and Lc is the length of main channel from the outlet to a point opposite the basin centroid (mi).

The other time lag equation is

$$t_p = C_t * (L * L_c / {}_{sr}\wedge 0.5) \wedge 0.38 \text{ (Linsley et al. 1982)} \tag{2.6}$$

where s_r is a weighted channel slope (ft/ft).

For the coefficients C_t and C_p, various combinations of values were used, in the ranges suggested in the literature. Table 2.1 lists the resultant values for t_p. All Snyder UHs resulted in peak flows that were lower than those derived using the SCS methods.

For this analysis, loss due to infiltration was assumed to be zero, but in the SCS UH method the curve number (CN) and dependent storage coefficient, S, also affect the shape of the hydrograph via the time lag, with lower peaks and longer runoff durations as CN decreases. The CN was set to 89, representing poorly permeable soil group D and rangeland in poor condition (Bedient and Huber 1992). The curvilinear version of the 15-minute SCS UH with CN = 89 was the one used for the PMF analysis; this is shown for all watersheds in Figure 2.9.

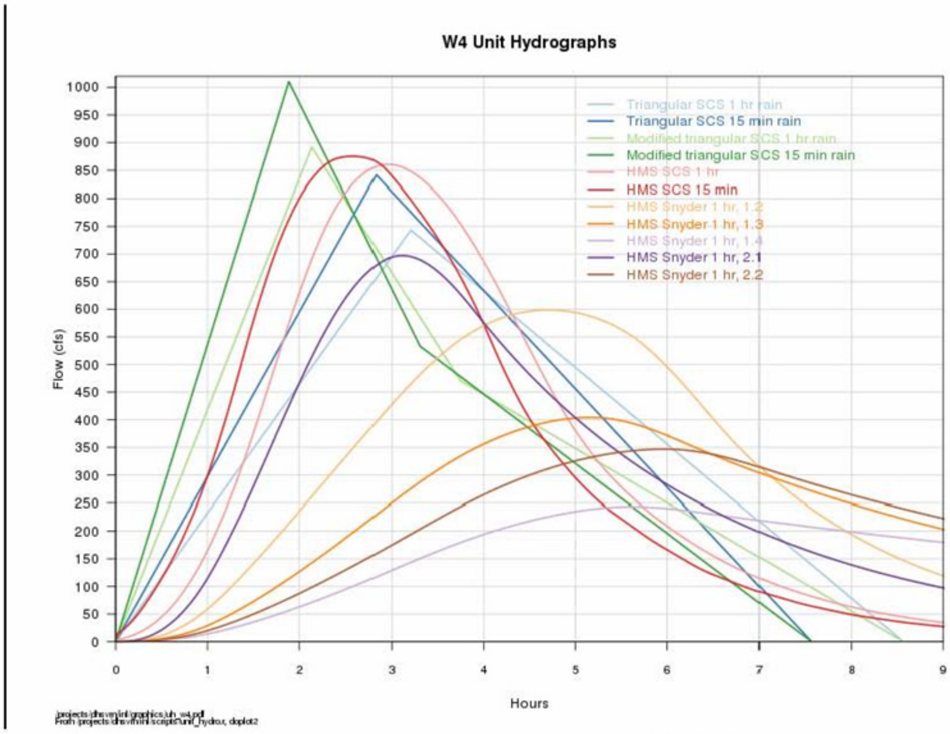


Figure 2.8. Unit Hydrographs for Watershed 4 (W4)

2.2.2 HEC-HMS Model Results

The local storm PMP and the UH were used in HEC-HMS to generate separate storm flows for all six watersheds. The resulting streamflows are shown in Figure 2.10. Watershed 4 (W4) is by far the largest contributor of flow, but the peak of the combined flows happens earlier than the W4 peak, at about 1.5 hours.

For comparison, the design hyetograph based on the general storm PMP was also used to drive the HEC-HMS model. As expected, the storm flows from this lower-intensity, longer-duration storm were less than half of those produced by the higher-intensity local storm (Figure 2.11). To include the contribution of snowmelt from an antecedent snowpack, a design storm with assumed complete melting over 4 hours was also run through HEC-HMS. The 4.12 in. of assumed snow water content was divided equally (1.03 in.) and added to each of the 4 hours of maximum rainfall intensity. The final water increments after addition of snowmelt during those 4 hours were 1.67, 2.49, 2.57, and 1.93 in., respectively. The resulting hydrographs are shown in Figure 2.12. This scenario resulted from a collection of maximizing assumptions for the general storm PMF: use of the all-season PMP instead of the lower winter season PMP, no loss of precipitation or snowmelt, a very high snowpack water content, and very rapid melting of the snowpack coincident with the time of highest rainfall. Even with all of these assumptions, the peak flows are still less than those from the warm season local storm. This outcome justified the use of the local storm for the subsequent HEC-RAS analysis of channel flood routing.

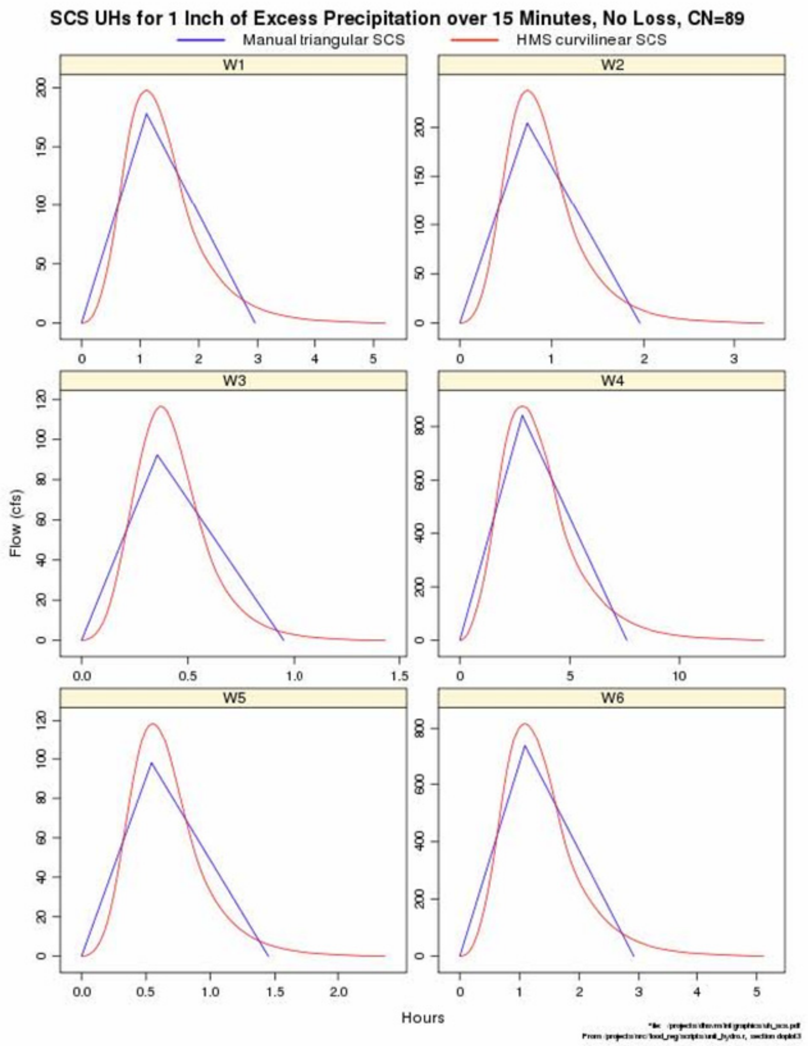


Figure 2.9. SCS Triangular and HMS Curvilinear Unit Hydrographs for All Watersheds

2.13

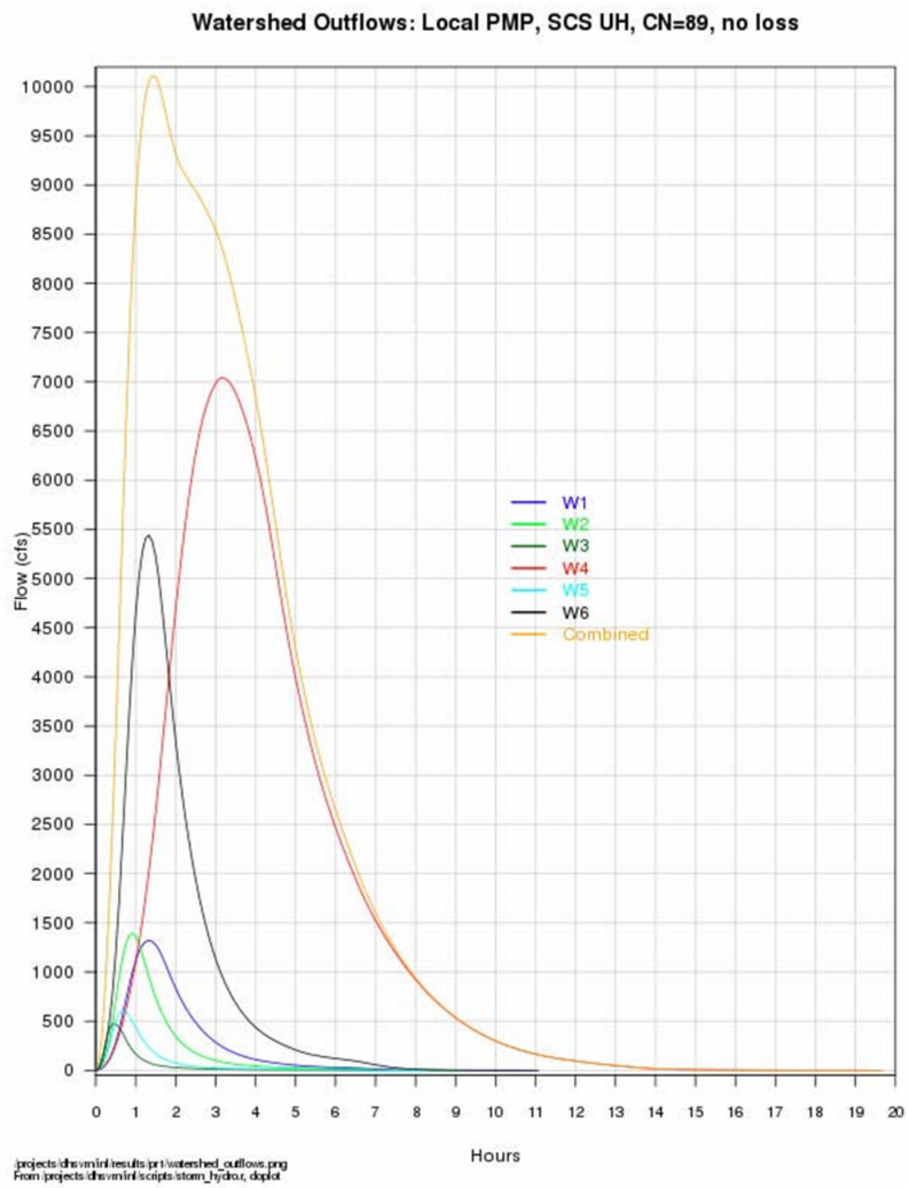


Figure 2.10. PMF Hydrographs for the Local Storm

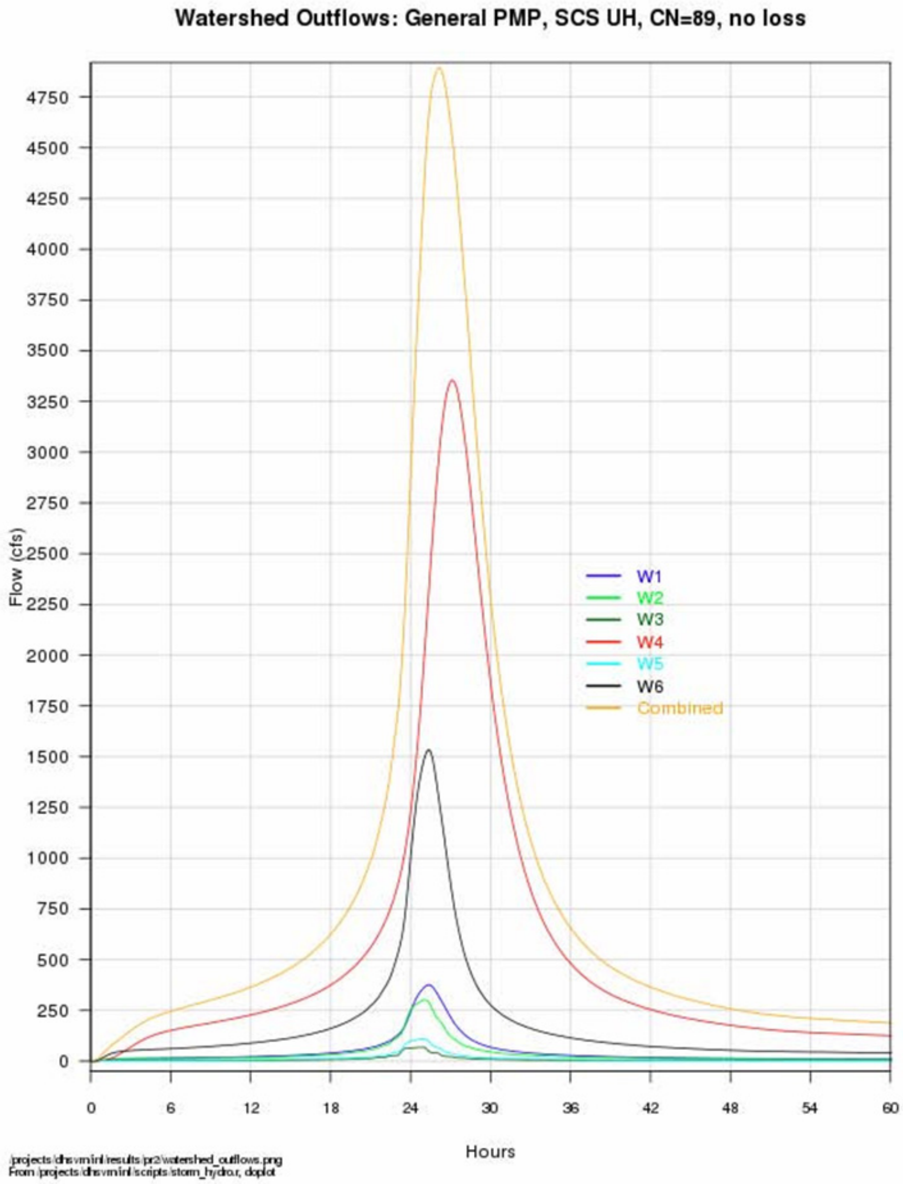


Figure 2.11. PMF Hydrographs for the General Storm

2.15

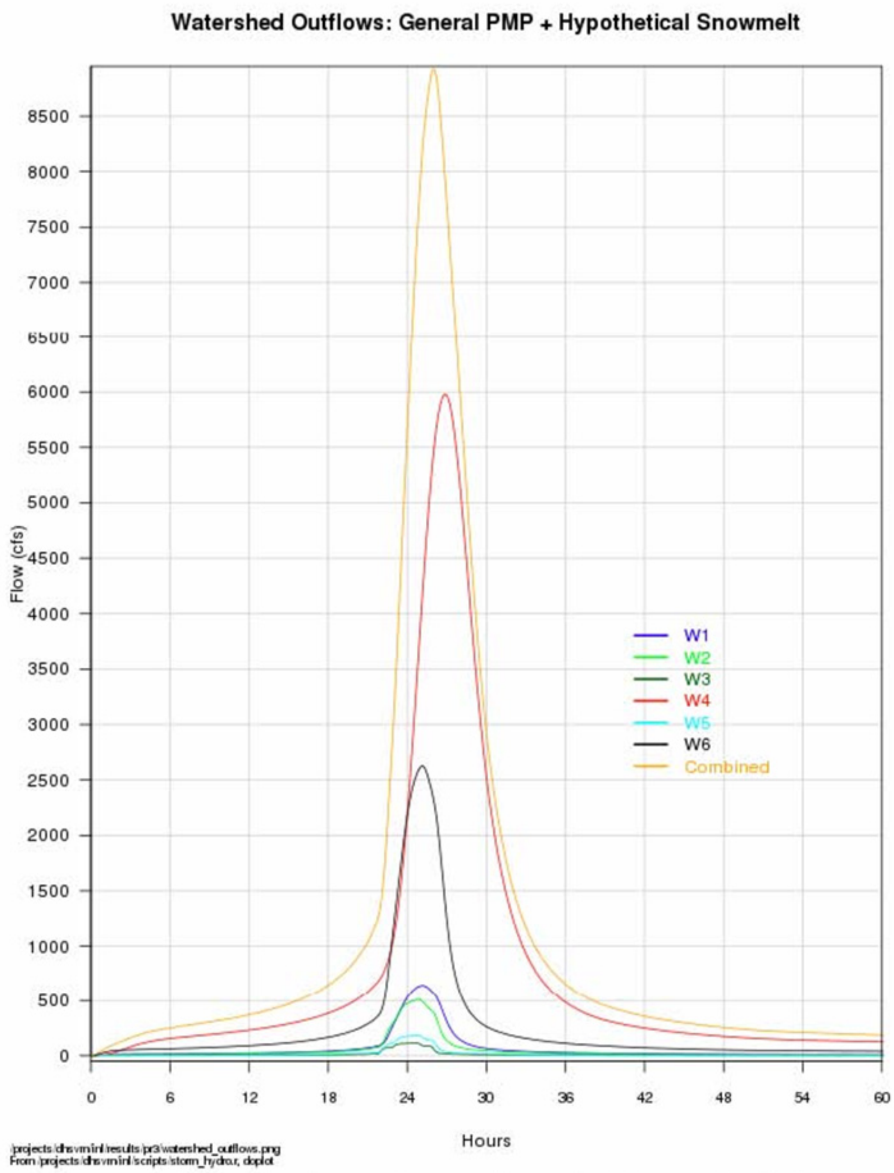


Figure 2.12. PMF Hydrographs for the General Storm with Hypothetical Snowmelt Added. The storm flows from the 10,000-year precipitation, shown in Figure 2.6, are much less than any of the PMP-based flows. The combined runoff from all six watersheds has peak value about 28% of that of the local storm PMP peak.

2.16

2.3 Hydraulic Analysis and Determination of Peak Elevation

The hydraulic analysis and peak elevation determinations for the MFC and the TREAT Facility are described in this section. The analysis uses the HEC-RAS hydraulic model to compute flow velocity and depth. Water-surface elevations are computed by the addition of depth and cross-section elevations.

2.3.1 MFC Site

The hydraulic analysis was conducted to estimate the maximum water-surface elevations during the PMF and a 10,000-year return period event at the MFC site. The analysis uses the HEC-RAS hydraulic model to route flow through the site with consideration of backwater effects that could occur due to diversion dams, roads, and blocked culverts. HEC-RAS is a one-dimensional model and its setup requires thoughtful assembly of cross-sectional and control-structure characteristics of the system.

2.3.1.1 Hydraulic Model Setup

Cross-section data input into HEC-RAS were extracted from available Light Detection and Ranging (LiDAR) data (see Appendix B). The geometric representation of the MFC site and surrounding areas included cross sections that characterize the presence of drainage channels, roads, and buildings. The site was divided into four subbasins: Subbasins 1 through 4 (labeled as B1, B2, B3, and B4 in Figure 2.13). Geographic information system (GIS) software (Global Mapper) was used to extract cross sections from high-resolution LiDAR data (Figure 2.13). A higher density of cross sections was added within the MFC site to characterize building structures, culverts, ditches, and roads, while a smaller number of cross sections were used in surrounding areas but nonetheless included features that provided hydraulic control (dams, channels, and roads). Cross sections were input to the HEC-RAS model using a GIS format, and each cross section was identified and checked against the original LiDAR data.

The one-dimensional channel connections were setup as follows (Figures 2.13 and 2.14):

- Subbasin B1 was selected as the mainstream basin.
- Subbasin B4 was connected to subbasin B1 from the left-hand side and subbasin B3 was connected to subbasin B1 from the right-hand side.
- Subbasin B3 receives flows from subbasin B2.

Figure 2.15 shows the plan view from the HEC-RAS model setup. Due to the one-dimensional method used in HEC-RAS, it was necessary to account for the connections between subbasins via lateral flow exchange. These flow exchanges were handled using lateral weir structures in HEC-RAS with the weir elevation and width based on LiDAR data. These weirs were included between subbasins B1 and B2 and between subbasins B1 and B3 (Figure 2.13).

The following assumptions were used for the analysis:

- Flow was one-dimensional using the HEC-RAS hydraulic model.
- The subbasins were lightly vegetated sagebrush (Manning's n = 0.035) with sandy channels (Manning's n = 0.030).

- Roads and dams were handled as weirs with the weir elevation based on road elevation derived from the LiDAR data. The default weir coefficient of 2.6 provided by HEC-RAS during model setup was used in all analyses.
- Two levels of conservatism were examined:
  - Most conservative with all culverts in the MFC area (subbasin B3) blocked. Also, the diversion channel that reroutes flow from the upper reach of subbasin B2 into the upper reach of Basin 1 was blocked. This is referred to as the Blocked case.
  - Least conservative with all of the ditches in the MFC area (subbasin B3) open. Also, the diversion channel that reroutes flow from the upper reach of subbasin B2 into the upper reach of subbasin B1 was open. This is referred to as the Open case.
  - For both levels of conservatism, the bridge under the access road crossing subbasin B1 was in place.
- The downstream boundary was set as a normal depth boundary with a friction slope of 0.0002. This value was estimated from the local topography near the downstream boundary.
- The stormwater detention pond northwest of MFC was filled.
- Local inflow from precipitation onto HEC-RAS subbasins was based on the Rational Method with no precipitation loss and was linearly distributed along the subbasins, with the distribution based on surface area.

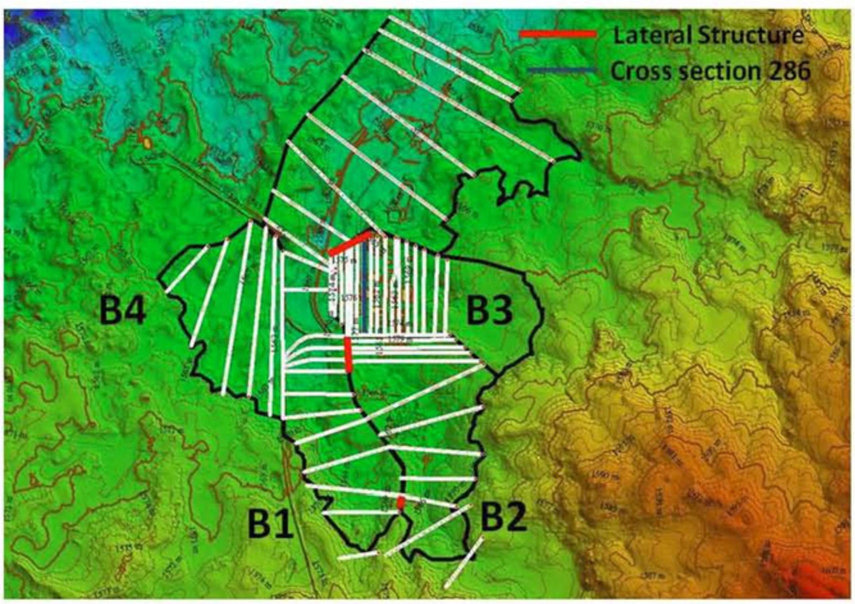


Figure 2.13. Four Subbasins Used for the HEC-RAS Model Analysis and HEC-RAS Cross-Section Layout for the MFC Site and Surrounding Areas

Figure 2.14. Detail of HEC-RAS Cross Sections Overlaid onto the Buildings of the MFC Site

Inflow hydrographs from HEC-HMS analyses provided the watershed drainage flows (discussed in Section 2.2), and the local inflow estimates mentioned above were input at 5-minute intervals. For the local runoff from the HEC-RAS basins (i.e., the drainage areas not included in the HEC-HMS analysis in the immediate vicinity of the channels), the spatial scale is small enough that lag time need not be considered. In addition, using the rational method and assuming no loss to infiltration provides a conservative estimated runoff. Any attenuation of the local runoff (and watershed runoff) occurs in the HEC-RAS model as a consequence of the geometric configuration of the system and the assumed surface roughness parameters. The flooding analysis included an examination of flows generated from two precipitation events: the PMP (generating the PMF) and the 10,000-year return period event. The watershed drainage and local inflows for the PMF event are shown in Figure 2.16, while the inflows for the 10,000-year return period are shown in Figure 2.24. Note that the estimated local inflows were large with respect to the watershed drainage especially in the 10,000-year return period event. Because the local inflows were computed from local precipitation intensities in the HEC-RAS basins, they peaked prior to the watershed drainage inflows.

Initial flows for the HEC-RAS model were set to be the same as the first 5-minute flows from the watershed drainage inflow hydrographs. For the PMF analysis, the model time step was 1 minute, and the simulation period was 11.5 hours. The analysis showed that the passage of the PMF peak through the site took about 4 hours from the start of the simulation. For the 10,000-year return period analysis, the model time step was 1 minute, and the simulation period was 13 hours. The analysis showed that the passage of the 10,000-year return period peak through the site took about 3.5 hours from the start of the simulation.

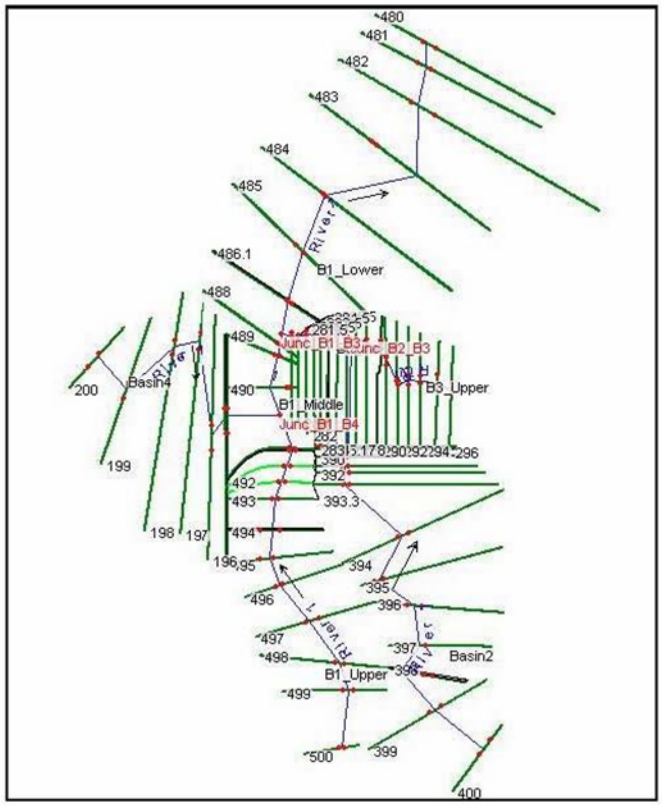


Figure 2.15. Plan View of HEC-RAS Model Setup for the MFC Site and Surrounding Areas

2.3.1.2 PMF Results at the MFC Site

As previously noted, the inflows for the PMF event are shown in Figure 2.16. Subbasin B1 and subbasin B2 watershed drainage inflows had comparable peak flows; however, the timing of the peak from subbasin B2 was about 2 hours later than from subbasin B1. Subbasin B1 had the largest area of the four HEC-RAS basins included in the analysis; consequently the local inflows are the greatest of the four, although these peaked early in the analysis.

The plots of maximum water level profiles from the HEC-RAS model analyses for the Blocked and Open cases show the hydraulic relationships between the subbasins. For the Blocked case, Figure 2.17 shows the combined profiles for all of the subbasins, while Figure 2.18 shows the profiles for individual subbasins. Figure 2.19 shows the maximum water-surface elevation for the Blocked case in the middle of the MFC site at cross-section station 286 (for reference, Figures 2.13 and 2.14 show the cross-section locations). At cross-section station 286, the maximum elevation was 5125.55 ft.

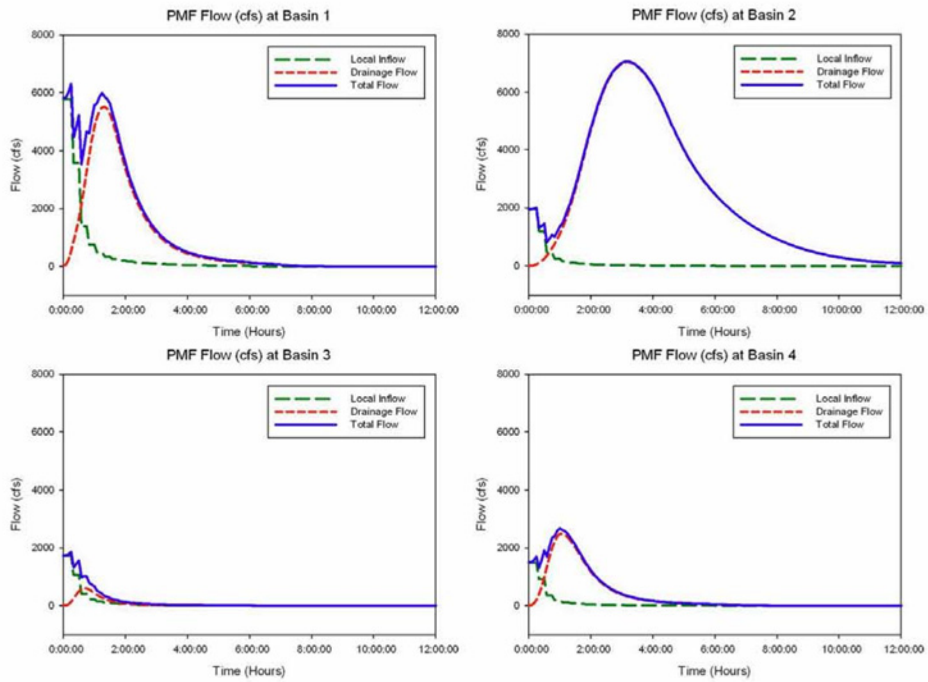


Figure 2.16. Flows Input to the HEC-RAS Hydraulic Model for the PMF Event. The plots include the watershed drainage flow from the HEC-HMS analysis plus the local inflow estimated using the rational method with the PMP intensities. The drainage flow is input at the upstream end of each basin, while the local inflows are linearly distributed along the basin.

2.21

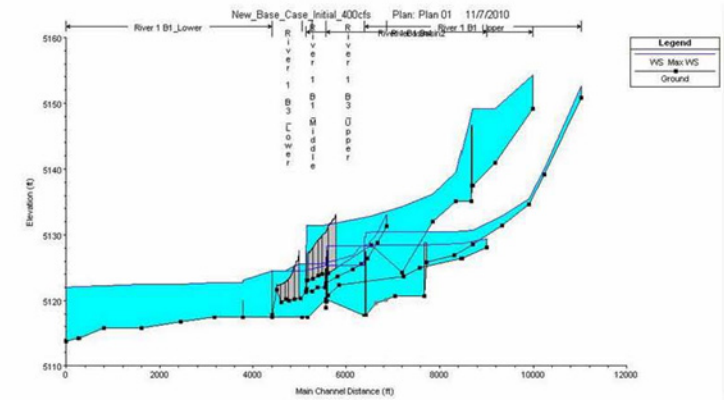


Figure 2.17. Combined Profiles of the Maximum Water Level for all Subbasins from the PMP Event (Note: gray shaded areas designate weirs.)

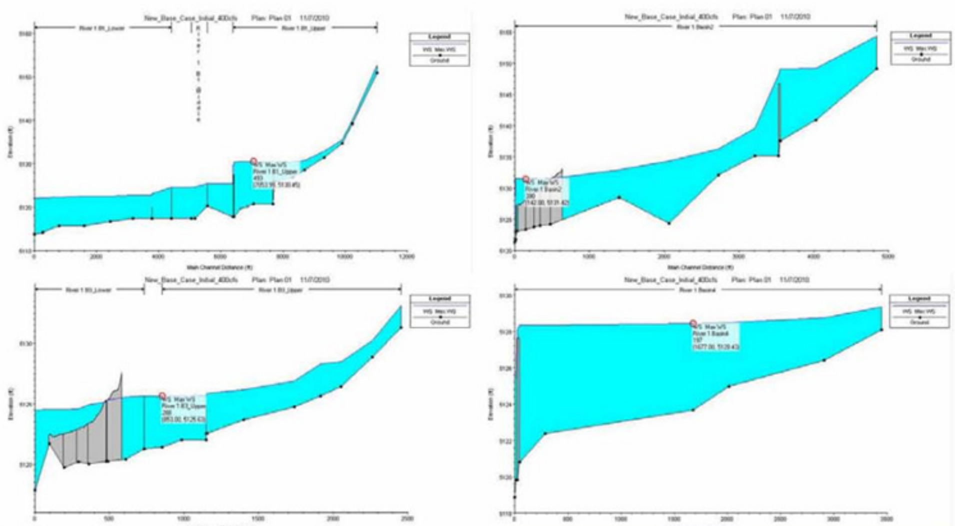


Figure 2.18. Individual Profiles of the Maximum Water Level for Each Subbasin from the PMP Event (Note: gray shaded areas designate weirs.)

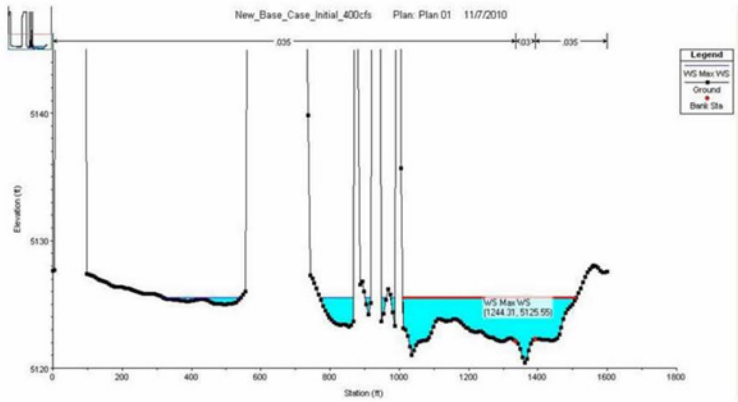


Figure 2.19. Maximum Water Level at Cross-Section 286 from the PMP Event

For the Open case, Figure 2.20 shows the combined profiles for all the subbasins, while Figure 2.21 shows the profiles for individual subbasins. Figure 2.22 shows the maximum water-surface elevation for the Open case in the middle of the MFC site at cross-section station 286 (for reference, Figures 2.13 and 2.14 show the cross-section locations). At station 286, the maximum elevation for the cross section was 5125.30 ft. This Open case elevation is 0.25 ft lower than for the Blocked case.

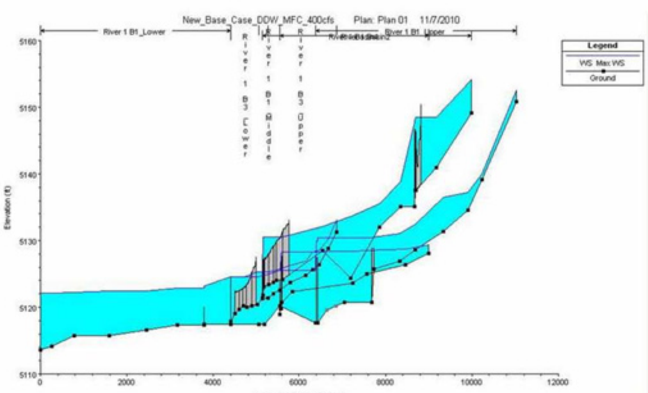


Figure 2.20. Combined Profiles of the Maximum Water Level for all Subbasins from the PMP Event (Note: gray shaded areas designate weirs.)

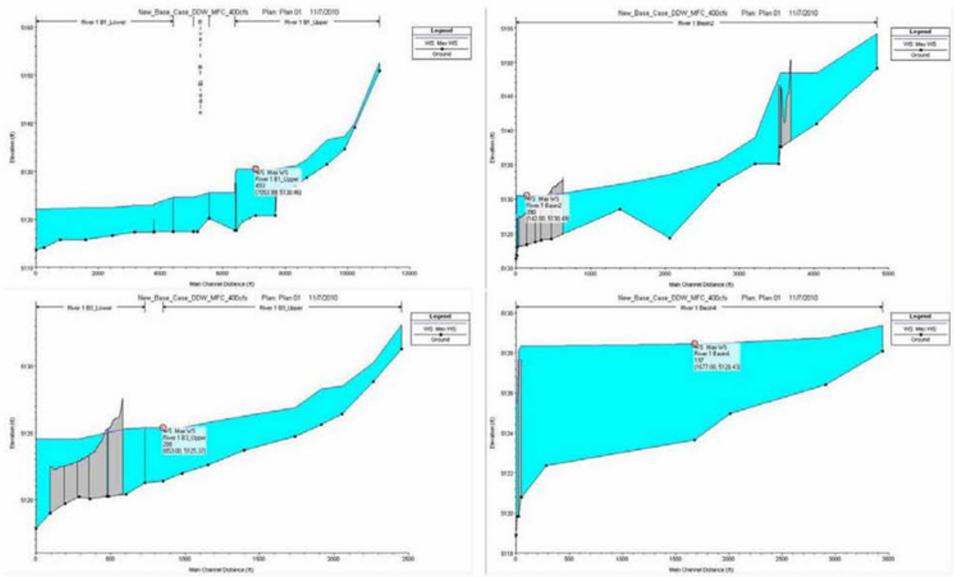


Figure 2.21. Individual Profiles of the Maximum Water Level for Each Subbasin from the PMP Event (Note: gray shaded areas designate weirs.)

Figure 2.23 illustrates the flow paths of the PMF runoff for both the Blocked and Open cases by indicating the location, direction, and magnitude of the maximum flows at the boundaries of the subbasins where flow is exchanged. For the PMF, overtopping of the diversion dam occurs for both the Blocked and Open cases, with both cases discharging large flows into the middle reach of Basin 2. Inclusion of the diversion channel (the Open case) reduces the maximum flow that overtops the diversion dam from 7031 cfs to 4320 cfs, a reduction of 39%. For the Open case, the 2706 cfs maximum flow diverted into Basin 1 reduces flow from Basin 2 into Basin 3 (the MFC site) by only about 900 cfs; the flow drops from 3812 cfs for the Blocked case to 2924 cfs for the Open case.

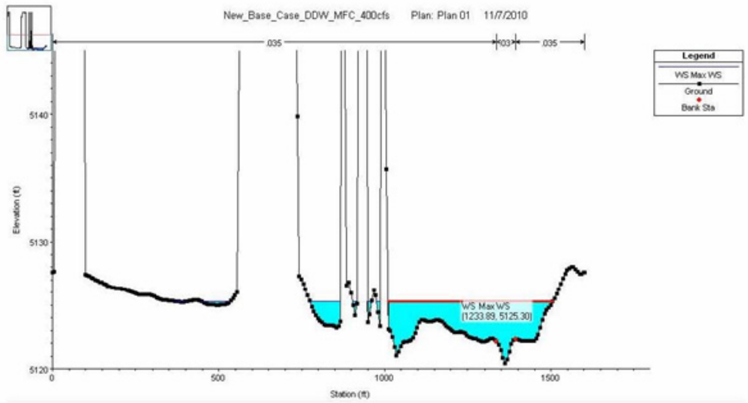


Figure 2.22. Maximum Water Level at Cross-Section 286 from the PMP Event

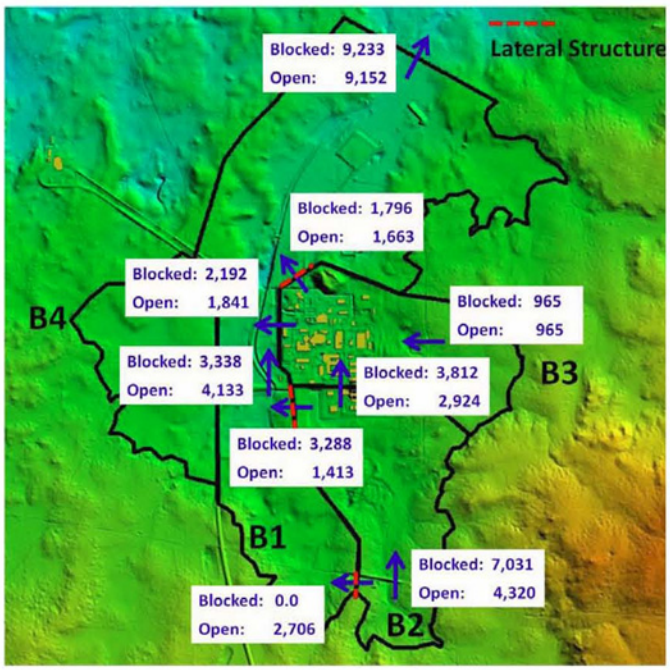


Figure 2.23. Flows Paths and Maximum Flows (cfs) for the PMF for Blocked and Open Cases

As previously noted, Figure 2.14 shows the layout of the HEC-RAS cross sections with respect to critical buildings in the MFC. Table 2.2 shows the maximum water-surface elevations extracted at cross sections associated with the critical buildings. The PMF results with the Blocked case shows nine

2.25

buildings at which the maximum water surface elevation exceeds the floor elevations. For subbasin B3 stations, the exceedances were up to 1.4 ft, while for the subbasin B2 stations, an exceedance of over 3 ft occurred. The Open case shows eight buildings at which the elevation is exceeded. The exceedances are smaller for the Open case compared to the Blocked case, with the maximum exceedance being less than 3 ft. While the diversion of flows from subbasin B2 into subbasin B1 in the Open case reduces over-topping of the diversion dam from 7031 cfs to 4320 cfs, the flows entering the MFC site directly are nonetheless large enough to affect the maximum water-surface elevations of the MFC site in subbasins B2 and B3.

Table 2.2. Comparison of the Floor Elevations of Buildings at the MFC Site and the Maximum Water-Surface Elevations Obtained from the HEC-RAS Model. The HEC-RAS identification (ID) numbers are the cross-section stations that are at or just upstream of the building. The building locations and the cross sections are shown in Figure 2.14. Maximum water-surface elevations highlighted in gray indicate the value exceeds the flood elevation. The Open cases have the subbasin B3 culverts and subbasin B2 diversion channel open, while the Closed cases have all culverts and the diversion channel blocked.

Building Number and Name	Location	Finished Floor Elevation	Station ID and Subbasin	PMF		10,000-yr Return Period	
				Blocked	Open	Blocked	Open
MFC-704 Fuel Manufacture Facility (FMF)	N. Door	5127.59	291 B3	5126.14	5126.25	5125.53	5125.59
MFC-752 Laboratory and Office Building	W. Dock	5128.23	288 B3	5125.63	5125.37	5124.91	5124.28
MFC-752 Laboratory and Office Building	E. Dock	5128.24	289 B3	5125.63	5125.40	5124.91	5124.37
MFC-752 Laboratory and Office Building	N. Door East Side	5128.23	289 B3	5125.63	5125.40	5124.91	5124.37
MFC-752 Laboratory and Office Building	S. Door East Side	5126.73	289 B3	5125.63	5125.40	5124.91	5124.38
MFC-765 Fuel Conditioning Facility (FCF)	SE Door	5128.23	287 B3	5125.63	5125.37	5124.91	5124.28
MFC-765 Fuel Conditioning Facility (FCF)	W. Bay Door	5126.21	286 B3	5125.55	5125.30	5124.86	5124.26
MFC-767 EBR-II Reactor Plant Building	Adj. to Building North	5126.94	285 B3	5125.27	5125.01	5124.61	5124.02
MFC-767 EBR-II Reactor Plant Building	N. Truck Dock	5128.01	285 B3	5125.27	5125.01	5124.61	5124.02
MFC-767 EBR-II Reactor Plant Building	Adj. to Building West	5125.17	285 B3	5125.27	5125.01	5124.61	5124.02
MFC-774 ZPPR Support Wing	N. Door	5128.22	390 B2	5131.42	5130.49	5128.91	5127.91
MFC-775 ZPPR Vault-Workroom Eq Rm	Inside S. Security Door	5128.74	390 B2	5131.42	5130.49	5128.91	5127.91

Table 2.2. (contd)

Building Number and Name	Location	Finished Floor Elevation	Station ID and Subbasin	PMF Blocked	PMF Open	10,000-yr Return Period Blocked	10,000-yr Return Period Open
MFC-776 ZPPR Reactor Cell	Based on 775 El. & Design Drawings	5128.74	389 B2	5131.47	5130.52	5128.90	5127.87
MFC-785 Hot Fuel Examination Facility (HFEF)	S. Main door	5124.18	287 B3	5125.63	5125.40	5124.91	5124.28
MFC-786 HFEF Substation	N. Door	5124.23	286 B3	5125.55	5125.30	5124.86	5124.26
MFC-787 Fuels and Applied Science Building (FASB)	N. Door	5125.23	287 B3	5125.63	5125.37	5124.91	5124.28
MFC-792A Space & Security Power Sys Fac Annex	N. Side	5130.26	392 B2	5131.43	5130.52	5128.98	5127.93
MFC-798 Radioactive Liquid Waste Treatment Facility	N. Bay Door	5124.71	288 B3	5125.63	5125.37	5124.91	5124.28
MFC-1702 Radiochemistry Laboratory (RCL)	NW Door	5128.20	288 B3	5125.63	5125.37	5124.91	5124.28

2.3.1.3 10,000-Year Return Period Results at the MFC Site

As previously note, the inflows for the 10,000-year return period event are shown in Figure 2.24. Subbasin B1 and B2 watershed drainage inflow had comparable peak flows; however, the timing of the peak from B2 was about 2 hours later than that from B1. Subbasin B1 had the largest area of the four HEC-RAS basins included in the analysis; consequently the local inflows are the greatest of the four basins, although these peaked early in the analysis.

The plots of maximum water-level profiles from the HEC-RAS model analyses for the Blocked and Open cases show the hydraulic relationships between the subbasins. For the Blocked case, Figure 2.25 shows the combined profiles for all of the subbasins, while Figure 2.26 shows the profiles for individual subbasins. Figure 2.27 shows the maximum water-surface elevation for the Blocked case in the middle of the MFC site at cross-section station 286 (for reference, Figures 2.13 and 2.14 show the cross-section locations). At cross-section station 286, the maximum elevation was 5124.86 ft.

For the Open case, Figure 2.28 shows the combined profiles for all of the subbasins, while Figure 2.29 shows the profiles for individual subbasins. Figure 2.30 shows the maximum water-surface elevation for the Open case in the middle of the MFC site at cross-section station 286 (for reference, Figures 2.13 and 2.14 show the cross-section locations). At station 286, the maximum elevation for the cross section was 5124.26 ft. This elevation is 0.40 ft lower than for the Blocked case.

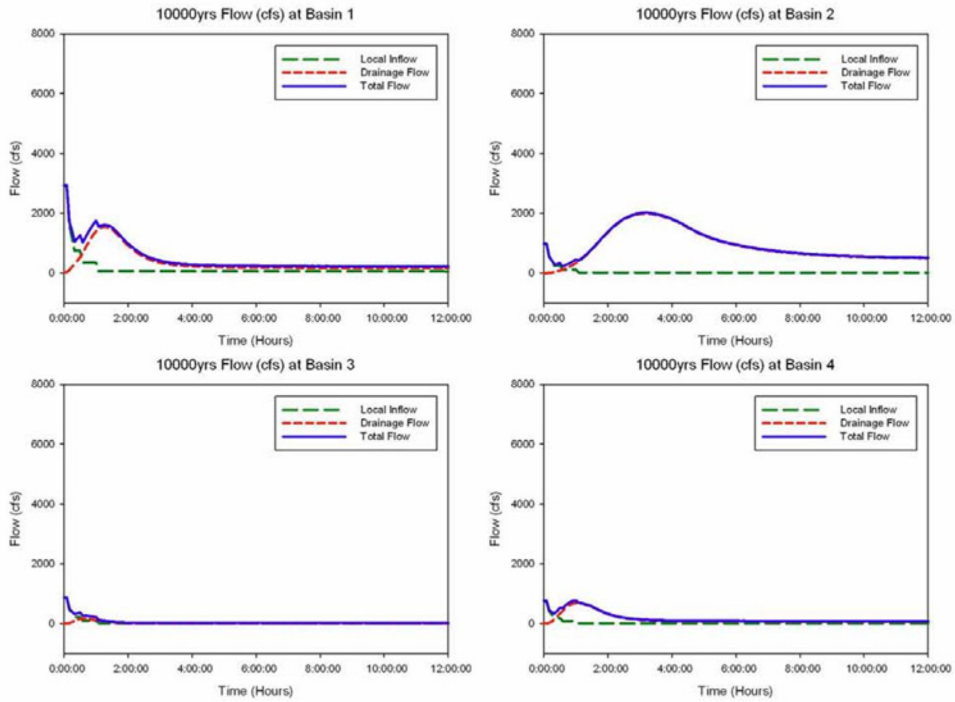


Figure 2.24. Flows Input to the HEC-RAS Model for the 10,000-Year Return Period Event. The plots include the watershed drainage flow from the HEC-HMS analysis plus the local inflow estimated using the rational method with the 10,000-year precipitation intensities. The drainage flow is input at the upstream end of each subbasin, while the local inflows are linearly distributed along the subbasin. The vertical scale for flow is kept the same as that used for Figure 2.16 for comparison with the PMF.

Figure 2.31 illustrates the flow paths of the 10,000-year return period event runoff by indicating the location, direction, and magnitude of the maximum flows at the boundaries of the subbasins where flow is exchanged. For the 10,000-year return period event, overtopping of the diversion dam occurs for both the Blocked and Open cases, although they are much reduced in comparison with the PMF. Inclusion of the diversion channel significantly reduces the maximum flow that overtops the diversion dam from 1994 cfs to 334 cfs. For the Open case, 1657 cfs was the maximum flow into the diversion channel. The diversion for the Open case produced a significant rerouting of watershed drainage flow around the MFC site at the upstream ends of Basins 1 and 2. However, under the Blocked case most of the flow was routed through the MFC site.

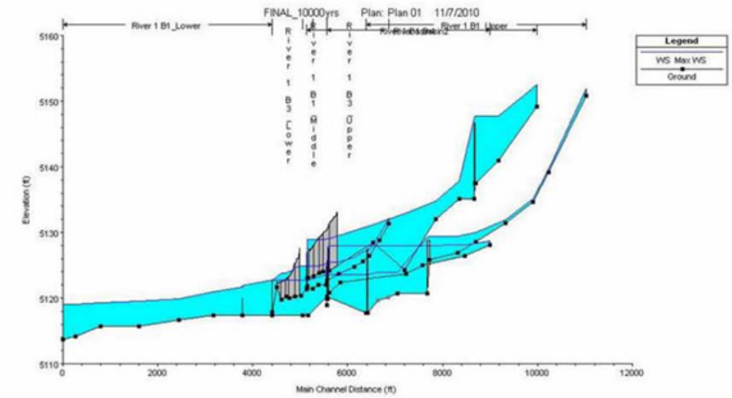


Figure 2.25. Combined Profiles of the Maximum Water Level for all Subbasins from the 10,000-Year Return Period Event (Note: gray shaded areas designate weirs.)

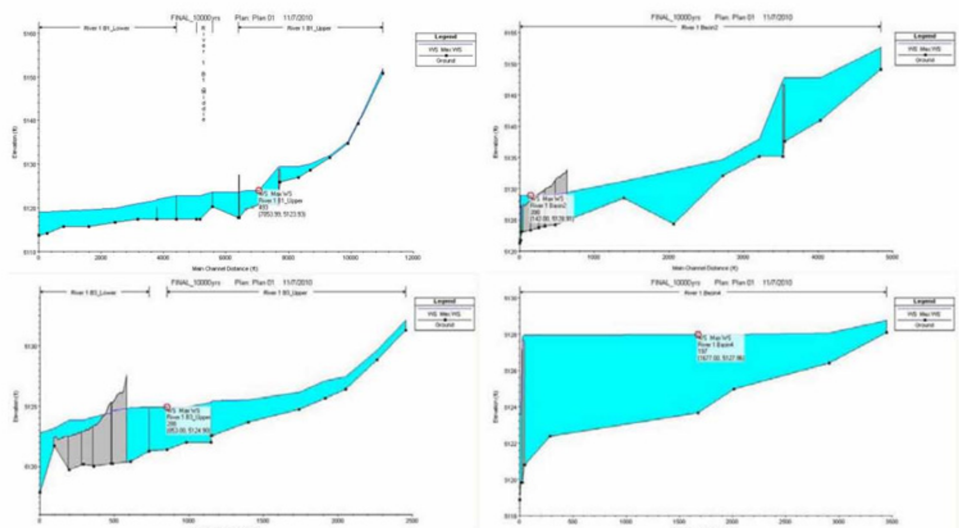


Figure 2.26. Individual Profiles of the Maximum Water Level for Each Subbasin from the 10,000-Year Return Period Event for the Blocked Case (Note: gray shaded areas designate weirs.)

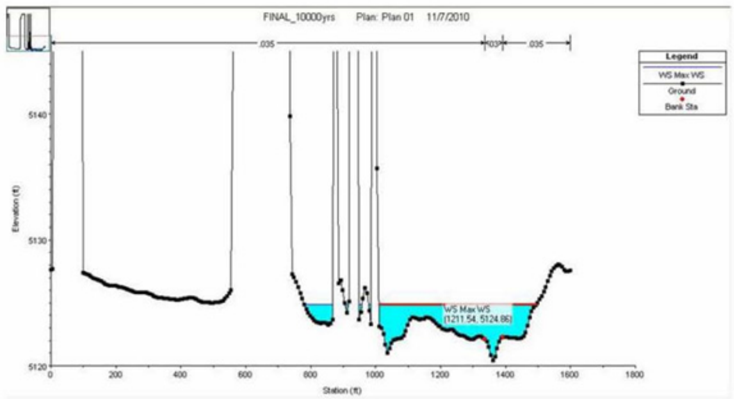


Figure 2.27. Maximum Water Level at Cross-Section 286 from the 10,000-Year Return Period Event

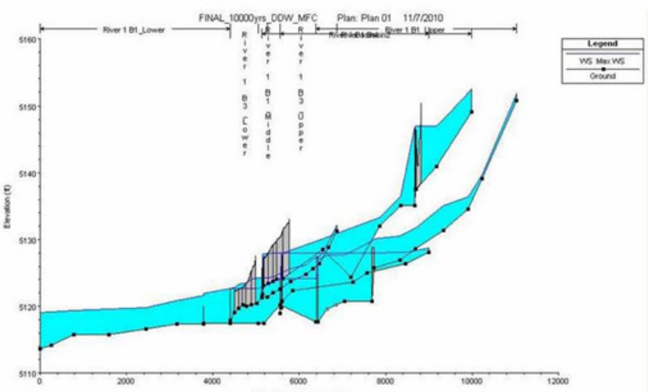


Figure 2.28. Combined Profiles of the Maximum Water Level for all Subbasins from the 10,000-Year Return Period Event (Note: gray shaded areas designate weirs.)

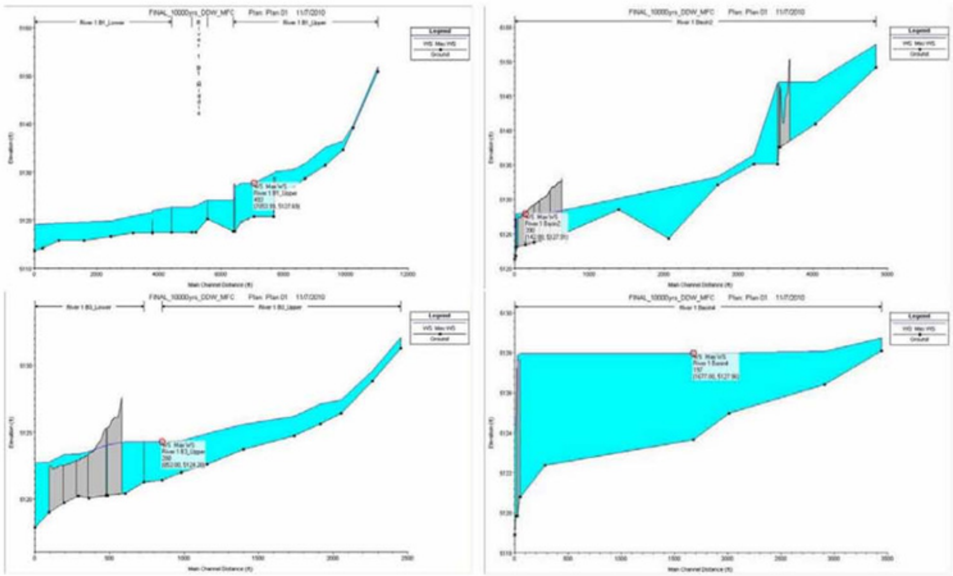


Figure 2.29. Individual Profiles of the Maximum Water Level for Each Subbasin from the 10,000-Year Return Period Event (Note: gray shaded areas designate weirs.)

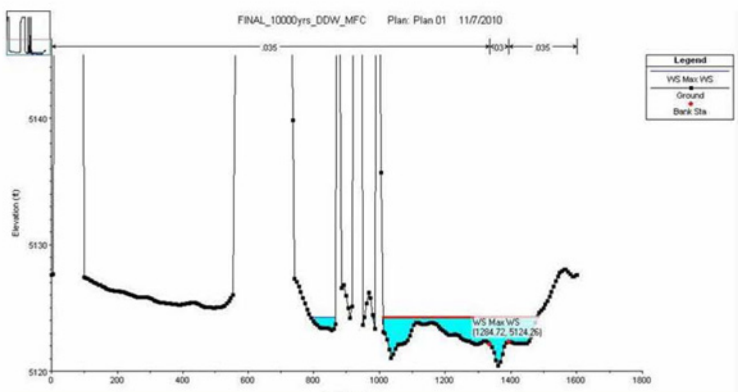


Figure 2.30. Maximum Water Level at Cross-Section 286 from the 10,000-Year Return Period Event

2.31

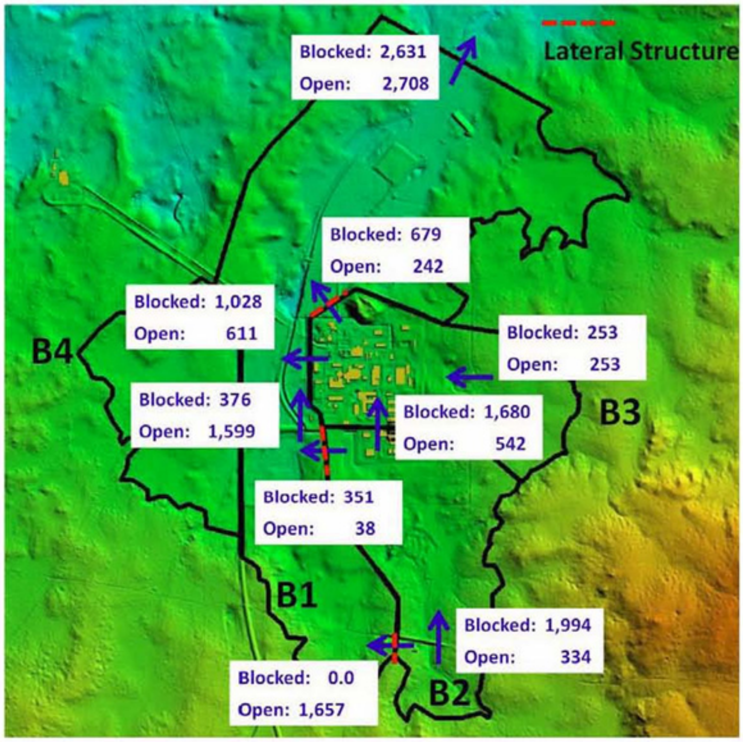


Figure 2.31. Flows Paths and Maximum Flows (cfs) for the 10,000-Year Return Period Event for Blocked and Open Cases

As previously noted, Figure 2.14 shows the layout of the HEC-RAS cross sections with respect to critical buildings in the MFC site. Table 2.2 shows the maximum water-surface elevations extracted at cross sections associated with the critical buildings. The 10,000-year results with the Blocked case shows six buildings at which the maximum water-surface elevation exceeds the floor elevations, although they are fewer than for the PMF cases. The maximum exceedance was about 0.7 ft, which was much smaller than in the PMF cases. The Open case shows two buildings at which the elevation is exceeded, with the maximum exceedance being about 0.1 ft. The diversion of flows from Basin 2 into Basin 1 in the Open case greatly reduces overtopping of the diversion dam from 1994 cfs to 334 cfs. The MFC site was still subject to flooding from locally generated runoff within the MFC subbasins themselves, particularly Basins 2 and 3. For example, the maximum flow at station 286 for the Open case was 611 cfs.

2.3.1.4 Sensitivity Analyses

To provide additional perspective of the relative significance of the results described previously, a limited sensitivity analysis was conducted on the hydraulic analysis evaluating the maximum water-surface elevations as a function of the assumed roughness coefficients used in the hydraulic analysis. The response of the water-surface elevations at the MFC was examined by adjusting two input parameters to the HEC-RAS model, as follows:

- downstream boundary condition slope (0.0002, 0.002, and 0.02). This was conducted to ascertain if there was a backwater effect produced by the assumed boundary friction value.
- Manning's roughness of the overbank portions of the cross section within the MFC site (0.035, 0.013). The 0.013 value is based on smooth asphalt (Chow 1959).

Table 2.3 presents the maximum water-surface elevations at cross-section station 286 for the four cases and the two sensitivity tests.

Table 2.3. Results from Sensitivity Analyses Comparing the Maximum Water-Surface Elevations at Cross-Section Station 286 at the MFC Site

	PMF		10,000-yr Return Period	
Sensitivity Test	Blocked	Open	Blocked	Open
Downstream Boundary Friction Slope				
0.0002 (base condition)	5125.55	5125.30	5124.86	5124.26
0.002	5125.55	5125.30	5124.86	5124.26
0.02	5125.55	5125.30	5124.86	5124.26
Manning's Roughness in the Overbanks of the MFC Site				
0.035 (base condition)	5125.55	5125.30	5124.86	5124.26
0.013	5125.19	5125.04	5124.67	5124.14

For the sensitivity test with the downstream boundary condition, the results at the MFC stations were insensitive to a change in boundary condition value (Table 2.3). This indicates that the channel properties in the lower end of Basin 1 govern the conveyance of floodwaters through the system. We also examined the influence of the assumption of the stormwater pond being filled by including it in the conveyance area. The results for this, too, were also found to be insensitive, but they are not included in Table 2.3.

For the sensitivity test of the Manning's roughness at the MFC site, the maximum water-surface elevation results at MFC station 286 were found to have decreased by less than 0.4 ft for the PMF and less than about 0.2 ft for the 10,000-year return period event (Table 2.3). Comparison at cross-section stations 286 and 287 of the maximum elevations from the Manning's roughness sensitivity test to the floor elevations in Table 2.2 shows that only the 10,000-year event for the Open case showed maximum elevations lower than the building floor elevations as follows:

- MFC-786 floor elevation = 5124.23 ft compared to the maximum water-surface elevation of 5124.14 ft at cross-section station 286.
- MFC-785 floor elevation = 5124.18 ft compared to the maximum water-surface elevation of 5124.15 ft at cross-section station 287.

2.3.2 TREAT Site

The TREAT site is located approximately 4700 ft northwest of the MFC site and lies in catchment area of 140 ac (Figure 2.32). A preliminary assessment of the TREAT site suggested that, given the relatively small contributing area and the height of the facility floor above the channel, the flood hazard was low. Results of the preliminary assessment were confirmed by a highly conservative, but somewhat simple analysis. The approach used for PMF analysis was to estimate flows for the drainage area and the local inflows, by transforming the maximum precipitation to runoff using the Rational Method with no precipitation loss. The estimated maximum runoff was input to HEC-RAS model at the upstream end of the catchment as a constant flow for the steady-state hydraulic modeling analysis. Coupled with the use of the maximum runoff, a series of model runs were also made over a range of flows that bracketed the estimated maximum runoff to provide information about how much flow would it take to reach the target elevation of 5121.85 ft.

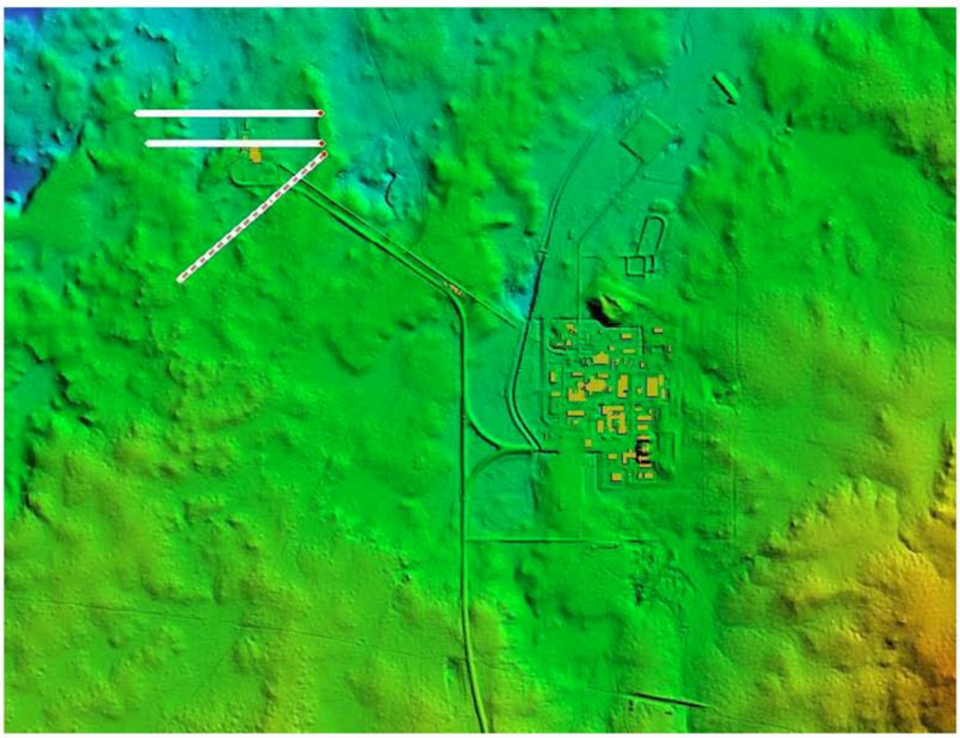


Figure 2.32. Cross Sections Used for the TREAT Site

The maximum precipitation intensity was 15.22 in./hr, which gave an estimated runoff of 2161 cfs. The bracketing flows examined ranged from 200 to 5000 cfs.

As was previously done for the MFC site, cross-section data input into HEC-RAS were extracted from LiDAR data of the TREAT subbasin, which included cross sections that characterize the presence of drainage channels, roads, and buildings (Figure 2.32).

The following assumptions were used for the analysis:

- Flow was one-dimensional using the HEC-RAS hydraulic model.
- The subbasins were lightly vegetated sagebrush (Manning's n = 0.035) with sandy channels (Manning's n = 0.030).

The water-surface elevation profile for the maximum flow is shown in Figure 2.33. The cross section that includes the TREAT site is shown in Figure 2.34, with a water-surface elevation of 5114.82 ft for the maximum flow.

The results from the series of bracketing flows are shown in Figure 2.35. Also shown is the result of the runoff from the peak precipitation intensity. The target elevation 5121.85 ft is not reached even with a flow of 5000 cfs, which is more than double the flow from runoff with the maximum PMP intensity.

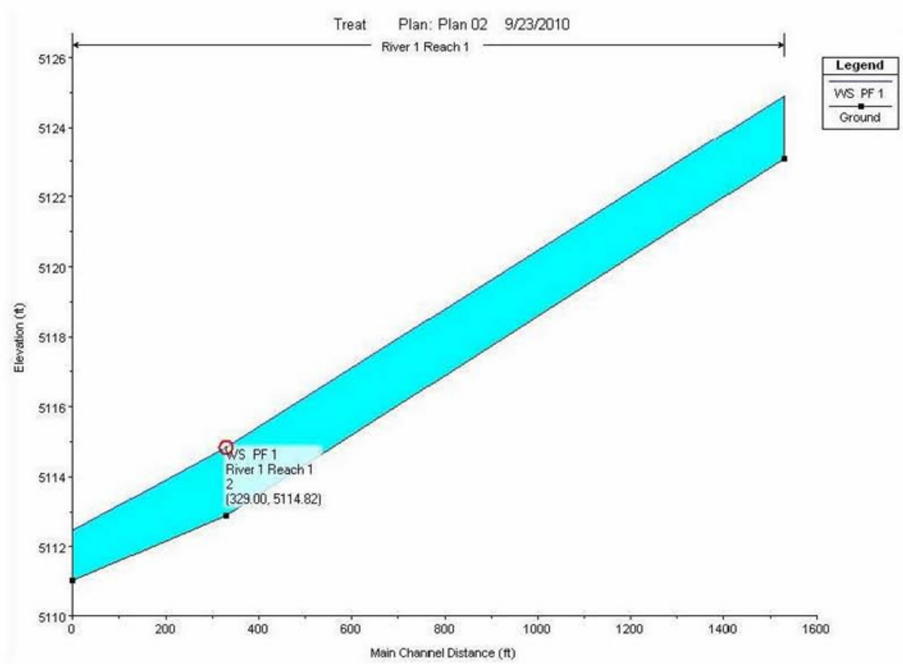


Figure 2.33. Profile and Location Where the Maximum Water Level Was Extracted for TREAT

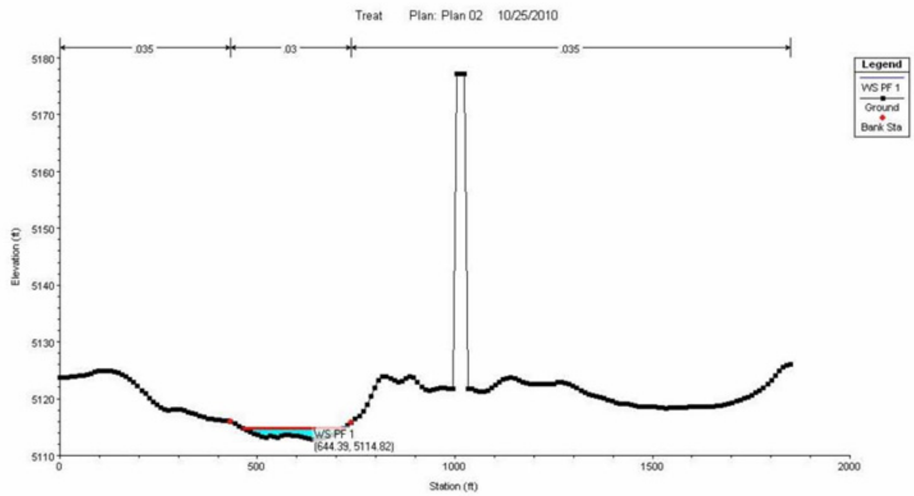


Figure 2.34. Inundation Areas and Maximum Water Level at the Middle Cross Section for TREAT Site

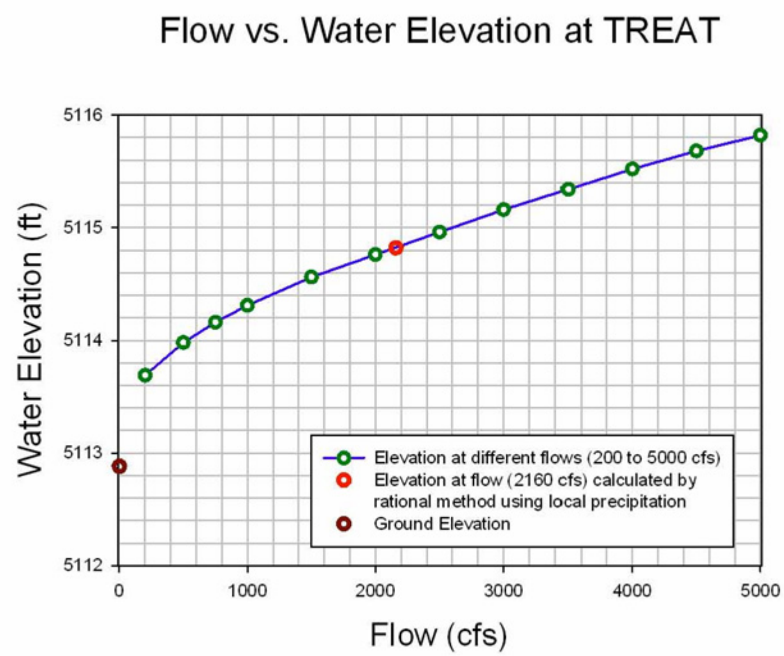


Figure 2.35. Comparison of Maximum Water-Surface Elevations for a Range of Flows and with a Flow Estimated from the Rational Method of the Peak PMP Intensity

3.0 Conclusions

The PMF and 10,000-year return period events were analyzed at the MFC site. The peak discharges for the PMF and 10,000-year events at the MFC site were 10,200 cfs and 2,880 cfs, respectively. Comparison of the maximum water-surface elevation results from the HEC-RAS model to floor elevations at critical building locations showed the following:

- Under the most conservative assumption used (i.e., PMF with the diversion channel and culverts blocked), nine locations were inundated to depths ranging from 0.10 ft to 3.20 ft.
- Under the least conservative assumptions used (i.e., 10,000-year event with the diversion channel and the culverts open), two locations (MFC-786 and MFC-785) were inundated to depths 0.03 ft and 0.10 ft, respectively.

It is noteworthy that the changes in maximum water-surface elevations at these two locations for the two events and different assumptions were relatively small. For example, for the PMF with the diversion channel and culverts blocked the maximum water-surface elevation at MFC-785 was 5125.63. For the 10,000-year event with the diversion channel and culverts open, the maximum water-surface elevation was only reduced 1.35 ft. Similarly, at MFC-786 the reduction was only 1.29 ft. This strongly suggests that maximum water-surface elevations are primarily determined by backwater effects from downstream hydraulic controls. An example is the road along the perimeter of the MFC site to the northwest that is overtopped and produces backwater effects at the MFC buildings during both the PMF and 10,000-year events.

As was noted in the report, analyses and results presented are based on a number of conservative assumptions in the absence of site-specific data and/or information. Examples include:

- use of the SCS UH because it had the highest peak discharge of the synthetic UHs evaluated
- use of the Rational Method to calculate local inflows to the hydraulic model, thus ignoring the attenuating affects of lag
- assuming zero infiltration losses.

In evaluating the flood hazard at the MFC site, under these conservative assumptions, during the 10,000-year event, the site is inundated at only two locations to depths of 0.10 ft. or less.

In addition, a limited sensitivity study of the assumed Manning's roughness coefficient showed that by reducing the coefficient from 0.035 to 0.013, the computed water-surface elevations for the 10,000-year event were reduced sufficiently that no critical building locations were inundated.

4.0 References

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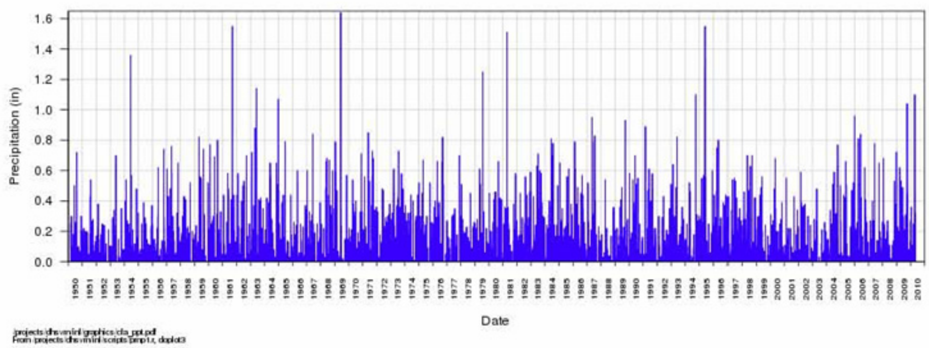
Appendix A

Additional Climate Data Plots

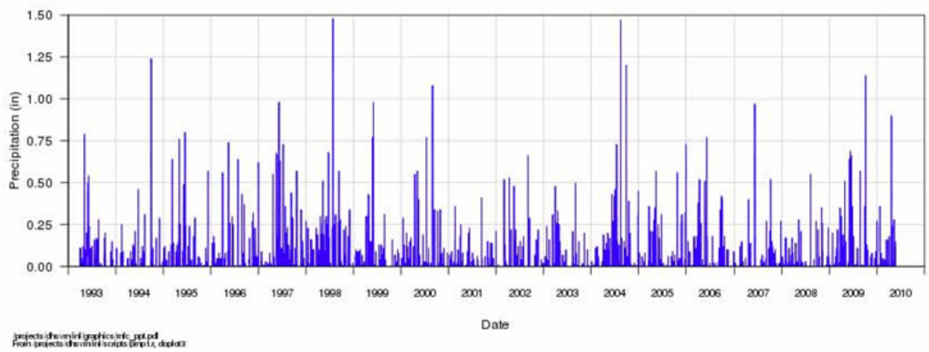
Appendix A

Additional Climate Data Plots

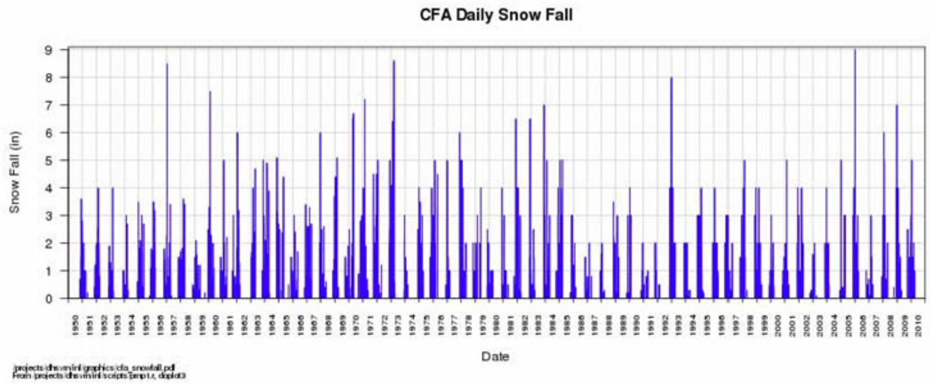
CFA Daily Total Precipitation

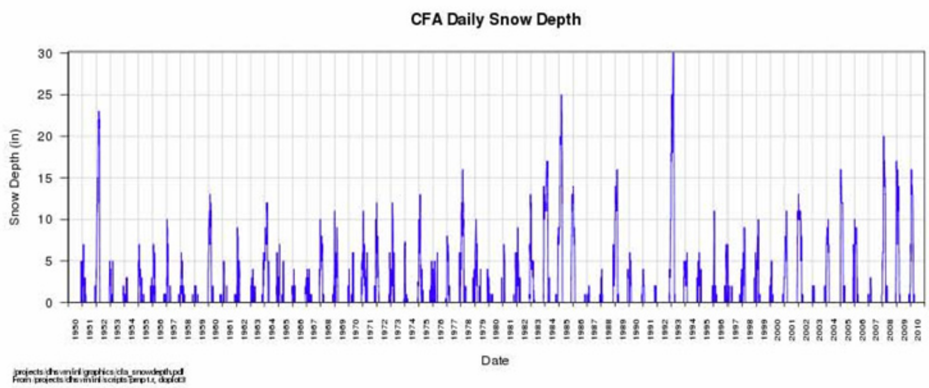


MFC Daily Total Precipitation



A.1


CFA Daily Snow Fall
Snow Fall (in)
Date


CFA Daily Snow Depth
Snow Depth (in)
Date

A.2

Appendix B

Terrain Data Processing

Appendix B

Terrain Data Processing

Two primary tasks were involved in pre-processing the available terrain data prior to performing the terrain and hydrologic analysis. First was the preparation of the source terrain data and second was the development and pre-processing implementation of an algorithm to overcome false terrain barriers. The objectives of the terrain data preparation were to 1) capture all the required areas (as defined by drainage basins), and 2) ensure all data share a common vertical datum of NAVD88 (North American Vertical Datum of 1988). The vertical datum issue became important because the extent of the catchment area was not fully covered by the available Light Detection and Ranging (LiDAR) data provided by the Idaho National Laboratory (INL). Standard 10-meter Digital Elevation Model (DEM) data from the U.S. Geological Survey (USGS) National Elevation Dataset were used to cover the additional extent. Because each of the two referenced terrain data sets were captured for different purposes and thus varying scales and degrees of accuracy are inherent to the data, it was determined that these data sets would be best used by keeping independent spatial domains and not mixing/overlapping the data.

Once the proper spatial extent of DEM coverage was achieved, the only data set requiring conversion into the NAVD88 vertical datum was the USGS 10-meter DEM data set. The DEM data were converted to ASCII X,Y,Z format, transformed from NGVD29 (National Geodetic Vertical Datum of 1929) to NAVD88 datum using CorpsCon v.6 software, and reimported as a raster data set that was used in some of the remaining tasks. The LiDAR data were converted from 'las' files to simple ASCII X,Y,Z format using Global Mapper 10. The 'las' file format is a common binary format used in dealing with LiDAR data sets. These files were further refined for input into Anudem via pre-written C++ code. Compiled code was used versus scripting languages due to the size of the overall LiDAR data set of about 60 million points. Interpreted languages simply will not process files of this magnitude quickly and efficiently enough.

A problem with artificial terrain barriers was identified early on with the surface interpolation of the LiDAR data. These barriers impeded flow in certain areas, thus creating a surface that was not hydrologically correct. Because the goal was to be able to produce accurate drainage basin boundaries, it was imperative that these barriers be dealt with in a realistic manner. Both high resolution aerial photography and the LiDAR surface were used to identify problem areas in the data. The problem with a LiDAR terrain model is that it does not recognize the presence and/or absence of these obstructions. LiDAR data can be so sensitive to change that nothing more than a scrub or small boulder could block the flow of a small stream. Simple line features were created at the problem areas to allow flow to pass and a more accurate terrain model to be developed.

Once the hydrologically correct terrain model was completed the hydrologic processing was done using the ArcHydro tools. An artificial stream network was modeled for both the LiDAR and USGS terrain models. A single pour point was identified for the six drainage areas that surrounded the upstream portion of the MFC site. The location of the pour point was determined by looking at the drainage pattern of the modeled stream network. Points were chosen along what was considered to be major contributing drainage networks. Pour-point selection gave way to drainage basin delineation, which could then be used in deriving other hydrologic parameters for the Hydrologic Engineering Centers Hydrologic

Modeling System (HEC-HMS) model. Each of the prefixed 'W' basins was joined with the prefixed 'B' Hydrologic Engineering Centers River Analysis System (HEC-RAS) basins with the overlapping areas subtracted out (see Figure B.1). Total catchment area was calculated for each basin along with the path of the longest drainage channel. Both the length of the channel and its length from the geometric centroid of the basin were determined. Slopes were then calculated from the underlying terrain models and the determined main channel path. Two different kinds of slope values were needed. The first was the average slope for each of the six watershed (W) basins. This was done by creating a slope grid from the terrain model within each basin boundary. The second was the slope at each segment of the main channel path. These slope calculations were done by splitting the channel path into equal stream segments based on the terrain model resolution. A point feature was created at each segment split so that the underlying slope grid could be sampled and a slope value determined.

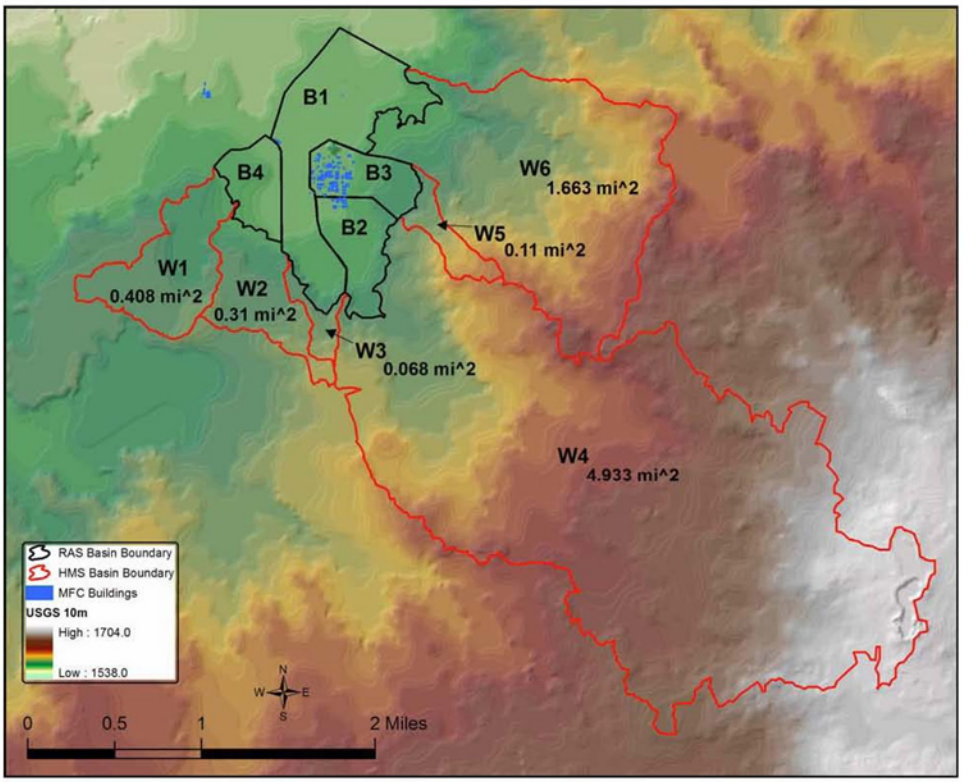


Figure B.1. The Basin Extents for the HEC-RAS and HEC-HMS Models

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Appendix B

Photographs of the January 1969 Overland Flood Event at MFC

Appendix B
Photographs of the January 1969 Overland Flood Event at MFC

An overland flow flooding event occurred at MFC in January 1969. Photographs of the flooding conditions and the impacts of flooding on the MFC Diversion Dam were taken by an INL Site photographer. A summary of photographs, as provided by the INL Site photographer, is provided below.

Flooding Conditions

9867- Flooding conditions 1/1969. Looking NE approaching EBR II on Taylor Blvd. Shows water backing up.
9868- Flooding conditions 1/1969. Looking south. Shows icing at bridge.
9869- Flooding conditions 1/1969. Looking south.
9870 thru 9882 - These photos show results of flooding conditions thru taken January 21 and 22, 1969) approaching EBR II.
9882- Highway 20 at NRTS site.

Flood Control Dam

9870- Flood control dam, looking ESE from overflow spillway. Engineers are standing at control culvert.

9871- Flood control dam looking E. from overflow spillway. Shows water backup by the dam.

9872- Flood control dam looking W. from control culvert. Vehicle in spill way (taken January 21, 1969).

9873- Flooding conditions. Looking N. from W. end of flood control dam.

9874- Flooding conditions. Shows water still flowing thru culvert after dam had washed out.

9875- Flooding conditions Shows washout of culvert January 21, 1969.
9876- Flooding conditions looking N.
9877- Flooding conditions. looking S.
9878- Flooding conditions. Water still flowing despite washout of culvert.

PROJECT

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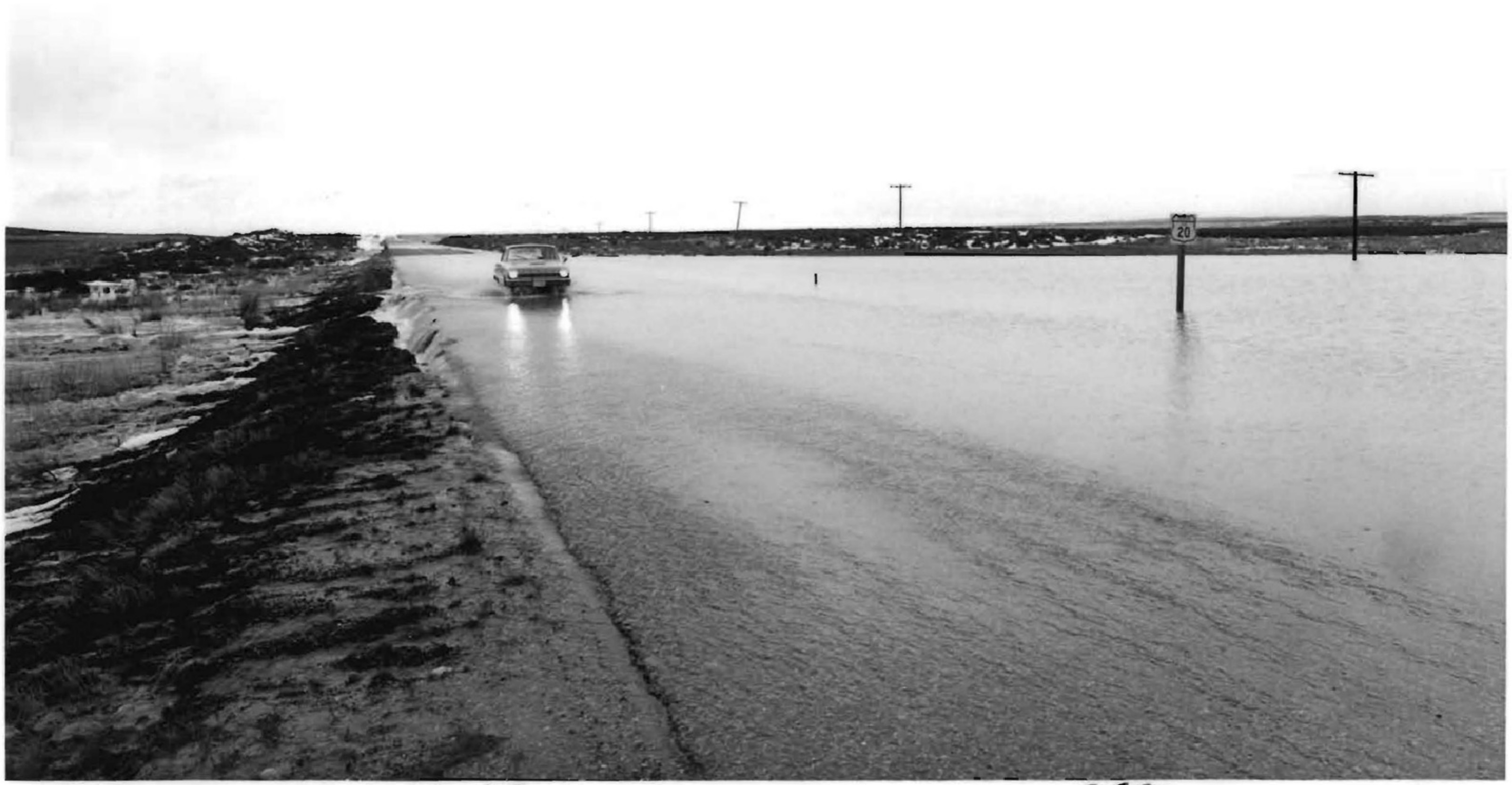

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