GO2-16-143
Enclosure 2

COLUMBIA GENERATING STATION
FLOODING HAZARD REEVALUATION REPORT

FLOOD HAZARD REEVALUATION REPORT

IN RESPONSE TO THE 50.54(f) INFORMATION REQUEST REGARDING NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING

for the
Columbia Generating Station

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1. PURPOSE

1.1 Background

In response to the nuclear fuel damage at the Fukushima-Daiichi power plant due to the March 11, 2011, earthquake and subsequent tsunami, the United States Nuclear Regulatory Commission (NRC) established the Near Term Task Force (NTTF) to conduct a systematic review of NRC processes and regulations, and to make recommendations to the NRC for its policy direction. The NTTF reported a set of recommendations that were intended to clarify and strengthen the regulatory framework for protection against natural phenomena.

On March 12, 2012, the NRC issued an information request pursuant to Title 10 of the Code of Federal Regulations, Section 50.54(f) (10 CFR 50.54(f) or 50.54(f) letter) (Reference 24) which included six (6) enclosures:

1. [NTTF] Recommendation 2.1: Seismic
2. [NTTF] Recommendation 2.1: Flooding
3. [NTTF] Recommendation 2.3: Seismic
4. [NTTF] Recommendation 2.3: Flooding
5. [NTTF] Recommendation 9.3: EP
6. Licensees and Holders of Construction Permits

In Enclosure 2 of information request (Reference 24), the NRC requested that licensees reevaluate the flooding hazards at their sites against present-day regulatory guidance and methodologies being used for early site permits (ESP) and combined operating license (COL) reviews.

On behalf of Energy Northwest Company, LLC, for the Columbia Nuclear Generating Station (CGS), this Flood Hazard Reevaluation Report (Report) provides the information requested in the March 2012, 50.54(f) letter; specifically, the information listed under the "Requested Information" section of Enclosure 2, paragraph 1 ("a" through "e"). The "Requested Information" section of Enclosure 2, paragraph 2 ("a" through "d"), Integrated Assessment Report, will be addressed separately if the current design basis floods do not bound the reevaluated hazard for all flood-causing mechanisms.

1.2 Requested Actions

Per Enclosure 2 of the NRC issued information request, 50.54(f) letter, CGS is requested to perform a reevaluation of all appropriate external flooding sources at CGS, including the effects of local intense precipitation (LIP) on the site, probable maximum flood (PMF) on streams and rivers, storm surges, seiches, tsunamis, and dam failures (as applicable). It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for ESPs and calculation reviews, including current techniques, software, and methods used in present-day standard engineering practice to evaluate the flood hazard. The requested information will be gathered in Phase 1 of the NRC staff's two-phase process to implement Recommendation 2.1, and will be used to identify potential "vulnerabilities." (See definition below.)

For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit the hazard evaluation along with an interim action plan that documents actions planned or taken to address the reevaluated hazard.

Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible to flooding due to the status of the flood protection features. The scope also includes evaluating those features of the ultimate heat sinks (UHS) that could be adversely affected by flood conditions and lead to degradation of the flood protection (the loss of UHS from non-flood associated causes are not included). It is also requested that the integrated assessment address the entire duration of the flood conditions.

A definition of vulnerability in the context of [Enclosure 2] is as follows: Plant-specific vulnerabilities are those features important to safety that when subject to an increased demand due to the newly calculated hazard evaluation have not been shown to be capable of performing their intended functions.

1.3 Requested Information

Per Enclosure 2 of the NRC-issued information request 50.54(f) letter, the Report should provide documented results, as well as pertinent CGS information and detailed analysis to include the following:

a) Site information related to the flood hazard. Relevant structures, systems and components (SSCs) important to safety and the UHS are included in the scope of this reevaluation, and pertinent data concerning these SSCs should be included. Other relevant site data include the following:

   - Detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, and site topography, as well as pertinent spatial and temporal data sets;
   - Current design basis flood elevations for all flood-causing mechanisms;
   - Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance;
   - Changes to the watershed and local area since license issuance;
   - Current licensing basis flood protection and pertinent flood mitigation features at the site;
   - Additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc.).

b) Evaluation of the flood hazard for each flood-causing mechanism, based on present-day methodologies and regulatory guidance. Provide an analysis of each flood-causing mechanism that may impact the site, including LIP and site drainage, flooding in streams and

rivers, dam breaches and failures, storm surge and seiche, tsunami, channel migration or diversion, and combined effects. Mechanisms that are not applicable at CGS may be screened out; however, a justification should be provided. Provide a basis for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data.

c) Comparison of current and reevaluated flood-causing mechanisms at CGS. Provide an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood-causing mechanism. Include how the findings from Enclosure 2 of the 50.54(f) letter (i.e., Recommendation 2.1, Flood Hazard Reevaluations) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood-causing mechanisms, include how this finding was determined.

d) Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment described below, if necessary.

e) Additional actions beyond Requested Information Item 1.d taken or planned to address flooding hazards, if any.

2. SITE INFORMATION

CGS is located in the southeast area of the U.S. Department of Energy's (DOE) Hanford Site in Benton County, Washington. CGS is approximately 3 miles west of the Columbia River at River Mile 352, approximately 10 miles north of North Richland, 45 miles downstream from Grant County Public Utility District (PUD) Priest Rapids Dam, 18 miles northwest of Pasco, and 21 miles northwest of Kennewick. CGS general location is presented in Figure 1.

Figure 1. CGS General Location Map

Topographic relief at CGS is low and relatively flat, with a mean station reactor floor elevation of 441 feet (ft) Mean Sea Level (MSL), as reported in the CGS Final Safety Analysis Report (FSAR) (Reference 2). Elevations in this report refer to National Geodetic Vertical Datum of 1929 (NGVD 29) and North American Vertical Datum of 1988 (NAVD 88). All referenced calculations and computations were conducted in NAVD 88. The FSAR identifies the critical elevation at CGS as 441 ft-MSL. The NGVD 29 was originally named the Mean Sea Level Datum of 1929 (Reference 16). Elevations referenced in the current FSAR that use the MSL are equivalent to NGVD 29. The datum shift of -3.4 ft from NAVD 88 to NGVD 29 was calculated using the web-based program VERTCON v2.1 developed by the National Geodetic Survey.

ft-MSL = ft-NGVD 29

ft-NGVD 29 = ft-NAVD 88 – 3.4

The relevant flooding data for CGS include the following:

2.1 Current Design Basis Flood

The current design basis flood elevation for CGS is 433.3 ft-MSL, which includes coincident wind-wave activity. This is less than the east spray pond overflow weir at elevation of 434.5 ft-MSL and the CGS reactor floor elevation of 441 ft-MSL (Reference 2).

The following is a list of flood causing mechanisms and their associated water surface elevations (WSEs) that are considered in the CGS current licensing basis (CLB).

2.1.1 LIP

Local thunderstorms can produce short duration rains, which have the potential for causing serious flooding. To provide adequate surface drainage during severe precipitation conditions, such as heavy rainfall and fast snowmelts, a system of catch basins and dry wells is constructed with inlet elevations lower than the finished floor elevation of the nearest building(s). The roofs of safety-related buildings are designed to handle LIP events with adequate drainage. In the event that the roof drains are completely blocked during the precipitation event, overflow scuppers limit the depth of water to within the design load carrying capability of the roofs. Those safety-related structures that do not have this relief capability are structurally able to carry the entire precipitation accumulations (Reference 2).

2.1.2 Flooding in Streams and Rivers

Analyses of the PMF are consistent with the requirements of Regulatory Guide 1.59, Revision 2. The FSAR indicates an unregulated PMF flow rate of 1,600,000 cubic ft per second (cfs) in the Columbia River at CGS (Reference 2). Adjustment of the flood profiles for the Hanford region results in a regulated PMF of 1,440,000 cfs and a water level of 390 ft-MSL at the Seismic Category II makeup water structure. As indicated in the FSAR (Reference 2), the design basis flood for CGS area results from the PMP event on the adjacent drainage basin and not from flooding of the Columbia River. The methodology for predicting the total amount of precipitation requires adding together the convergence PMP and the orographic PMP to obtain a single precipitation for a general storm. The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) standard-step procedure for seven (7) cross sections was utilized to determine the WSE from the PMF event on the adjacent drainage basin, which was determined to be 431.1 ft-MSL (Reference 2).

2.1.3 Dam Breaches and Failures

There are seven dams upstream and four dams downstream of CGS on the main stream of the Columbia River within the U.S. The current license basis dam failure considered a flood from a breaching of Grand Coulee Dam in lieu of a seismically induced flood. A massive hydraulic failure of the Grand Coulee Dam with a release of 8,800,000 cfs was considered for analyses of floods resulting from potential dam failures. Following the assumed failure of the Grand Coulee Dam, all downstream dams between Grand Coulee Dam and CGS suffer some degree of failure and release their storage reservoirs to the flood. The effect of potential dam failure on the water levels at CGS was determined using the assumption that the Columbia River is at flood stage with a standard

project flood (SPF) of 570,000 cfs. A base flow of 50,000 cfs was assumed above the mouth of the Snake River for the dam failure flood (Reference 2).

The failure of Arrow and/or Mica Dams in Canada could result in greater releases of storage in terms of volume than that from the Grand Coulee Dam, but the effects of such postulated releases are mitigated by a combination of valley storage and critical (flow limiting) valley cross sections. The river channel restrictions at Trail, British Columbia, would restrict river flow to about 3,100,000 cfs, regardless of the postulated dam failure. A major failure upstream would result in this maximum flow for many days, causing overtopping of Grand Coulee Dam. An analysis by the U.S. Bureau of Reclamation concluded that overtopping which might result from the failure of upstream dams will not cause failure of either the Grand Coulee Dam or the Forebay Dam (Reference 2).

The failure of the Grand Coulee Dam represents an upper limit to seismically induced failures. This failure would initiate a catastrophic flood, which would be augmented by the failure of the earthen portions of downstream dams and subsequent release of the storage pools behind them. This flood would have an outfall peak of 8,800,000 cfs at Grand Coulee Dam at the moment of breaching, and a peak discharge at River Mile (RM) 338 (Richland) of 4,800,000 cfs.

The resulting dam breach elevation at RM 350 is 422 ft-MSL. An additional 2-ft allowance was included for wind and wave action, and adequate margin exists between the resultant flood elevation and the plant elevation of 441 ft-MSL (Reference 2).

2.1.4 Storm Surge

The FSAR (Reference 2) indicates flooding due to surges is not applicable. CGS has an inland location and does not connect directly with any of the water bodies considered for meteorological events associated with a storm surge. Flooding due to a surge is not plausible at CGS.

2.1.5 Seiche

The FSAR (Reference 2) indicates flooding due to seiche is not applicable. CGS has an inland location and does not connect directly with any of the water bodies considered for meteorological events associated with a seiche. Flooding due to a seiche is not plausible at CGS.

2.1.6 Tsunami

The FSAR (Reference 2) indicates flooding due to tsunamis is not applicable. CGS is not adjacent to any coastal area; furthermore, it has an inland location and does not connect directly with any of the water bodies considered for tsunami events. Flooding due to a tsunami is not a plausible at CGS.

2.1.7 Ice-Induced Flooding

Historically, the Columbia River has never experienced complete flow stoppage or significant flooding due to ice blockage, so no instances of complete stoppage have occurred. Periodic ice blocking has caused reduced flows and limited flooding for only relatively short periods of time. Therefore, it was concluded that ice jam flooding potential is insignificant for CGS (Reference 2).

2.1.8 Channel Migration or Diversion

The Columbia River flow in the Hanford reach is controlled to a large extent by regulation of the upstream reservoir projects. Columbia River flow is controlled by the operation of upstream reservoirs by the USACE. The riverbed in the vicinity of CGS is well defined. Therefore, it is very unlikely that it would be diverted from its present location by natural causes. Any possible effect on water supply to the makeup water pump house from riverbed changes would come from extremely slow changes which can be corrected if and when they occur (Reference 2).

2.1.9 Combined Effect Flood (including Wind-Generated Waves)

Procedures published by the USACE were used to determine the wind-wave activity. The maximum wave height was calculated to be 4.0 ft, the wind setup was computed to be 0.3 ft, and the maximum wave runup 1.9 ft. The resulting design WSE including coincident wind-wave activity is 433.3 ft-MSL, which is less than the east spray pond overflow weir elevation of 434.5 ft-MSL (Reference 2).

The allowance for simultaneous wind and wave action on the Columbia River is 2 ft. The resulting WSE for the Columbia River including coincident wind-wave activity is 424 ft-MSL (Reference 2).

2.2 Flood-Related Changes to the Licensing Basis

2.2.1 Flood-Related Changes to the Licensing Basis since License Issuance

There have been no flood-related changes to the CGS licensing basis with respect to an external flooding event.

2.2.2 Flood Protection Changes (Including Mitigation) Since License Issuance

There have been no flood protection changes made since license issuance. No physical modifications have been installed specifically in support of the external flooding response.

2.3 Changes to the Watershed since License Issuance

The Columbia River drains an area of approximately 258,000 square miles, lying to the west of the Continental Divide in the northwestern part of the U.S. (85%) and southwestern part of Canada (15%) (Reference 2). The Columbia River watershed upstream of CGS has a total drainage area of approximately 98,000 sq. miles. The Yakima and Snake River watersheds, downstream of CGS and tributary to the Columbia River, have watershed drainage areas of 6,156 sq. miles and 108,000 sq. miles, respectively (Reference 5). Based on a review of aerial images, the most significant change to the watershed is the expansion of urbanized areas. These areas comprise a very small fraction of the watershed area. Even if watershed changes are anticipated in the Columbia River Basin, the impact of any changes will be insignificant due to the regulation effect of the large number of flow regulation structures upstream of CGS.

2.4 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features at CGS

All safety-related facilities are housed in Seismic Category I structures protected from flooding and designed to withstand the static and dynamic forces of all postulated floods. CGS can be safely shut

down in the event of any flood and maintained in a safe condition for flood levels up to 441 ft-MSL (Reference 2).

The PMF elevation of the Columbia River is estimated to be 390 ft-MSL, so emergency flood protection procedures are not necessary for Columbia River PMF (Reference 2). For flood levels between 373 ft-MSL and 440 ft-MSL, any mode of operation is possible with no additional protective measures. The plant is maintained in a safe condition and is safe for any shutdown condition (Reference 2).

All non-safety-related facilities are above the limiting case flood (LCF) elevation, except for the makeup water pump house, which would not be affected by flooding, and thus would not affect any safety-related equipment and would not hinder the safe shutdown of the plant. The approximate finished grades at all Seismic Category I structures except the spray ponds are at elevation 440 ft-MSL. The finished grade of the spray ponds is 434 ft-MSL. The current design basis flood elevation for CGS is 433.3 ft-MSL, which includes coincident wind-wave activity. This is less than the east spray pond overflow weir elevation of 434.5 ft-MSL and the CGS reactor building floor elevation of 441 ft-MSL (Reference 2).

Seismic Category I structures are designed to withstand the static and dynamic forces which could result from a flood due to a breach of Grand Coulee Dam. Since this represents the LCF, the structures are also considered secure against the forces due to the lower PMF. The access openings to all seismic Category I structures are located well above all flood water elevations, including those due to wind and wave action.

The groundwater table elevation at CGS is approximately 380 ft-MSL. The groundwater design basis is 420 ft-MSL (Reference 2). Seismic Category I structures house safety-related systems and components. The lowest floor level of these structures, except the standby service water pump houses, is above the groundwater design basis. The standby service water pump houses are designed to resist the increased hydrostatic pressure from the groundwater design basis elevation. Seismic Category I piping and electric conduit penetrations that are below grade are above the design basis groundwater table, and sealing against groundwater pressure is therefore not required. However, all pipes penetrating exterior walls are waterproofed sealed by boots installed on both sides of the wall penetration; all electrical conduit penetrations are through-wall waterproof sealed using silicon foam.

3. SUMMARY OF FLOOD HAZARD REEVALUATION

NUREG/CR-7046 *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America* (Reference 23), which references guidance from the American Nuclear Society (ANS), states that a single flood-causing event is inadequate as a design basis for power reactors and recommends that combinations should be evaluated to determine the highest flood water elevation at CGS. For CGS, the combination that produces the highest flood water elevation is the effects of LIP. Energy Northwest Calculation No. CE-02-13-22 (Reference 8) evaluated the effects of LIP.

The CGS FSAR Section 2.4 (Reference 2) provides elevations in MSL datum. The CGS flood hazard reevaluation calculations provide elevation results based on the NAVD 88 and NGVD 29. Prior to 1973, NGVD 29 was named the Mean Sea Level Datum of 1929. Elevations in the CGS flood hazard reevaluation calculations referring to NGVD 29 are equivalent to MSL and directly compared to the elevations referenced in the CGS FSAR (Reference 2) that use MSL.

The methodology used in the flooding reevaluation for CGS is consistent with the following standards and guidance documents:

- NRC Standard Review Plan, NUREG-0800, revised March 2007 (Reference 22).
- NRC Office of Standards Development, Regulatory Guides, RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976 (Reference 20).
- NRC RG 1.59, Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977 (Reference 21).
- NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," dated November 2011 (Reference 23).
- American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8-1992), dated July 28, 1992 (Reference 1)
- NEI Report 12-08, Overview of External Flooding Reevaluations (Reference 13).

The following provides the flood-causing mechanisms and their associated WSEs that are considered in the CGS flood hazard reevaluation.

3.1 Local Intense LIP Analysis (Reference 8 and Reference 10)

Calculation No. CE-02-14-10 (Reference 10) evaluates site-specific PMP depths used as input to the LIP analysis. Calculation No. CE-02-13-22 (Reference 8) evaluates the runoff and the resulting WSE due to a LIP event at CGS.

3.1.1 Basis of Inputs

The inputs used in the LIP analysis are based on the following:

- A site-specific storm-based approach PMP analysis based on CGS elevation and location was performed to define the PMP depths. The site-specific PMP analysis utilized actual data from extreme rainfall events within a Pacific Northwest watershed domain that included portions of Washington, Oregon, California, Nevada, Utah, and Idaho.
- The Digital Terrain Model (DTM) was developed from an aerial survey performed by the Sanborn Map Company. The grid system for computation was developed based on a selected cell size of 10 ft and optimized to provide the best possible combination of topographic data resolution and acceptable model run time. The topographic data were processed using ArcGIS 10.1.
- The obstructions at CGS were identified using the aerial image for CGS and the AutoCAD survey file from Sanborn Map Company. Two types of obstructions were modeled: (1) buildings and structures that completely block the water passage regardless of calculated WSE; and (2) vehicle and security barriers that may be overtopped if the calculated WSE is above the top of the barrier.
- Flow velocities and depths in overland flow are affected by topography and grid element roughness. The land cover information was obtained using aerial images and the AutoCAD survey file from Sanborn Map Company.
- FLO-2D, a two-dimensional physical process and volume conservation model, was used to estimate the maximum water surface and water depth at CGS.
- For surface roughness coefficients the Manning's n-values are used in the analysis. The roughness coefficients are selected based on the land cover type identified using aerial topographical survey information and available aerial imagery. The Manning's n-values are selected following the suggested range for the overland flow runoff provided in the FLO-2D Reference Manual.
- Associated effects for hydrostatic and hydrodynamic loads were determined using the FLO-2D output depths and velocities.

3.1.2 Computer Software Programs

- ArcGIS Desktop 10.1
- AutoCAD Civil 3D 2012 Service Pack 1
- FLO-2D Pro Build 14.08.09
- SPAS Version 9.5

3.1.3 Methodology

Site-Specific PMP

The steps for the CGS site-specific PMP analysis using a storm-based approach in accordance with the most recent NWS Hydrometeorological Report No. 57, HMR-57 (Reference 14) and the World Meteorological Organization (WMO) Manual for PMP determination (Reference 26 and Reference 27) are as follows:

- Identify a set of storms, which represent rainfall events that are LIP-type events. This includes storms where extreme heavy rainfall accumulates over short durations at a given location.
- Each storm event was then maximized in-place to produce a scenario representing how much larger the rainfall could have been had all atmospheric processes been combined in ideal conditions.
- Each storm is then transpositioned from its original location to CGS. In this transposition process, differences in moisture between the original location and CGS are accounted for and quantified.
- The greatest depth of the total adjusted rainfall of all transpositionable storms becomes the LIP depth for CGS at hourly increments up to 6 hours.
- Sub-hourly increments were determined using the ratios from HMR-57.
- Cumulative LIP values were plotted on a smooth curve, which was used to interpolate 15-minute increment values.
- Five temporal distributions of the 15-minute incremental LIP values were developed that included centering the highest 15-minute increment at the front, one-third, center, two-thirds, and end position within the 6-hour storm duration, while placing the next highest values on alternating sides of the highest 15-minute increment.

Effects of LIP

The Effects of LIP analysis uses a two-dimensional (2D) hydrodynamic model, the FLO-2D model (FLO-2D). FLO-2D is a volume conservation model. The FLO-2D model simulates open channel flow through a numerical approximation of the shallow water equations. Flood wave progression over the flow domain is controlled by topography and resistance to flow. Flood routing in two dimensions is accomplished through a numerical integration of the equations of motion and the conservation of fluid volume.

A two-dimensional model is appropriate and better suitable model compared to a one-dimensional model to simulate the overland flow conditions at CGS, which are sheet flow, shallow concentrated flow, and open channel flow. The two-dimensional model determines the flow direction based on well-defined ground topography, where the one-dimensional model requires the flow direction to be assigned. The one-dimensional models, such as unsteady-state HEC-RAS model, are capable to utilize similar computational approaches as the FLO-2D model (equations of motion and volume conservation). However, because the flow direction is initially assigned, the model forces water flows

in the assigned general direction rather than determining the direction. Additionally, the flow path in the one-dimensional model is represented by cross sections along the assumed flow direction and the averaged cross sections are utilized for the computational processes. On the other hand, the two-dimensional model uses a grid to represent the ground surface. Each grid element is treated as a computational cell and the hydraulic relationships are determined for each cell depending on the hydrologic and hydraulic properties of the cell itself and the surrounding cells. The ground is closely represented in the two-dimensional model because each grid element is assigned a corresponding ground surface elevation, roughness coefficient, and, when applicable, reduction factor(s) to account for obstructions (buildings, walls, etc.).

The steps for CGS LIP analysis using FLO-2D are as follows:

- Create a grid system using ground surface topographical data.
- Assign properties/details to the model such as computational boundary and outflow elements.
- Specify roughness coefficients (Manning's coefficients) corresponding to the site's land cover type (e.g., concrete, grass, etc.).
- Identify obstructions completely blocking water flow (i.e., buildings).
- Identify obstructions diverting water flow (i.e., vehicle barrier). The model boundary of FLO-2D and the obstructions at CGS are shown in Figure 3.
- Assign precipitation inflow to the model (the five temporal distributions of LIP).
- Perform the FLO-2D computation.
- Analyze the results produced by FLO-2D.
- The ground elevation of each CGS-protected area survey point of interest (POI) is then compared with the calculated WSE to determine the water depth.

Following the guidance outlined in NUREG/CR-7046 the runoff losses are ignored. The roof rainfall is assumed to be contributing to the overland runoff. The drainage system at CGS is assumed to be non-functional at the time of the LIP event.

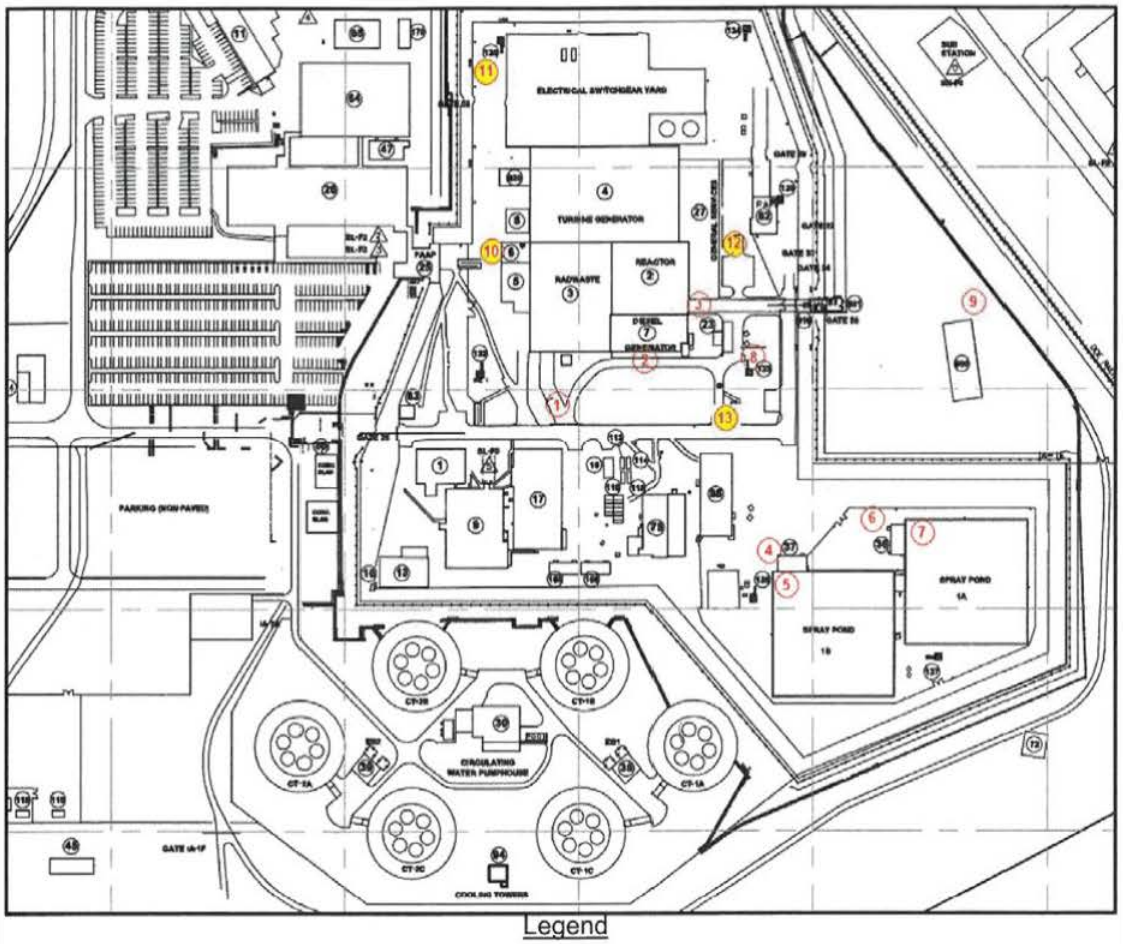
The steps for CGS LIP analysis associated effects are as follows:

- Obtain depth and velocity output from the FLO-2D modeling.
- FEMA guidance (Reference 11) is utilized for calculating hydrostatic and hydrodynamic pressure.
- Other associated effects are discussed qualitatively and determined to be insignificant based on the shallow depths and low velocities.

3.1.4 Results

The site-specific PMP analysis resulted in input LIP values that have a cumulative depth of 8.21 inches for the total 6-hour duration. The 1-hour depth is 6.29 inches and the 15-minute depth is 3.15 inches.

Following the guidance of NUREG/CR-7046, Appendix B, a front-peaking temporal distribution was utilized in the FLO-2D model to calculate LIP water surface elevation at CGS. Calculation No. CE-02-13-22 (Reference 8) evaluates a total of 23 POIs at CGS, which includes both safety and non-safety related structures. The power block buildings and the location of the doors that could potentially provide a pathway for the floodwaters are identified as Protected Area Survey Points (PASP), as shown in Figure 2.



Legend

1. High point near Radwaste Building truck ramp
2. DG building entrance doors (3)
3. Reactor Building crane bay door
4. Service Water A pumphouse personnel door
5. Service Water A spray pond wall (not scupper)
6. Service Water B pumphouse personnel door
7. Service Water B spray pond wall (not scupper)
8. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)
9.
10. Benchmark in front of TSC entrance (N11900, E1580, el 439.72)
11. Benchmark in by NW remote air intake (no data on marker)
12. Benchmark in front of GSB entrance (N11900, E1100, el 440.12)
13. Benchmark in front of Bldg. 88 (couldn't get cap off to read marker)

Figure 2. CGS Protected Area Survey Points (PASP)

The LIP results, including the depth and duration of flooding at each designated point, are provided in Table 1. The LIP maximum water surface elevations at CGS varies between 435.14 ft-NGVD 29 and 443.27 ft-NGVD 29. The calculated maximum water depths vary between 0.03 ft and 0.79 ft. The minimum ponding depth of 0.025 ft (surface detention) is used to initiate flow routing in the FLO-2D model. This flow depth is a reasonable minimum value to adapt as a starting depth for depth-duration analysis based on FLO-2D Data Input Manual recommendations and guidelines. The depth-

duration is the length of time in hours to which the flood of depth above 0.025 ft occurred on the surface.

The water surface elevation results at PASP Nos. 1, 3, 4, 17, 18, and 20 exceed the critical elevation of 441 ft-NGVD 29. However, these results do not indicate flooding of CGS safety-related SSCs. The topographic contours in the vicinity of these reference points indicate positive runoff away from the reference point. The minimal depth of flooding at the short duration is due to the peak intensity rainfall directly on the modeled grid cell.

The LIP results in a depth of 0.03 ft at the railroad bay door (PASP No. 3) for less than 1 hour. The railroad bay door would not likely be compromised by the short duration and limited depth of LIP flooding. However, assuming small amount of floodwater entered through the railroad bay door, it will not impact any safety-related equipment. No safety-related equipment located within the Reactor Building railroad bay would be adversely affected by a small amount of water on the floor. Additionally, the access doorway within the Reactor Building railroad bay has a threshold that will prevent the water from entering the rest of the Reactor Building.

The LIP results in a depth of 0.03 ft at the Diesel Generator Tank Access (PASP No. 20) for less than 1 hour. The Diesel Generator Tank Access is located at a high spot. The Diesel Generator Tank Access is capable of withstanding much higher water levels and is not compromised by the limited depth and duration of LIP flooding.

The LIP results in a depth of 0.03 ft at the Service Water B pump house personnel door (PASP No. 4) for less than 1 hour. The concrete pad in front of the personnel access door is about 25 sq. ft and it slopes away from the door. There is a drop off from the slab down to gravel (on three sides) of at least 2 inches. The door has a weather strip across the bottom. There is also a threshold across the bottom that is water proof and approximately one-inch high. Inside the doorway, the hallway concrete floor rises slightly for about six feet. However, assuming floodwater entered through the personnel door, it would flow on the floor to grating, under which there is no safety-related equipment that will be impacted. The entry door would not likely be compromised by the short duration and minimal depth of flooding. However, assuming floodwater entered through the Service Water B pump house personnel door, the effects are inconsequential.

The LIP results in a depth of 0.03 ft near the Radwaste Building truck ramp (PASP No. 1) for less than 1 hour. If any runoff enters the building, it will be most likely intercepted by grates across the top and bottom of the ramp just outside the building rollup door. There are no safety-related equipment at floor elevation 437 ft-NGVD29 that would be adversely affected by limited runoff.

The ISFSI Pad North (PASP No. 17), and the ISFSI Pad South (PASP No. 18) are not directly adjacent to safety-related SSCs.

The associated effects of the LIP are provided in Table 2. The hydrostatic pressures at the designated locations vary between 1.87 lb/sq. and 49.30 lb/sq. ft. The hydrodynamic pressure at the designated locations vary between 0.11 lb/sq. and 12.18 lb/sq. ft. The hydrostatic and hydrodynamic loads associated with the LIP event are insignificant due to the shallow depth and low velocity resulting from the LIP event at CGS. The shallow flow depth and low velocity due to LIP also would

not be expected to cause debris impact loading, sediment deposition, or erosion at CGS. A local intense precipitation event has no appreciable warning time except those provided by a weather (precipitation) forecast.

The groundwater table elevation at CGS is approximately 380 ft-MSL. The groundwater design basis is 420 ft-MSL. The LIP is a short duration event and is not expected to result in changes to the groundwater level. However, all piping and electric conduit penetrations that are below grade are waterproof sealed.

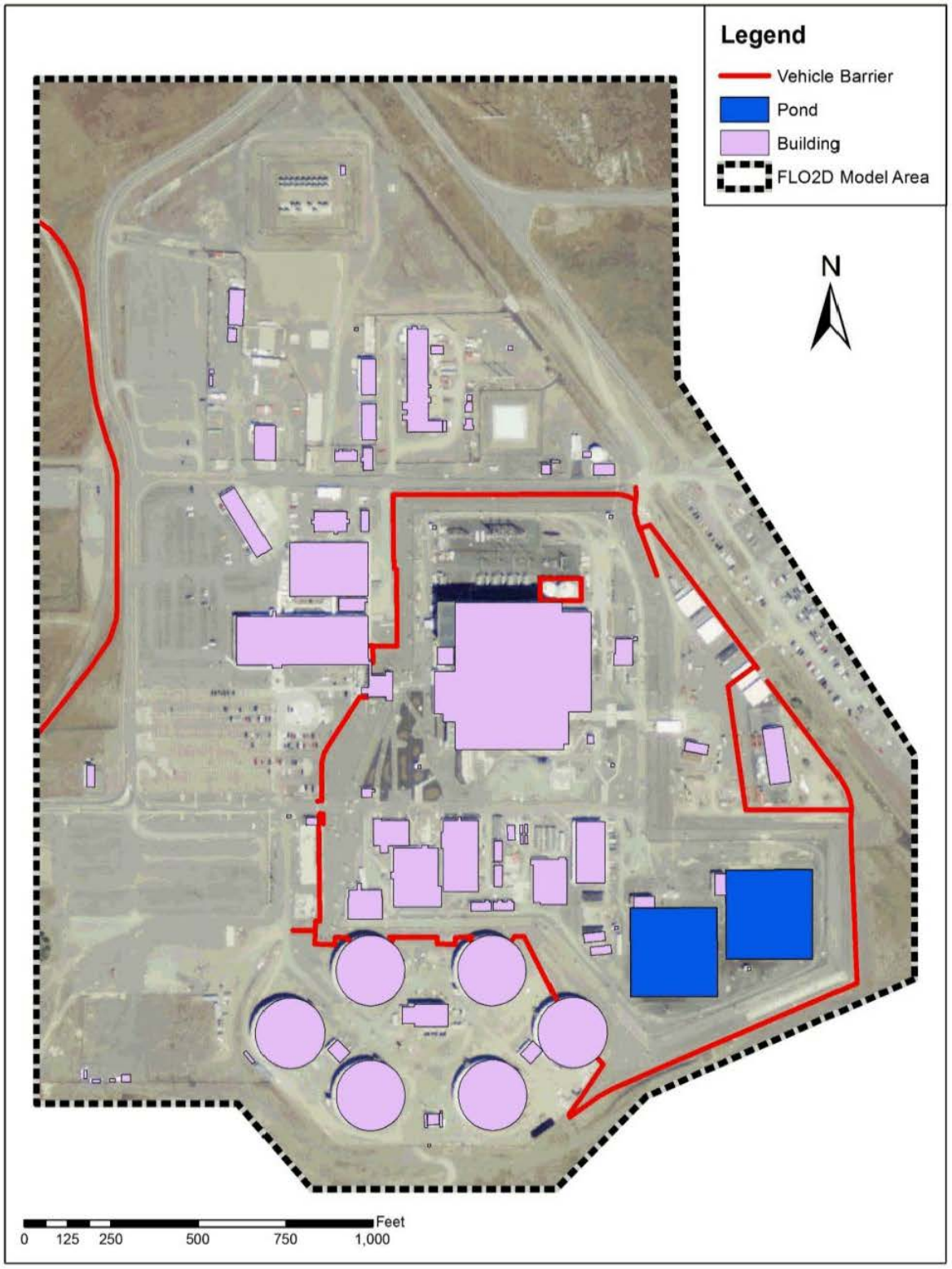


Figure 3. CGS Map for LIP

Table 1. Summary of LIP Results

No.	PASP	POI Elevation		Maximum Water Surface Elevation		Maximum Flow Depth from POI Elevation	Flooding Duration Over PASP
		NAVD 88 (ft)	NGVD 29 (ft)	NAVD 88 (ft)	NGVD 29 (ft)	(ft)	(hours)
1	High point near Radwaste Building truck ramp	444.65	441.21	444.68	441.24	0.03	0.73
2-1	DG Building Exterior Door (DG2)	444.26	440.82	444.30	440.86	0.04	2.25
2-2	DG Building Exterior Door (DG1)	444.29	440.85	444.34	440.90	0.05	2.76
2-3	DG Building Exterior Door (HPCS DG)	444.27	440.83	444.32	440.88	0.05	3.08
3	Reactor Building railroad bay door	444.52	441.08	444.55	441.11	0.03	0.89
4	Service Water B pump house personnel door	444.42	440.98	444.45	441.01	0.03	0.68
5	Service Water B spray pond wall	438.55	435.11	438.62	435.18	0.07	1.22
6	Service Water A pump house personnel door	442.42	438.98	442.45	439.01	0.03	0.82
7	Service Water A spray pond wall	438.47	435.03	438.58	435.14	0.11	1.70
8	Roof drain (storm sewer) header manhole MH-S5	443.61	440.17	443.77	440.33	0.16	2.55
9	Vehicle Barrier System high point	444.17	440.73	441.46	438.02	N/A*	N/A*
10	Benchmark in front of TSC entrance	443.25	439.81	444.04	440.60	0.79	5.20
11	Benchmark in by NW remote air intake	443.72	440.28	444.40	440.96	0.68	0.73
12	Benchmark in front of GSB entrance	443.67	440.23	444.37	440.93	0.70	3.76
13	Benchmark in front of Building 88	443.40	439.96	443.99	440.55	0.59	3.42
14	North Side of FLEX Building 82	443.85	440.41	444.02	440.58	0.17	2.35
15	North Side of FLEX Building 600	441.19	437.75	441.24	437.80	0.05	1.81
16	South Side of FLEX Building 600	441.36	437.92	441.39	437.95	0.03	0.94
17	ISFSI Pad (North)	446.66	443.22	446.71	443.27	0.05	0.73
18	ISFSI Pad (South)	446.66	443.22	446.69	443.25	0.03	0.73
19	Facilities Fuel Station	444.04	440.60	444.07	440.63	0.03	0.73
20	Diesel Generator Tank Access	445.52	442.08	445.55	442.11	0.03	0.71
21	FLEX Gasoline Storage Module	443.54	440.10	444.19	440.75	0.65	3.73

* N/A: Not Applicable. The maximum water surface elevation reported for the grid element is lower than the VBS height.

Table 2. Summary of Associated Effects

PASP		Maximum Flow Depth from Grid Elevation	Maximum Flow Velocity at Grid	Hydro-static Pressure	Hydro-dynamic Pressure
No.	Name	(ft)	(ft/s)	(lb/sq. ft)	(lb/sq. ft)
1	High point near Radwaste Building truck ramp	0.03	1.13	1.87	2.49
2-1	DG Building Exterior Door (DG2)	0.04	1.29	2.50	3.24
2-2	DG Building Exterior Door (DG1)	0.05	1.44	3.12	4.02
2-3	DG Building Exterior Door (HPCS DG)	0.05	1.44	3.12	4.04
3	Reactor Building railroad bay door	0.03	0.75	1.87	1.08
4	Service Water B pump house personnel door	0.03	2.26	1.87	9.90
5	Service Water B spray pond wall	0.07	0.95	4.37	1.74
6	Service Water A pump house personnel door	0.03	2.51	1.87	12.18
7	Service Water A spray pond wall	0.11	1.16	6.86	2.61
8	Roof drain (storm sewer) header manhole MH-S5	0.16	0.85	9.98	1.41
9	Vehicle Barrier System high point	0.43	1.82	26.83	6.42
10	Benchmark in front of TSC entrance	0.79	1.21	49.30	2.84
11	Benchmark in by NW remote air intake	0.03	0.81	1.87	1.29
12	Benchmark in front of GSB entrance	0.25	0.56	15.60	0.61
13	Benchmark in front of Building 88	0.59	0.28	36.82	0.15
14	North Side of FLEX Building 82	0.17	1.54	10.61	4.60
15	North Side of FLEX Building 600	0.05	1.22	3.12	2.89
16	South Side of FLEX Building 600	0.03	0.67	1.87	0.87
17	ISFSI Pad (North)	0.05	0.35	3.12	0.24
18	ISFSI Pad (South)	0.03	0.24	1.87	0.11
19	Facilities Fuel Station	0.03	0.62	1.87	0.74
20	Diesel Generator Tank Access	0.03	1.71	1.87	5.65
21	FLEX Gasoline Storage Module	0.65	0.48	40.56	0.45

3.2 Flooding in Streams and Rivers (Reference 4, Reference 5, and Reference 9)

The probable maximum flood is the hypothetical flood (peak discharge, volume and hydrograph shape) that is considered to be the most severe reasonably possible, based on comprehensive hydrometeorological application of probable maximum precipitation and other hydrologic factors favorable for maximum flood runoff such as sequential storms and snowmelt.

As outlined in the guidance provided in ANSI/ANS-2.8-1992 and in NUREG/CR-7046, Appendix H, the design basis from flood hazards should include several flood-causing mechanisms and combinations of these mechanisms. For the floods caused by precipitation events, the following should be examined:

Flooding in Rivers and Streams

Alternative 1 – Combination of:

- Mean monthly base flow
- Median soil moisture
- Antecedent of subsequent rain: the lesser of (1) rainfall equal to 40 percent of PMP, and (2) a 500-year rainfall
- The PMP
- Waves induced by 2-year wind speed applied along the critical direction

Alternative 2 – Combination of:

- Mean monthly base flow
- Probable maximum snowpack
- A 100-year snow-season rainfall
- Waves induced by 2-year wind speed applied along the critical direction

Alternative 3 – Combination of:

- Mean monthly base flow
- A 100-year snowpack
- Snow-season PMP
- Waves induced by 2-year wind speed applied along the critical direction

CGS is located approximately 3 miles west of the Columbia River in Benton County, Washington, as shown on CGS location map provided in Figure 4. CGS location map also identifies the following landmarks which are important to the PMF analysis:

- McNary Dam – Located on the Columbia River 60 miles downstream of CGS.

- Wallula Gap – A geological constriction of the Columbia River and floodplain located upstream of McNary Dam. Backwater effects from the natural constriction of PMF flows at the Wallula Gap heavily influence WSEs on the Columbia River at CGS.
- Snake River – The Snake River has its confluence with the Columbia River approximately 27 miles downstream of CGS. Snake River PMF flows contributing to the Columbia River create backwater effects at CGS.
- Yakima River – The Yakima River has its confluence with the Columbia River approximately 18 miles downstream of CGS. PMF flows from the Yakima River create backwater effects on the Columbia River at CGS.

Peak PMF discharge values on the Columbia, Yakima, and Snake Rivers due to PMP events occurring in respective watersheds were developed to determine PMF WSEs on the Columbia River at CGS. In addition, peak PMF discharge due to PMP events occurring on an adjacent local drainage basin was developed to determine the PMF WSE for the adjacent area.

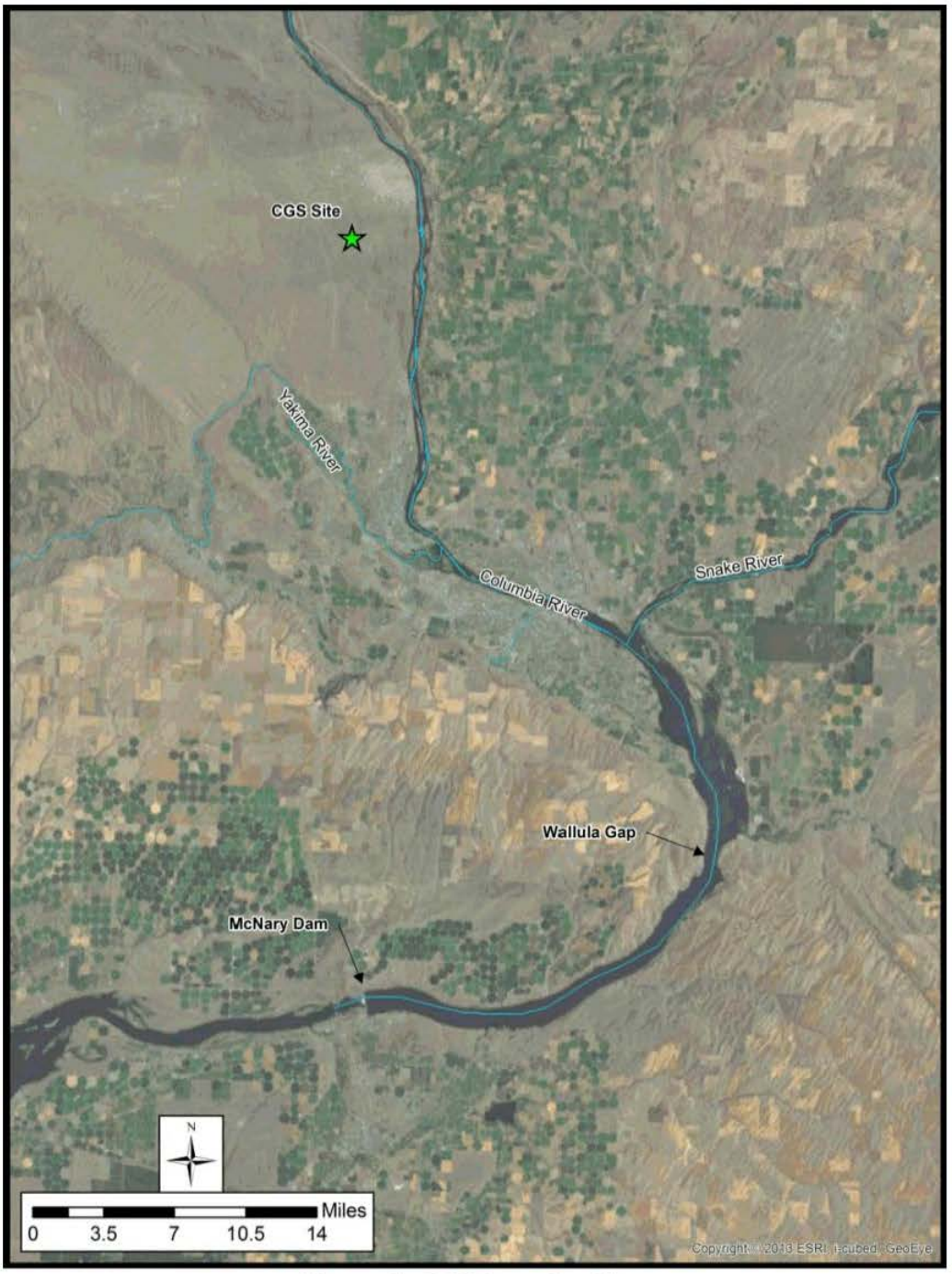


Figure 4. CGS Location Map

3.2.1 Basis of Inputs

3.2.1.1 PMP and Snowmelt (Reference 4 and Reference 5)

- Procedures in NUREG/CR-7046 (Reference 23), NUREG-0800 (Reference 22), and ANSI/ANS 2.8 (Reference 1) for determining PMP and LIP.
- Historic rainfall and other meteorological data collected by the National Weather Service (NWS) at numerous recording and cooperative climate stations and available from the National Climatic Data Center (NCDC), the National Oceanic and Atmospheric Administration (NOAA) Atlas 2 for the U.S. portions of the contributing drainage area, and the Rainfall Frequency Atlas for Canada for the Canadian portions of the contributing drainage area.
- Hydrologic Unit Code (HUC) sub-regions data are provided in GIS format from the U.S. Department of Agriculture's (USDA) online Geospatial Data Gateway (Reference 18).
- NWS HMR-57 (Reference 14), standard isohyetal patterns, storm orientation, percentage of 6-hour increment of PMP, and standard isohyetal geometry information.
- Precipitation frequency maps.
- Estimates of snow density for cool-season months were computed to calculate the Snow Water Equivalent (SWE) needed for use in the snowmelt analysis.
- Other snowmelt parameters were determined using the calculation procedure provided in Sections 15.2 and 15.3 of HMR-57 (Reference 14).
- Snowmelt rate (energy budget) equations and constants are based on USACE Engineering Manual EM 1110-2-1406 (Reference 15).

3.2.1.2 PMF Hydrologic and Hydraulic Analysis (Reference 5)

- PMP and associated snowmelt – 72-hour all-season and cool-season (with coincident snowmelt) PMPs for the watershed area upstream and downstream of CGS.
- United States Geologic Survey (USGS) stream and flow gage data.
- Hourly precipitation data at all NWS cooperative stations throughout and adjacent to the watershed in Washington, distributed by the NCDC.
- Soil types within the watershed, developed using Natural Resources Conservation Service (NRCS, formerly Soil Conservation Service) soils data.
- Manning's Roughness Coefficients (Manning's n-value), based on visual assessment of aerial photography and values recommended in published literature.
- Riverine and floodplain geometry from USGS topographic maps.

3.2.1.3 Local Drainage Basin PMF (Reference 9)

- Basin Topography – The 10-meter Digital Elevation Model (DEM) from the USGS National Elevation Dataset (Reference 19), and used as input to develop drainage basin delineation, basin parameters, elevation versus storage relationship, and PMF inundation mapping. The 10-meter DEM references the State Plane Washington South horizontal coordinate system, NAD 83 and the NAVD 88.
- Aerial Photography – National Agricultural Imagery Program (NAIP) aerial photography obtained from the USDA Geospatial Data Gateway was used for inundation mapping.
- Snowpack Data – The month in which snowpack is maximized in the Columbia watershed, as well as 100-year snowpack data developed from a statistical analysis of snow gage data, were obtained from Calculation No. CE-02-13-17 (Reference 4) and used as input to snowmelt computations.
- Snow Density – The historical snow depth and snow density data was obtained from the NOAA interactive snow website (Reference 4), which was used as an input to snowmelt computations.
- Land Cover – Land cover information was obtained from Calculation No. CE-02-13-17 (Reference 4).
- Four precipitation alternatives calculated in Calculation No. CE-02-13-27 (Reference 9) were distributed into five different temporal distributions for calculation of the local drainage basin PMF analysis using the HEC-HMS model. All simulations were executed at a 15-minute time step. The four precipitation alternatives input into the model were the general storm PMP, 100-year rainfall on probable maximum snowpack, the cool-season PMP on 100-year snowpack, and the local storm PMP.
- A hydrologic model of the local drainage basin was developed using the USACE's HEC-HMS computer software. Inputs to the hydrologic modeling include identification of sub-basins, PMP, and an elevation versus storage curve.
- The watershed draining to the CGS is a 40.5-square-mile depression, with the CGS centrally located (north-south) and near the lowest point in the basin. The basin was sub-divided into two sub-basins, with the boundary near CGS and the low point in the basin. Sub-basin delineation was conducted to denote the direction of flow towards CGS. The upstream sub-basin (or northern sub-basin) drains to the south towards CGS. The downstream sub-basin (or southern sub-basin) drains to the north towards CGS until water reaches the elevation of the outlet, which is located at the south end of the basin. If water reaches the elevation of the outlet, at 437.9 ft-NAVD 88, flow in the downstream sub-basin will reverse direction and will flow towards the south over the outlet, and then towards the Columbia River.
- An elevation versus storage relationship for input to the HEC-HMS model was developed using the 10-meter DEM data and ArcGIS. The elevation versus storage

curve shows that approximately 31,130 acre-ft of storage is available below the outlet elevation of 437.9 ft.

3.2.2 Computer Software Programs

3.2.2.1 PMP and Snowmelt Analysis

- ArcGIS Desktop 10.0

3.2.2.2 PMF and Local Drainage Basin PMF Analysis (Hydrologic and Hydraulic Analysis)

- ArcGIS Desktop 10.0
- HEC-HMS 3.5
- HEC-RAS 4.1.0
- HEC-GeoRAS 10.1
- VERTCON 2.1

3.2.3 Methodology

3.2.3.1 PMP Analysis

CGS is located in Benton County, Washington, approximately 3 miles west of the Columbia River. PMP was developed for the Columbia River watershed upstream of CGS. The Yakima River and the Snake River merge with the Columbia River downstream of CGS and upstream of McNary Dam. Backwater effects from McNary Dam on the Columbia River at CGS were also considered in the PMF analysis; therefore, PMP was also developed for the Yakima and Snake River watersheds.

Sub-basin parameters for PMP computations include drainage area, location of the sub-basin centroid, and elevation. Drainage area, centroid coordinates, and elevation were computed using ArcGIS and are discussed in Calculation No. CE-02-13-17 (Reference 4). Snowmelt computations require definition of the land cover, specifically the percent forested area, of each sub-basin. The percent forested area of each sub-basin was computed using land cover information obtained in a GIS format from the USDA Data Gateway and the Canadian Council on Geomatics Geo-Base website.

The Columbia River watershed above CGS, the Yakima River watershed, and the Snake River watershed are located within the Columbia River Region HUC 017. The HUC system is a consistent hierarchical dataset that contains drainage basin boundaries on four primary levels, including regions, sub-regions, accounting units, and cataloging units. The Columbia River watershed above CGS is divided into two HUC sub-regions, including the Upper Columbia (HUC 1702) and the Kootenai-Pend Oreille-Spokane (HUC 1701). An additional sub-region contributing to the Columbia watershed located entirely in Canada was identified as the Headwaters Columbia, but is not included in the HUC data set. The Headwaters Columbia sub-region boundary is defined by the Canadian National Hydro Network (NHN). The Snake River watershed is divided into three sub-regions including the Upper Snake (HUC 1704), the Middle Snake (HUC 1705), and the Lower Snake (HUC

1706). The Yakima River watershed is one sub-region (HUC 1703). All the rivers HUCs and watershed boundaries are shown in Figure 5.

HUC sub-region and cataloging unit boundaries were downloaded in GIS format from the USDA Geospatial Data Gateway (Reference 18) and imported for use into ArcGIS. HUC boundary data are in the Universal Transverse Mercator (UTM) Zone 11N coordinate system, and reference the horizontal North American Datum of 1983 (NAD 83). Boundaries for the Headwaters Columbia sub-region and sub-sub-drainage (the equivalent of a cataloging unit in the HUC system) located in Canada were obtained in ArcGIS format from the NHN as provided by the National Resources Canada Earth Sciences Sector Center for Topographic Information Geo-Base website.

Sub-basins in each sub-region were defined by first using and reviewing the HUC cataloging units and NHN sub-sub-drainages. Based upon drainage area or the location of major dams some cataloging units/sub-sub-drainages were either combined or subdivided. USGS topographic maps for the United States were accessed through the ArcGIS USA Topo Maps Map Server and used to sub-divide and refine HUC cataloging unit areas using ArcGIS. Subdivision of Canadian sub-sub-drainages was conducted using the National Topographic Database available from the Canadian data server GeoGratis.

A shape file containing all sub-basin boundaries was developed using ArcGIS for use in PMP calculations. The sub-basin GIS shape file references the UTM Zone 11N coordinate system and the horizontal NAD 83. All sub-basins with drainage areas less than 10,000 square miles were defined.

The Columbia River watershed is divided into sub-basins, and these sub-basins are grouped into three (3) sub-watersheds (Columbia, Yakima, and Snake River). The sub-watersheds were selected to have similar hydrometeorological characteristics, based on location of major tributaries to the Columbia River and location of USGS stream gages located on the main stem of the Columbia River. The sub-watersheds were divided into nine sub-regions.

The location of the CGS watershed is within the domain of the HMR-57 guidance. Generally, the all-season PMP is determined using generalized PMP estimates derived from HMR-57 guidance. However, the use of HMR-57 for developing PMP values is not applicable due to its watershed area size limitation of 10,000 square miles (maximum). The watersheds analyzed in the PMP and PMF analysis are identified in Figure 5 and are as follows:

- Columbia (98,000 sq. miles)
- Yakima (6,156 sq. miles)
- Snake River (108,000 sq. miles)

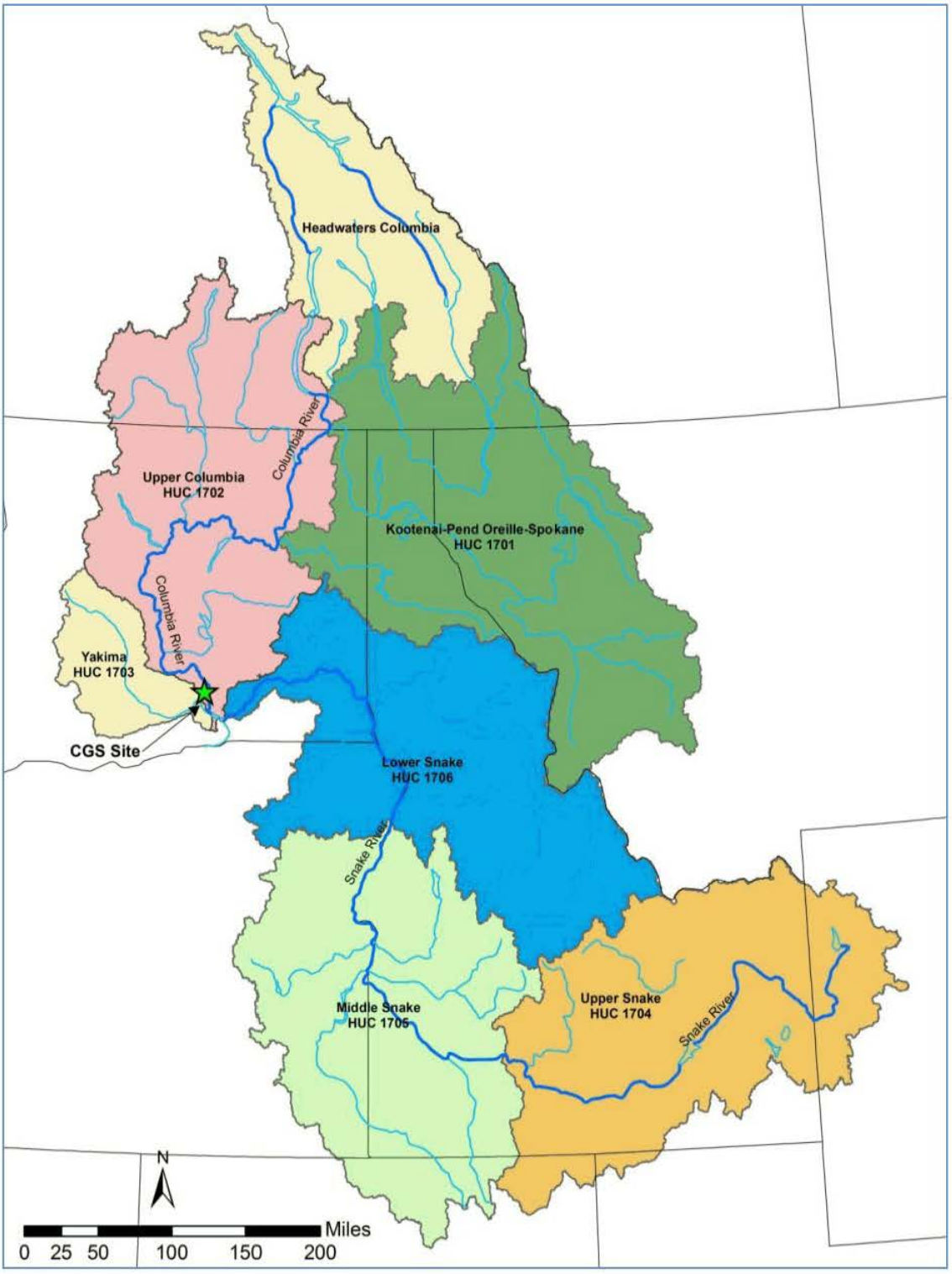


Figure 5. Columbia, Snake, and Yakima Sub-watershed Map

The method of analysis for determining the PMP included the following:

- Develop a method or strategy to calculate PMP based on HMR-57 guidance and limitation of 10,000 square miles.
- Calculate site-specific depth-area-duration data for various PMP storm centers.
- Calculate 6-hour incremental 72-hour PMP values.
- Calculate 100-year snowpack and snowmelt rates for cool-season PMP.

A 10,000-square-mile PMP storm was placed at different locations throughout each sub-watershed to determine which storm center produces the critical PMF. HEC-HMS hydrologic models were developed for each of the sub-regions identified in the above sub-watersheds. For sub-watersheds, with total drainage area of approximately 10,000 square miles, PMP storms were conservatively placed over the most downstream sub-watershed. If a sub-region has a total drainage area less than 10,000 square miles, the PMP was applied to the entire sub-regional area. The purpose was to calculate PMP values applicable to the entire contributing watershed of the Columbia River upstream of CGS. This included an all-season and cool-season (rain-on-snow) PMP analysis.

3.2.3.2 100-year Snowpack and Snowmelt

The 100-year snowpack for the watershed is calculated from historic snow depth data from 311 weather gages located throughout the watershed with periods of record greater than 25 years, calculated on a sub-watershed level. The 100-year snowpack is calculated by applying the Fisher-Tippett Type I (FT-I), or more commonly named Gumbel distribution, to the snow depth data at each gage. This is judged to be a conservative approach for the large watershed, as the actual recurrence interval for each gage to have a coincident 100-year snowpack for the entire area of the watershed is likely to be greater than one in one hundred years. The cool-season PMP is considered to be an April event. The Thiessen polygon method is used to calculate the 100-year snowpack for each sub-watershed by using an area-weighted average of the 100-year snow depth from the stations.

Snowmelt is included in two of the alternatives, and is determined using the USACE energy budget method. The energy budget method accounts for six (6) external sources of heat energy that contribute to snowmelt. The energy budget method yields a set of six equations; the selection of the appropriate equation is dependent upon a rain-on-snow or rain-free melt period and the percent forest cover within the drainage area. The available water in the snowpack is calculated based on Federal Energy Regulatory Commission (FERC) guidelines, which stipulate:

> *Water equivalence data are rarely recorded. If total snowpack depth is available, assume a 100-year snowpack for the month of the cool-season Probable Maximum Storm and a starting water equivalence of 30 percent.*

3.2.3.3 PMF Analysis (Reference 5)

Alternative 1 – All-Season PMP (General Storm PMP (72-hr Storm))

The all-season PMF resulted in a flow of 2,897,000 cfs on Columbia River downstream of Snake Confluence based on scenario No. 3 – Headwater Columbia.

An antecedent storm equivalent to 40 percent of the all-season PMP is applied to the HEC-HMS model with a 72-hour dry period between the antecedent storm and the PMP event.

The all-season PMP as described above is applied to the HEC-HMS model to determine flow hydrographs at key points in the model.

The all-season PMF is determined not to be the controlling PMF scenario and additional combined event analysis is not performed.

Alternative 2 – Probable Maximum Snowpack (24-hr storm) and 100-Year Cool-Season Rainfall

The 100-year rainfall would be significantly less rainfall than the PMP, resulting in the majority of the snowmelt occurring as rain-free. The rain-free snowmelt rates are typically close to or lower than the verified constant losses for each sub-watershed; therefore, most of the snowmelt would be lost through the constant losses and would not be available for runoff. For similar reasons, given the large size of the CGS watershed (approximately 212,156 square miles), calculating the probable maximum snowpack covering the entire watershed from the cool-season PMP precipitating as snow would result in comparable snow-water equivalent available for melting as the snow-water equivalent to be used in Alternative 3. Thus, this alternative is not the controlling flooding scenario at CGS.

Alternative 3 – 100-Year Snowpack and Cool-Season PMP

Two different time periods, including the months of November through February and the month of March, are utilized per the PMP data. Both data sets were evaluated by sub-basin. The time period with the highest cool season PMP on 100-year snowpack was selected for use in the HEC-HMS modeling. Five temporal distributions of the hourly incremental precipitation data were utilized in the hydrologic model to determine the critical PMF. The 100-year snowpack and a cool-season PMP resulted in the controlling PMF at CGS.

3.2.3.4 Hydrologic Model (HEC-HMS)

USACE HEC-HMS hydrologic software is used to convert rainfall to runoff. Rainfall is applied to each sub-watershed and transformed to runoff using the Clark unit hydrograph methodology. The sub-watershed parameters are calibrated and validated with historic extreme events for which sufficient stream flow and rainfall data are available. The HEC-HMS software is used to model and calibrate Clark unit hydrograph parameters and Muskingum-Cunge reach routing parameters. Baseflow is obtained from gage data and monthly average base flow is used in the HEC-HMS model.

Mean Monthly base flow for Columbia, Yakima, and Snake Rivers was determined using USGS gage data. Base flow was determined by calculating the average monthly discharge. The highest average mean monthly base flow was applied as a constant inflow within each HEC-HMS model.

The unit hydrographs for each sub-watershed are then modified to account for the effects of nonlinear basin response in accordance with NUREG/CR-7046 (Reference 23). The peak of each unit hydrograph is increased by one-fifth (20%) and the time-to-peak is reduced by one-third (33%). The remaining hydrograph ordinates are adjusted to preserve the runoff volume to a unit depth over the drainage area. In this case, hydrographs with nonlinear basin adjustment resulted in lower flows.

Therefore, to be conservative, hydrographs without nonlinear adjustments are used in the PMF analysis. The HEC-HMS modeling conservatively assumes no precipitation losses, no accounting for reservoir storage, and coincidental flooding occurring on adjacent rivers contributing to the Columbia River.

The PMF runoff hydrographs computed in HEC-HMS due to PMP were combined with a constant inflow of lesser flood events occurring in adjacent drainages to develop the most conservative WSEs on the Columbia River at CGS. Resulting WSEs due to the PMF on the Columbia River were then compared with the lowest elevation of critical facilities at CGS.

3.2.3.5 Hydraulic Model (HEC-RAS)

The HEC-RAS hydraulic model requires:

- Columbia River Topography – The DEM from the USGS National Elevation Dataset (Reference 19) and used to develop model geometry.
- Aerial Photography – Aerial photography obtained from the USDA Geospatial Data Gateway was used to get landmarks and verify Manning's n-values utilized in the model.
- Critical PMF peak discharge profiles and hydrographs were determined from HEC-HMS modeling results.
- Downstream boundary conditions – Normal depth of flow was utilized for the downstream boundary condition in HEC-RAS.

The controlling PMF scenario is determined to be from Alternative 3, a snow or cool-season PMP with a 100-year snowpack. The steady flow module within HEC-RAS model is used to perform steady state backwater computation for a sub-critical flow regime and to transform the resulting flow hydrographs from the controlling alternative into a WSE hydrograph under steady flow conditions.

The HEC-GeoRAS extension for ArcGIS was used to construct the hydraulic model geometry, based on including PMP events occurring in the Columbia watersheds of interest. The scope of the HEC-RAS model is based on landmark locations and features influencing the PMF analysis, such as dams and main rivers, which provide natural breakpoints.

The hydraulic model includes 156 miles of the Columbia River starting about 50 miles downstream of McNary Dam, extending upstream to Priest Rapids Dam. Therefore, the downstream boundary condition location is approximately 100 miles downstream of CGS, so that the model accounts for backwater effects due to the Wallula Gap and the McNary Dam. Figure 6 provides a map showing HEC- RAS cross section locations between CGS and McNary Dam.

Hydraulic modeling assumes that McNary Dam is inoperable and incapable of passing flow through its various outlets, spillways, and lock facilities. The HEC-RAS model assumes that all Columbia River flows overtop McNary's main channel concrete crest and overbank earthen embankments. In other words, the dam is conservatively assuming that that flow would overtop and not pass through at lower elevations. This overtopping elevation is also conservative since

the flows from the three river sources are directly combined. The model also assumes that McNary Dam remains intact and does not fail during the PMF event.

The PMF flow hydrographs are entered into the HEC-RAS model at the upstream end of the model and at the intermediate points representing other tributary rivers/streams using the "Lateral Flow" tool in HEC-RAS.

The HEC-RAS model is evaluated using steady-state flow for cool-season PMP (Alternative 3), which is the controlling scenario for CGS. Hydraulic modeling results were reviewed for PMF events occurring in each watershed individually. Results in each watershed were then compared to determine which event produces the highest WSEs near CGS. To demonstrate impacts at CGS due to the PMF event, WSEs at Cross Section 302788 (or OT#2) and Cross Section 281758 (or OT#1) were compared with the overtopping elevation of the west bank (HEC-RAS right bank). Cross section locations and identification of OT#1 and OT#2 are provided in Figure 6, and the Columbia River PMF water surface profile is shown in Figure 7. Cross section 302788 is adjacent to CGS at OT#2. WSEs at this cross section would have to be at or above an elevation of 441 ft-NAVD 88 (437.6 ft-NGVD 29) in order to spill water into CGS local basin, west of the Columbia River's west bank (HEC-RAS right bank). Cross section 281758 is located just downstream of CGS at OT#1. WSEs at this location would have to be at or higher than an elevation of 437.9 ft-NAVD 88 (434.5 ft-NGVD 29) in order to overtop the west bank (HEC-RAS right bank) and spill water in to CGS local basin.

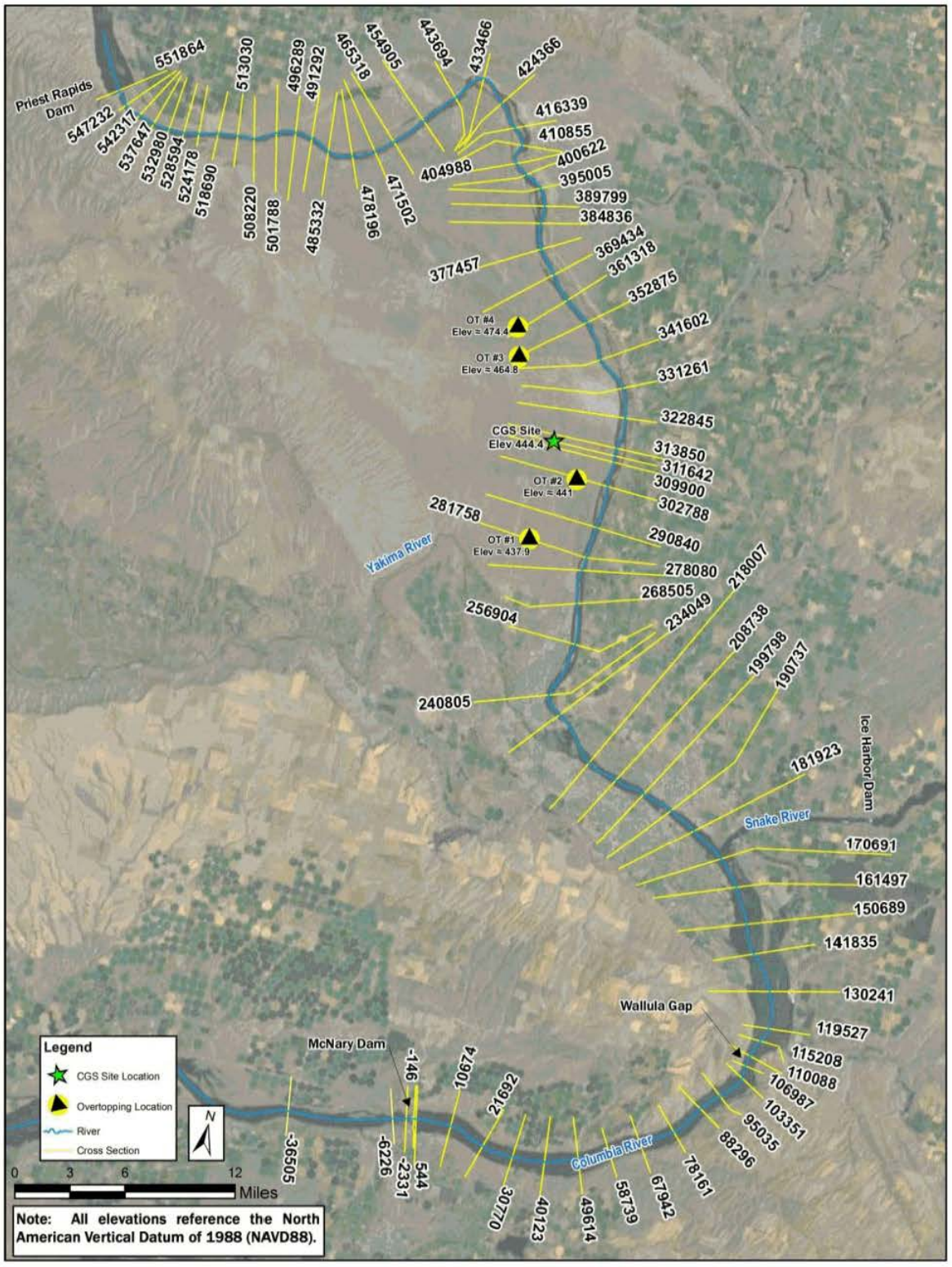


Figure 6. HEC-RAS Model Cross Section Locations

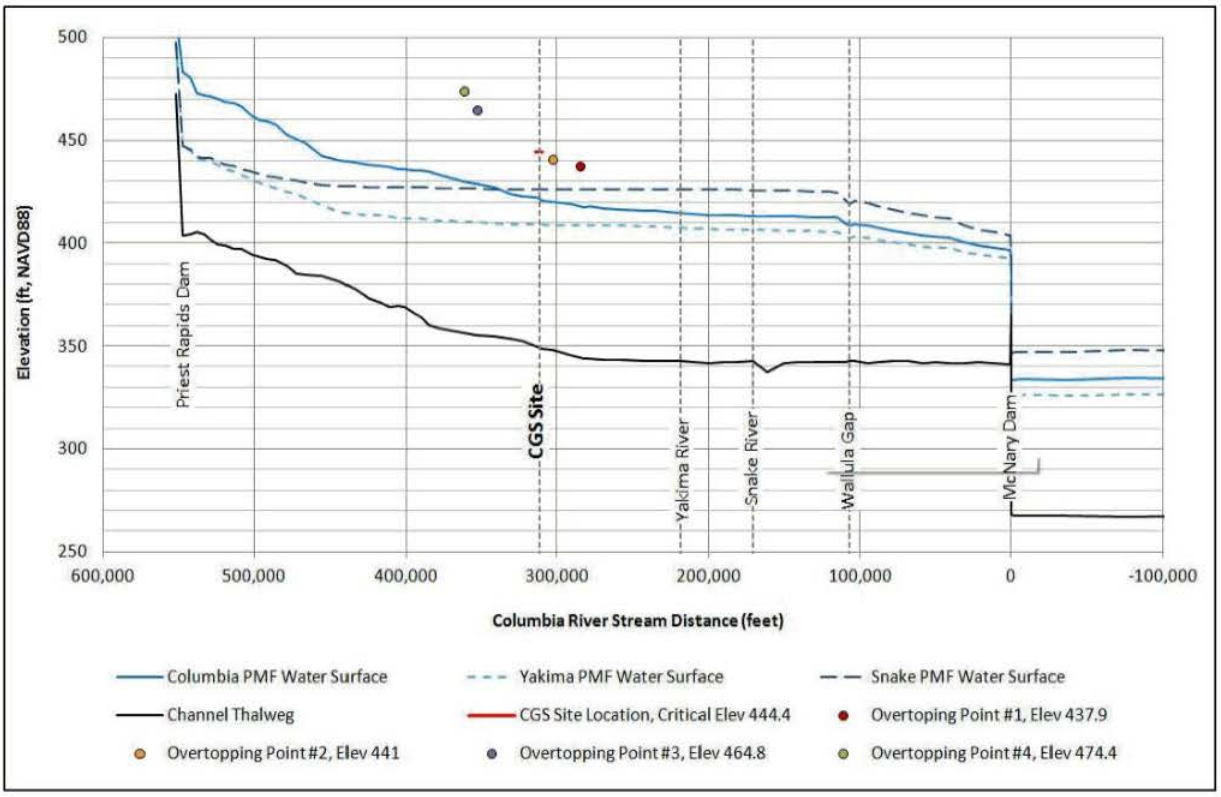


Figure 7. Columbia River PMF Water Surface Profiles

3.2.3.6 Local Drainage Basin PMF (Reference 9)

PMF due to PMP was analyzed for the 40.5-square-mile local drainage basin at CGS. Review of topography and aerial photographs indicate that the local basin is topographically closed and does not have a defined waterway, creek, or river running through it. Elevations in the basin range from a minimum of 409.3 ft-NAVD88 up to a maximum of 594 ft-NAVD88, with an outlet elevation of 437.9 ft-NAVD88. CGS is located in a depression that will fill up and store water before water spills out of the basin. Topographic data indicate that the basin is capable of storing approximately 31,000 acre-ft of water with a maximum depth of 28.6 ft before flow reaches the basin outlet and spills towards the Columbia River.

The PMP calculations for the local drainage basin were conducted using the current applicable guidance contained in HMR-57. Four alternative PMP storms were developed for input to hydrologic modeling per ANS guidance provided in NUREG CR-7046.

The stepwise procedure used to calculate the local storm PMP can be found in Section 15.4 of HMR-57. Local storm PMP estimates can be determined for durations between 15 minutes and 6 hours applicable to storm areas of 1 to 500 square miles.

The local storm PMP was computed for the CGS local basin and used as input to precipitation computations as part of the PMF analysis. Local storm PMP values were computed in 15-minute increments for a 6-hour storm duration. Five different temporal distributions of the 15-minute

incremental local storm PMP values were developed. Distributions include centering the highest 15-minute incremental value at the front, one-third, center, two-thirds, and end position within the storm duration, while placing the next highest values on alternating sides of the highest 15-minute value.

Hydrologic modeling of the local drainage basin was developed using the USACE's HEC-HMS computer software. The hydrologic model computes direct runoff of PMP storms, assuming no losses, no infiltration, no rainfall-to-runoff transformation, and no routing. PMP storm volumes were found to be less than the volume of storage available in the basin below the elevation of the basin outlet. This indicates that runoff due to PMP can be stored in the local basin without any outflow, eliminating the need to develop hydraulic computations. The HEC-HMS model used an elevation-versus-storage relationship developed from basin topography to compute PMF WSEs as a result of direct runoff due to PMP.

CGS local basin was split into two sub-basins, one upstream of CGS (or north of CGS with a drainage area of 26.7 sq. miles) and one downstream of CGS (or south of CGS with a drainage area of 13.8 sq. miles). The HEC-HMS model applies the PMP to both sub-basins. Both sub-basins are connected to a reservoir storage element in the HEC-HMS model. The reservoir storage element applies an elevation-versus-storage relationship developed from basin topography to compute PMF WSEs as a result of direct runoff due to PMP. The elevation-versus-storage relationship was calculated using 10-meter DEM data in ArcGIS. The CGS local sub-basins are identified in Figure 8.

Four PMP alternatives were run in the local drainage basin HEC-HMS model:

1. General storm PMP with antecedent storm equal to 40 percent of the PMP
2. 100-year rainfall on probable maximum snowpack
3. Cool season PMP on 100-year snowpack
4. Local storm PMP

The PMP was input into the HEC-HMS hydrologic model in the form of incremental hyetographs. All five temporal distributions of each alternative precipitation event were included in the HEC-HMS modeling, resulting in a total of 20 different precipitation simulations. Multiple HEC-HMS runs of alternative precipitation with varying temporal distributions were included to determine which event results in the most critical PMF WSE.

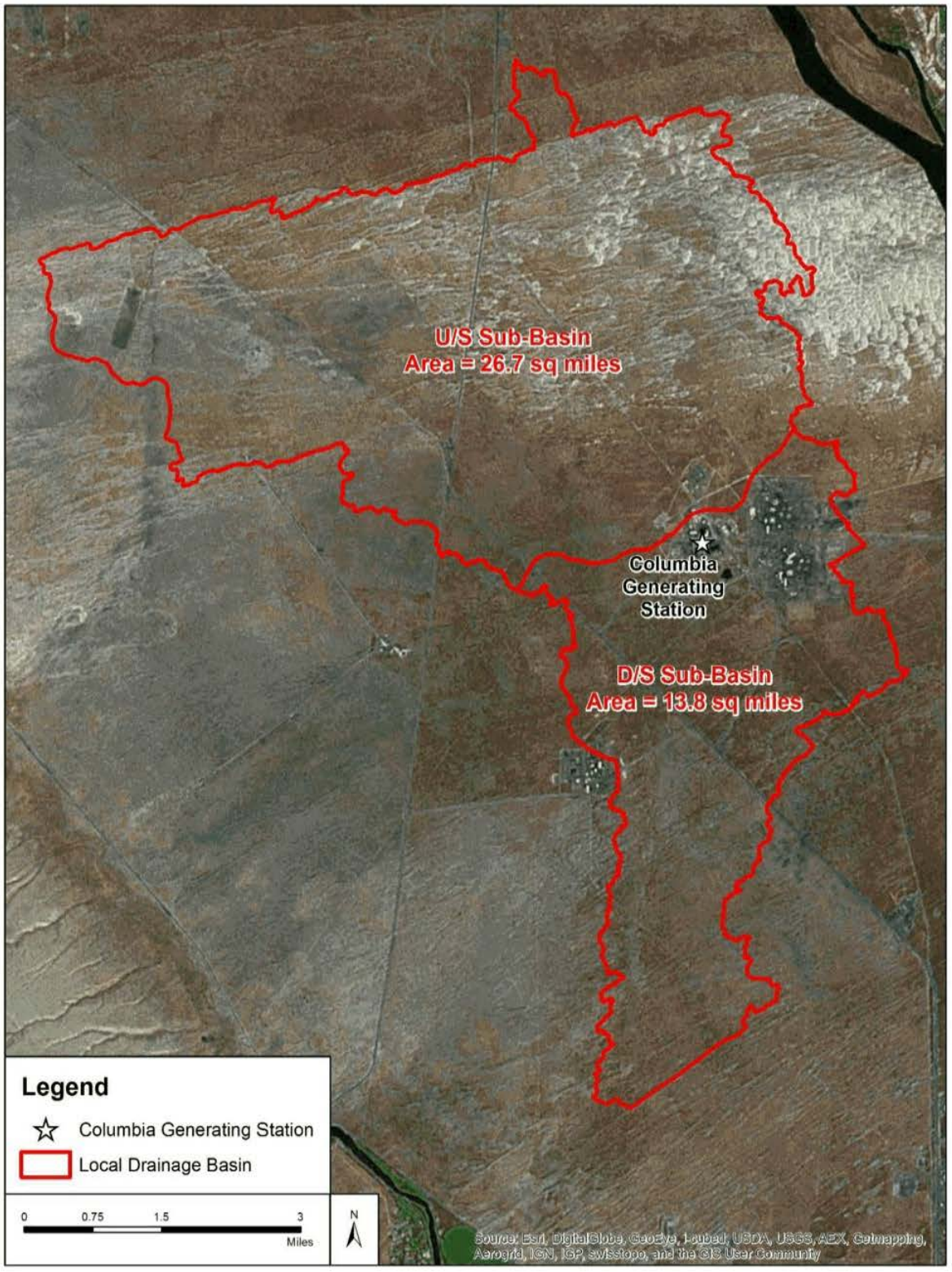


Figure 8. CGS Local Drainage Basin

3.2.4 Results

Calculation of the Columbia River PMF considered PMP events occurring in various locations throughout the Columbia, Yakima, and Snake River watersheds.

- The Columbia River PMF is a combination of runoff from PMP occurring in the Headwaters Columbia sub-region with a discharge of 2,560,000 cfs, a 500-year flood on the Yakima River with a discharge of 97,000 cfs, and a peak historic discharge of 312,000 cfs on the Snake River. The total discharge at the Wallula Gap constriction is 2,969,000 cfs. The PMF water surface elevations on the Columbia River near the CGS show a water surface elevation of 412.2 ft-NAVD 88 (408.8 ft-NGVD 29) at OT#1 (Cross Section 281758) and 415.2 ft-NAVD 88 (411.8 ft-NGVD 29) at OT#2 (Cross Section 302788 – adjacent to CGS). The water surface elevation at OT#1 is 25.7 ft below the overtopping elevation of 437.9 ft-NAVD 88 (434.5 ft-NGVD 29), which indicates that water will not spill into the local basin. Similarly, the water surface elevation at OT#2 is 25.8 ft below the overtopping elevation of 441 ft-NAVD 88 (437.6 ft-NGVD 29). Results indicate that there is a minimum of 25.7 ft of freeboard on the west bank (HEC-RAS right bank) of the Columbia River near CGS.
- The Yakima River PMF is a combination of runoff from PMP occurring in the Yakima watershed with a discharge of 1,056,000 cfs, the un-regulated standard project flood on the Columbia River of 740,000 cfs, and a peak historic discharge of 312,000 cfs on the Snake River. The total discharge at the Wallula Gap constriction is 2,108,000 cfs. PMF water surface elevations on the Columbia River near the CGS show a water surface elevation of 401.6 ft-NAVD 88 (398.2 ft-NGVD 29) at OT#1 (Cross Section 281758) and 402.2 ft-NAVD 88 (398.8 ft-NGVD 29) at OT#2 (Cross Section 302788 – adjacent to CGS). The water surface elevation at OT#1 is 36.3 ft below the overtopping elevation of 437.9 ft-NAVD 88 (434.5 ft-NGVD 29), which indicates that water will not spill into the local basin. Similarly, the water surface elevation at OT#2 is 38.8 ft below the overtopping elevation of 441 ft-NAVD 88 (437.6 ft-NGVD 29). Results indicate that there is a minimum of 36.3 ft of freeboard on the west bank (HEC-RAS right bank) of the Columbia River near CGS.
- The Snake River PMF is a combination of runoff from PMP occurring in the Upper Snake sub-region with a discharge of 2,860,000 cfs, the un-regulated standard project flood on the Columbia River of 740,000 cfs, and a 500-year flood on the Yakima River with a discharge of 97,000 cfs. The total discharge at the Wallula Gap constriction is 3,697,000 cfs. PMF water surface elevations on the Columbia River near the CGS show a water surface elevation of 415.2 ft NAVD 88 (411.8 ft-NGVD 29) at OT#1 (Cross Section 281758) and 415.4 ft-NAVD 88 (412.0 ft-NGVD 29) at OT#2 (Cross Section 302788 – adjacent to CGS). The water surface elevation at OT#1 is 22.7 ft below the overtopping elevation of 437.9 ft-NAVD 88 (434.5 ft-NGVD 29), which indicates that water will not spill into the local basin. Similarly, the water surface elevation at OT#2 is 25.6 ft below the overtopping elevation of 441 ft-NAVD 88 (437.6 ft-NGVD 29). Results indicate that there is a minimum of 22.7 ft of freeboard on the west bank (HEC-RAS right bank) of the Columbia River near CGS.

Comparison of results from each watershed PMF indicates that the Snake River PMF produces the highest water surface elevations on the Columbia River near CGS. Results are driven by the total discharge in the Columbia River at the Wallula Gap constriction, which creates backwater effects upstream past CGS. The time (based on the HEC-HMS simulation) for the discharge to peak on the Columbia River at the Wallula Gap constriction is 1 day, 16 hours, and 45 minutes.

Water surface elevations from HEC-RAS modeling of the Snake River PMF were used to conduct inundation mapping on the Columbia River near CGS. Inundation mapping of the critical PMF water surface profile (from the Snake River PMF) was conducted using the 10-meter DEM in ArcGIS. Inundation mapping of the water surface elevations on the Columbia River near CGS associated with the Snake River PMF is shown in Figure 9. The inundation mapping shows that the west bank (HEC-RAS right bank) of the Columbia River near CGS is not overtopped, and CGS local basin remains dry during the PMF on the Columbia River. The flooding results for the PMF do not encroach on CGS. Therefore, no associated effects are determined.

Analysis of the PMF due to PMP occurring in CGS local basin is addressed in a separate calculation entitled Local Drainage Basin PMF Analysis for CGS, Calculation No. CE-02-13-27 (Reference 9). The critical PMF peak water surface elevation of 435.4 ft-NAVD 88 (432.0 ft-NGVD 29) was compared with CGS critical facilities elevation of 444.4 ft-NAVD 88 (441 ft-NGVD 29), indicating that there is approximately 9.0 ft of freeboard during the critical PMF event. Inundation mapping of the critical PMF WSE of 435.4 ft-NAVD 88 associated with Alternative 1 precipitation was conducted using the 10-meter DEM and ArcGIS. Critical PMF inundation mapping is provided in Figure 10. From the onset of the PMP storm, the maximum water surface elevation is reached in 69 hours. The flooding results for local basin do not encroach on CGS. Therefore, no associated effects are determined.

The groundwater table elevation at CGS is approximately 380 ft-MSL. The groundwater design basis is 420 ft-MSL. The peak effects of the PMP event occur over a relatively short duration. Therefore, changes to the groundwater design basis level are not expected. However, all piping and electric conduit penetrations that are below grade are waterproof sealed.

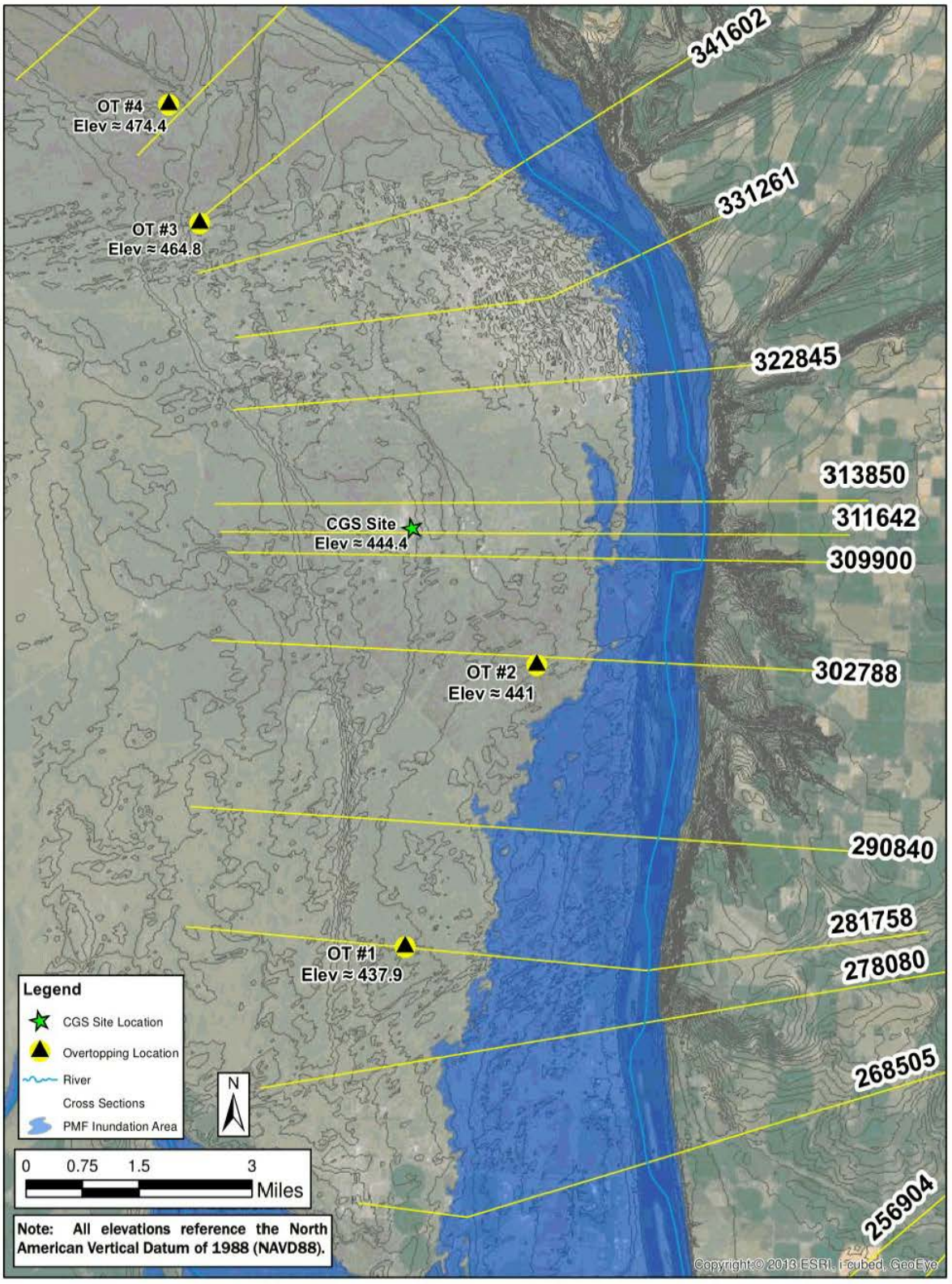


Figure 9. Columbia River near CGS Inundation Map – Snake River PMF

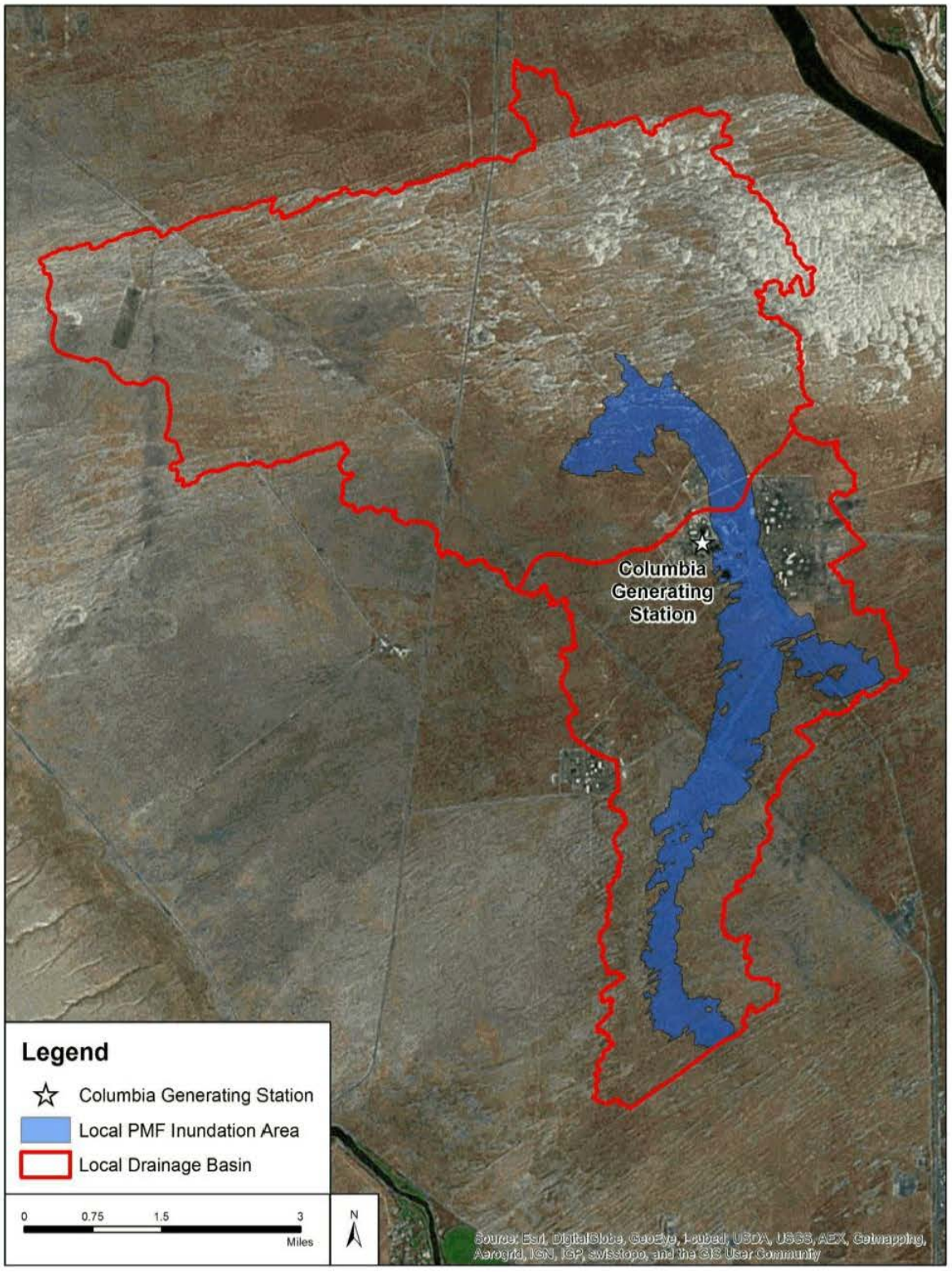


Figure 10. Local Drainage Basin PMF Inundation Map

3.3 Dam Breaches and Failures (Reference 25)

The dam failure analysis for the Columbia River watershed was performed by the USACE under For Official Use Only (FOUO) restrictions (Reference 25). Therefore, details of the analysis are not provided.

Onsite impoundments are located southeast and northeast of the power block. The power block is located at a higher elevation than the ponds. The topography slopes away from the power block to the east. Therefore, potential failure of the onsite impoundments would release impounded water away from the power block into the adjacent depression of the local drainage basin discussed in Section 3.2.

3.3.1 Results

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

3.4 Storm Surge

CGS has an inland location and does not connect directly with any of the water bodies considered for meteorological events associated with storm surge. Therefore, flooding due to a storm surge is not applicable at CGS.

3.5 Seiche

CGS has an inland location and does not connect directly with any of the water bodies considered for meteorological events associated with seiche. Therefore, flooding due to a seiche is not applicable at CGS.

3.6 Tsunami

CGS has an inland location and does not connect directly with any of the water bodies considered for Tsunami events. Therefore, flooding due to a tsunami is not applicable at CGS.

3.7 Ice Induced Flooding (Reference 6)

As identified by NUREG/CR-7046 (Reference 23), ice jams and ice dams can form in rivers and streams adjacent to a site and may lead to flooding by two mechanisms:

- *Collapse of an ice jam or a dam upstream of the site can result in a dam breach-like flood wave that may propagate to the site* and,
- *An ice jam or a dam downstream of a site may impound water upstream of itself, thus causing a flood via backwater effects.*

3.7.1 Basis of Inputs

Calculation inputs include the following:

- USACE Ice Jam Database.
- Historical ice jam information was obtained from the USACE Ice Jam Database.

3.7.2 Computer Software Programs

- Arc GIS 10.1
- VERTCON 2.1

3.7.3 Methodology

Ice-induced flooding is assessed by reviewing the USACE national ice jam database to determine the most severe historical events that have occurred on the Columbia River, in the vicinity of CGS. As discussed in Calculation CE-02-13-20 (Reference 6), the period of historical ice jam record is available from 1919 through January 2011. The ice jam height was calculated as a recorded gage height due to ice jam minus the base flow in the stream. The maximum ice jam height was selected. The peak elevation of recorded ice jam flooding minus the normal surface water elevation at that location is assumed to represent the full height of the ice jam at the recorded location. The historic ice-induced flood is calculated to be the result from the February 2, 2009, ice jam occurring in Twisp, Washington, recorded on the Methow River in Twisp.

The maximum ice jam is determined by selecting the historic event that produced the maximum flood stage relative to the normal WSE at that location. Regardless of specific conditions that produced the historic flood stage at a specific location, the full height is conservatively assumed to represent the ice jam.

The maximum ice jam height occurring on the Methow River near Twisp is transposed to CGS. This approach is conservative as it maximizes the ice jam in the immediate vicinity of CGS. An upstream ice jam can cause flooding by impounding water and then collapsing. The nearest probable location for an upstream ice jam is the Vernita Bridge on State Highway 24, approximately 36.7 miles upstream of CGS. A downstream ice jam can cause flooding by impounding water and then creating backwater effects. The nearest applicable location for a downstream ice jam is the bridge on Interstate 182 in Richland, Washington, approximately 16.1 miles downstream of CGS. The locations of the upstream and downstream bridges are shown in Figure 11.

The results of the ice jam effects are compared to the finished floor elevations of CGS and to the resulting flood elevations from the PMF and dam failure analyses to demonstrate that ice jam flooding is bounded by these flooding mechanisms.

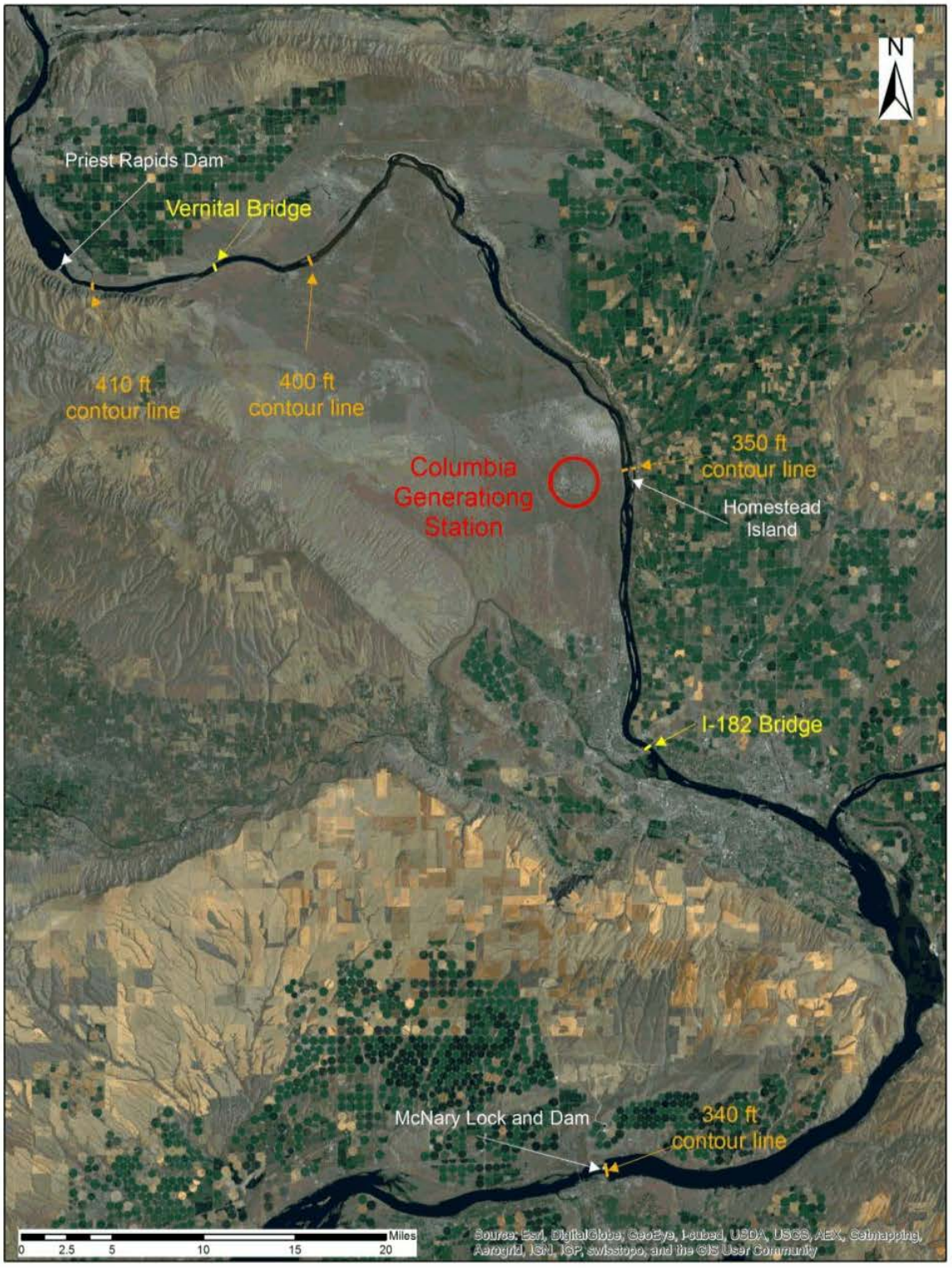


Figure 11. Upstream and Downstream Bridge Locations for Ice Jam

3.7.4 Results

The maximum WSE at CGS resulting from the upstream ice jam breaching was calculated to be 419.9 ft-NAVD 88 (416.5 ft-NGVD 29), which is more than 24 ft below CGS grade of 441 ft-MSL.

The maximum WSE at CGS resulting from backwater from a downstream ice jam is 362.8 ft-NAVD 88 (359.4 ft-NGVD 29), which is more than 81 ft below CGS grade of 441 ft-MSL. The upstream and downstream ice jam locations are identified in Figure 12.

The ice jam elevation near CGS was determined to be 365.5 ft-NAVD 88 (362.1 ft-NGVD 29), which is more than 78 ft below CGS grade of 441 ft-MSL.

The resulting flood elevation from the PMF analysis (Reference 5) is 415.4 ft-NAVD 88 (412 ft-NGVD 29). Note the upstream ice jam breaching result is bounded by the dam failure analysis performed by the USACE under For Official Use Only (FOUO) restrictions. By comparison, the ice jam effects are bounded by the PMF and Dam Failure flooding water surface elevations and do not produce the flooding conditions for CGS.

3.8 Channel Migration or Diversion (Reference 6)

As identified by NUREG/CR-7046 (Reference 23), Section 3.8, there are no well-established predictive models for channel diversions. Therefore, it is not possible to postulate a probable maximum channel diversion event. Instead, historical records and hydro-geomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards CGS.

Evidence of channel migration was examined using historical maps obtained from the USGS. Topographic map of 2011 was used as the basis for comparison as it reflects current conditions (at the time of development of Calculation CE-02-13-20). Historical topographic maps from the years of 1917, 1951, 1978, and 1992 are downloaded from the USGS store and compared to the current topographic map of 2011 (Reference 6). The historical map of 1917, which is assumed to be based on a hand drawing, shows a small discrepancy in the boundaries of the river banks and islands with a maximum difference of 0.18 mile, when compared to the topographic map of 2011. The other maps represent almost the same river bank and island boundaries when compared to the current map. Therefore, it is concluded that there is no evidence of channel migration at CGS and its vicinity.

3.9 Combined Effect Flood (Including Wind Generated Waves, Reference 7)

The criteria for combined events are provided in NUREG/CR-7046 (Reference 23). The combined events incorporate the flood causal mechanisms previously discussed for precipitation events and hydrologic or seismic dam failures. Each combined event also incorporates waves induced by 2-year wind speed applied along the critical direction. Based on the resulting water surface elevations previously discussed, basin topography is used to develop the fetch length. The 2-year wind speed is determined using the ANSI/ANS-2.8-1992 guidance (Reference 1).

3.9.1 Basis of Inputs

- Aerial Photography - National Agricultural Imagery Program (NAIP) aerial photography obtained from the U.S. Department of Agriculture (USDA) Geospatial Data Gateway (Reference 18) was used for inundation mapping (Reference 7).
- Basin Topography - 10-meter DEM used in the determination of Columbia River PMF WSE was used (Reference 5). The 10-meter DEM references the State Plane Washington South horizontal coordinate system and NAD 83 and NAVD 88.
- CGS Structures - CGS structures such as buildings, VBS, and other related facilities were obtained from the effects of Local Intense Precipitation Calculation CE-02-13-22 (Reference 8).
- The Local Drainage Basin PMF was determined in calculation CE-02-13-27, Local Drainage Basin PMF Analysis for CGS (Reference 9).
- The controlling dam failure WSE for Columbia River was determined by the USACE under FOUO restrictions (Reference 25).

3.9.2 Computer Software Programs

ArcGIS 10.1

3.9.3 Methodology

Coincident Wind-Wave Activity

The simplified method for wave forecasting as outlined in the USACE Coastal Engineering Manual (Reference 17) is used to determine the inputs (significant wave height, wave period, wind speed, and wavelength) for calculating the wave runup at CGS (Reference 7).

The wave setup is the elevation of the water surface due to wave action, in particular, wave breaking. Wind setup is the effect of the horizontal stress of the wind on the water, driving it in the direction of the wind. Based on the maximum water surface elevation from the flooding analysis, the longest critical straight line fetch is determined. The 2-year wind speed is applied to the straight line fetch to develop the wind-wave characteristics.

In accordance with NUREG/CR-7046 (Reference 23), different external flooding mechanisms are combined to calculate the maximum WSE including coincident wind-wave activity at CGS. The results of the combined events analysis (Reference 7) yielded that floods caused by precipitation events based on the following combination of mechanism resulted the highest water level:

- (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)
- Local Drainage Basin PMF coincident with wave induced by 2-year wind speed applied along the critical fetch direction. This corresponds to the local drainage Basin PMF WSE of 435.4 ft-NAVD 88 (Reference 7).

3.9.4 Results

The following summarizes the results of the combined events analysis (Reference 7) for CGS:

Wind-Wave Activity

- The maximum combined events WSE for the CGS is determined by adding the wind setup and wind-wave run-up, including wave setup, to the local drainage PMF WSE and Dam Failure Maximum WSE (PMF WSE + wind setup + runup) at CGS. The maximum dam failure WSE including coincident wind-wave activity is [redacted] ft-NGVD 29. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)
- The maximum combined events WSE for the local drainage basin PMF is determined by adding the wind setup and wind-wave run-up, including wave setup, calculated for the local drainage basin PMF to the resulting WSE due to local drainage basin PMF. The maximum WSE of the local drainage basin PMF including coincident wind-wave activity is 433.3 ft-NGVD 29.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Therefore, the controlling maximum WSE is the dam failure scenario coincident with wind-wave activity is [redacted] ft-NGVD 29. When compared to CGS elevation of 441 ft-MSL, the maximum WSE of the combined wind-wave activity, which is [redacted] ft-NGVD 29 is [redacted] lower than the ground elevation at CGS. Therefore, CGS is safe against flooding from the combined effects maximum WSE. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

4. CONCLUSION

As discussed in Section 3.0, several flood causing mechanisms were considered following the guidance provided in NUREG/CR-7046 (Reference 23). In summary, the reevaluated flood elevation is either below the critical elevation of 441 ft-NGVD 29 (444.4 ft-NAVD 88) or does not impact any safety-related equipment.

The maximum WSE due to the LIP event at CGS varies between 435.14 ft-NGVD 29 and 443.27 ft-NGVD 29. Areas where maximum WSE results exceed 441 ft-NGVD 29 are either not adjacent to safety-related SSCs, or based on specific configuration at the survey point location, the results are do not compromise any safety-related SSCs.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

The maximum WSE due to riverine flooding is [redacted] ft-NGVD 29, which includes dam failure and associated wind-wave activity. The reevaluated WSE, including coincident wind-wave activity is below the critical facility elevation of 441ft-MSL.

5. COMPARISON WITH CURRENT DESIGN BASIS

The reevaluated maximum water surface elevations due to the Columbia River PMF, the Adjacent Drainage Basin PMF, dam failure, and the combined effect flood for dam failure with coincident wind-wave activity exceed the design basis. For each flood-causing mechanism that exceeds the design basis, the maximum water surface elevation is still below the 441 ft-MSL critical elevation at CGS.

The reevaluated maximum water surface elevations due to ice-induced flooding are bounded by other flood-causing mechanisms and are below the 441 ft-MSL critical elevation at CGS. The reevaluated maximum water surface elevations due to the LIP vary between 435.14 ft-NGVD 29 and 443.27 ft-NGVD 29 at the designated protected area survey points. The reevaluated maximum water surface elevations at six of the survey points slightly exceed the 441 ft-MSL critical elevation at CGS. However, the six survey points are either not adjacent to safety-related SSCs, or based on specific configuration at the survey point location, the results are inconsequential and do not compromise safety-related SSCs, as discussed in Section 3.1.4.

During the flooding walkdowns conducted for Recommendation 2.3, the following was reported on November 12, 2012 in letter GO2-12-164 (Reference 3):

1. The safety-related SSCs are built within the Protected Area and at the Standby Service Water Pumphouses at the finished floor slab of el. 441 ft-MSL. Below-grade areas in the Reactor Building are at slab el. 422 ft-3 inches MSL, and below-grade areas in the Pumphouses are at 431 ft-MSL. Penetrations in the Reactor Building's below-grade concrete walls were visually observed as sealed, and design-basis groundwater (420 ft-MSL) is below this level so no hydrostatic loading is applied to these seals. Penetrations in the Standby Service Water Pumphouses exposed to the design basis flood elevation of 433.3 ft-MSL were noted as sealed. Penetrations located below grade el. 441 ft-MSL at exterior walls are above the groundwater el. of 420 ft-MSL. In the safety-related structures, exterior concrete walls showed no cracking equal or greater than 0.04 inches that challenged the ability to withstand water infiltration.

2. No flood protection features were excluded from the walkdowns. No degraded, non-conforming, or unanalyzed conditions credited for flood protection were identified by the walkdown visual inspection. There were no flooding hazard findings or actions that required entry into the Corrective Action Program (CAP). There are no exterior incorporated or temporary flood barriers or advance preparations of emergency flood-related equipment credited in the CLB for CGS.

3. CGS Site topography was noted to maintain overall natural drainage profiles of the original CLB. No washouts or significant areas of erosion were found. Concrete, asphalt, and gravel paved areas are well maintained and no degraded areas were observed. The perimeter concrete security barrier is not a hazard or obstruction relative to flooding.

4. The walkdown visual inspections of CGS site modifications and building flood protection measures required per the NRC-endorsed flooding walkdown guidelines were satisfactorily accomplished. There was no restricted access or inaccessible features concerning flood-related inspections.

5. Energy Northwest chose to visually inspect accessible below grade penetrations, walls, and floors in the Reactor Building and the Standby Service Water Pumphouses, which house the SSCs important to safety. Below grade walls, floors, and penetration seals that were visually inspected were found to be in good condition, with two exceptions. There were two conduits in one of the Standby Service Water Pumphouses that did not have a visible seal viewed from the building side; the other side was in a buried duct bank and was not accessible. The interiors of the conduits were clean and there was no dirt, corrosion, debris, or evidence of water or insect intrusion from the outside. These exceptions are not deficiencies as defined in NEI 12- 07 (Reference 12) because the conduit seals are not credited to perform an intended flood protection function. These two conduits are located above the design-basis groundwater elevation.

In summary, the reevaluated maximum water surface elevations validate the current flood mitigation strategy of the current license basis, which states that CGS can be maintained in a safe condition for water levels up to 441 ft-MSL. Therefore, no interim actions are identified. No additional actions are planned to address flooding hazards as the reevaluated maximum water surface elevations do not impact any safety-related structures.

The comparisons of existing and reevaluated flood hazards are shown in Table 3.

Table 3. Summary of Current Design Basis and Reevaluated Flood Hazard Elevations

Flood-Causing Mechanism	Current Design Basis Flooding Hazard Elevations	Comparison	Flood Hazard Reevaluation Elevation
LIP	Description in FSAR is very general and does not provide a WSE due to LIP for CGS.	The resulting WSE from this flood-causing mechanism is not described in the FSAR.	The maximum WSE at the protected area survey points varies between 435.14 ft-NGVD 29 and 443.27 ft-NGVD 29. The corresponding calculated maximum water depths vary between 0.03 ft and 0.79 ft. Areas where maximum WSE results exceeding 441 ft-NGVD 29 are either not adjacent to safety-related SSCs, or based on specific configuration at the survey point location, the results are inconsequential and do not compromise safety-related SSCs.
Flooding in Streams and Rivers (PMF)	Columbia River PMF is 390 ft-MSL (at the Seismic Category II makeup water structure). Adjacent Drainage Basin PMF is 431.1 ft.-MSL.	Not Bounded, but available physical margin exists. Not Bounded, but available physical margin exists.	The Columbia River reevaluated critical PMF WSE of 412.0 ft-NGVD 29 is below the critical facility elevation of 441 ft-MSL. The local drainage basin reevaluated critical PMF WSE of 432.0 ft-NGVD 29 is below the critical facility elevation of 441 ft-MSL.
Dam Breaches and Failures	The FSAR considers the effect of upstream dam failure (from breaching Grand Coulee Dam) in lieu of a seismically induced flood. Dam Failure coincident with the Standard Project Flood is 422 ft-MSL.	Not Bounded, but available physical margin exists.	The controlling dam failure WSE at CGS is [redacted] ft-NGVD 29. The dam failure reevaluated WSE is below the critical facility elevation of 441 ft-MSL. b)(3)(16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)
Storm Surge	Not specifically addressed in the FSAR.	Bounded*	Screened Out.
Seiche	Not specifically addressed in the FSAR.	Bounded*	Screened Out.
Tsunami	Not specifically addressed in the FSAR.	Bounded*	Screened Out.

Flood-Causing Mechanism	Current Design Basis Flooding Hazard Elevations	Comparison	Flood Hazard Reevaluation Elevation
Ice-Induced Flooding	As indicated in the FSAR, ice flooding is considered insignificant at CGS and in any event, ice flooding will not affect the capability to shut down the reactor in a safe and orderly manner.	The resulting WSE from this flood-causing mechanism is not described in the FSAR.	Upstream bridge ice jam elevation is 416.5 ft-NGVD 29 Downstream bridge ice jam elevation is 359.4 ft-NGVD 29 Ice jam elevation near CGS is 362.1 ft-NGVD 29. Ice Jam elevations are bounded by PMF and Dam Failure mechanisms and are below the critical facility elevation of 441 ft-MSL.
Channel Migration or Diversion	As indicated in the FSAR, the riverbed is well defined near CGS, and therefore, unlikely to be diverted from its present location.	Bounded.	No evidence of channel migration near CGS.
Combined Effect Flood (including wind generated waves)	Local Drainage PMF + Coincident Wind-Wave = 433.3 ft-MSL Dam Failure + Coincident Wind-Wave = 424 ft-MSL	Bounded. Not Bounded, but available physical margin exists.	Local Drainage PMF + Coincident Wind-Wave = 433.3 ft-NGVD 29. Dam Failure + Coincident Wind-Wave = [redacted] ft-NGVD 29. (b)(3):16 U.S.C. § 824o-1(d),(b)(7)(F) The reevaluated WSE, including coincident wind-wave activity is below the critical facility elevation of 441 ft-MSL.

* Since these mechanisms were screened out as part of flood hazard revaluation, they were also considered bounded by the current design basis.

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