The elevation of the facility is 420 ft msl and the PMP flood level is 431.1 ft msl (Section 2.4.3.5). See Section 10.4.10 for more discussion.

Roofs of buildings are designed to take, with adequate drainage, any instantaneous or local intense precipitation. Discharge from roof drains is carried by means of a storm sewer system to a manhole located southeast of the reactor building. From that point a pipeline with a northeast alignment transfers the discharge to a low point of disposal about 1500 ft away from the plant site.

The roofs of safety-related buildings (diesel generator building, radwaste/control building, standby service water pump house) are concrete beam and slab construction except the high roof of the reactor building, which is metal deck on steel framing. The minimum roof slope for all structures is 1/8 in. per ft for adequate drainage and the roof areas are encompassed by curbs or parapet walls up to 3 ft 6 in. high. Roof plans, including details of roof drains and overflow scuppers, are provided in Figure 2.4-6. Assuming that the roof drains are completely blocked during the PMP 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 structurally can carry the entire PMP accumulations.

2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

Analyses for probable maximum flood (PMF) and SPF on the Columbia River (Reference 2.4-2) are consistent with the requirements of Regulatory Guide 1.59, Revision 2. The SPF for the Mid-Columbia Reach of the highly developed and regulated Columbia River is defined as 570,000 ft^3/sec (Reference 2.4-4). The unregulated SPF for the same reach is 740,000 ft^3/sec. The unregulated PMF at the site is estimated to be 1,600,000 ft^3/sec (References 2.4-2, 2.4-4, and 2.4-5).

Adjustment of the flood profiles for the Hanford region reported in Reference 2.4-4, results in a regulated PMF of 1,440,000 ft^3/sec and a water level of 390 ft at the Seismic Category II makeup water structure. This structure is not designed to function throughout the PMF but is designed for the SPF (unregulated) of 740,000 ft^3/sec.

Although assumed to exist for the purpose of flood hydrograph calculations, Ben Franklin Dam is not a federally authorized project. As originally planned it would have been a low head dam with only a negligible effect on extreme flood flows (Reference 2.4-6).

The design basis flood for the CGS site area results from the PMP event on the adjacent drainage basin and not from flooding of the Columbia River.

2.4.3.1 Probable Maximum Precipitation

The PMP event which was presented in the CGS PSAR was subsequently reevaluated in the preparation of the PSAR for WPPSS Nuclear Project No. 1 (Docket 50-460). The analysis presented here is consistent with the latter document.

Precipitation in the vicinity of the site has been classified by the U.S. Weather Bureau, Reference 2.4-3, as convergence precipitation, orographic precipitation, and thunderstorm precipitation. The methodology for predicting the total amount of precipitation from each of these events, as given in Reference 2.4-3, requires the adding together of the convergence PMP and the orographic PMP to obtain a single precipitation for a general storm. A separate analysis is then required for thunderstorms. Thunderstorms in the vicinity of the site can be locally very intense for short periods of time and hence, have the potential for causing serious flooding. The PMP for both a general storm and a thunderstorm were analyzed as given in Chapters 6 and 5, respectively, of Reference 2.4-3 for a 38.5 mile^2 basin at the site. This basin is shown in Figure 2.4-8 and is described in Section 2.4.3.3. The calculated general storm PMP results in a 24-hr and 48-hr precipitation of 7.9 in. and 10.1 in., respectively. A thunderstorm PMP yields 9.2 in. in a 5-hr period. Therefore, the thunderstorm is considerably more severe. The thunderstorm PMP hydrograph is

Time (hr)	Rain (in.)
1	0.6
2	1.6
3	5.2
4	0.9
5	0.5
6	0.4
Total	9.2

2.4.3.2 Precipitation Losses

Infiltration losses have been estimated in the vicinity of the sites as 1.5 in./hr (Reference 2.4-7). However, for the analysis below, an average antecedent moisture condition (Condition II as defined in Reference 2.4-8) was assumed. As explained in the following section, the 60-minute retention loss rate is 0.15 in./hr.

2.4.3.3 Runoff and Stream Course Models

The drainage basin common to the reactor building and spray ponds is shown in Figure 2.4-8. The entire area drains to a broad channel that extends in a north-south direction for about 7 miles, and ranges from about 2000 ft to over a mile wide. All plant structures are located on high ground to the west of the channel. At a point about 2.8 miles south of the reactor site, the four-lane Department of Energy (DOE) highway crosses the drainage basin. The area above this section is 33.2 miles^2.

To evaluate the effect of the PMP event on the plant area, the peak discharge at the highway crossing, 2.8 miles downstream of the plant, was calculated using the U.S. Bureau of Reclamation procedure for computing design floods on ungauged basins from thunderstorm rainfall in the western U.S. (Reference 2.4-8). Important assumptions used in the triangular hydrograph procedure of Reference 2.4-8 are

a. Hydrologic soil group B,

b. Land use and treatment class - poor pasture or range,

c. Thunderstorm cover-index is brush-sage-grass combination with 50% or less cover density, and

d. Thunderstorm minimum 15-minute retention loss rate of 0.06 in./15 minutes and 60-minute retention loss rate of 0.15 in./hr.

Additionally, no credit was taken in the hydrograph analysis for potential storage in the stream channel or upstream sub-basins.

The time of concentration, T_c, for the watershed above the highway crossing was computed to be 7.5 hr. The PMF hydrograph is shown in Figure 2.4-7 for the 33.2 mile^2 drainage basin. A peak discharge of 21,400 ft^3/sec was determined.

Based on this PMF, an upstream water surface profile was determined using the Corps of Engineers HEC Standard-Step Procedure (Reference 2.4-9). A total of eleven cross sections were used (seven downstream, one at the plant, and three upstream as shown in Figure 2.4-3). Details of the channel cross sections are shown in Figure 2.4-9. The Manning roughness coefficient was conservatively taken as n=0.035 in the main channel sections, and n=0.05 in the overbank areas.

Using the computational procedure of Reference 2.4-9, it was determined that the channel restrictions at cross sections 5 and 7 (Figure 2.4-3) do not control the flow. The stillwater elevation at the plant site (cross section 8) was determined to be 431.1 ft msl. The water surface profile is shown in Figure 2.4-10.

2.4.3.4 Probable Maximum Flood Flow

The PMF runoff hydrograph produced by the PMP at cross section 1 (Figure 2.4-3) is shown in Figure 2.4-7. The peak discharge at this location is 21,400 ft^3/sec.

2.4.3.5 Water Level Determinations

As discussed in Section 2.4.3.3, the water elevation of a flood at the plant site generated by the PMP event is 431.1 ft msl. This flood condition has a higher estimated elevation than any flood of the Columbia River.

2.4.3.6 Coincident Wind Wave Activity

Procedures published by the Corps of Engineers (References 2.4-10 and 2.4-11 were used to determine the wind wave activity. The effective fetch for the predominant July wind direction (north) is 3450 ft (0.65 miles). The effective fetch diagram is shown in Figure 2.4-11. The calculated extreme 2-year over water wind for the north-to-south direction, based on area data, is 63.5 mph. This wind results in a maximum wave height of 4.0 ft, with the assumption of a water depth of 12 ft (the average depth in cross sections 8, 9, and 10). The other potential wind directions ENE and ESE were evaluated but found to be less severe.

The wind setup has been computed to be 0.3 ft, and the maximum wave runup is 1.9 ft on a smooth, 1 on 8 slope of compacted naturally occurring sands and gravels. Therefore, the design water surface elevation is 433.3 ft msl. This is less than the east spray pond wall elevation of 435.0 ft msl.

2.4.4 POTENTIAL DAM FAILURES, SEISMICALLY INDUCED

Analyses of floods resulting from potential dam failures were investigated by the Corps of Engineers for the Columbia River. These studies are consistent with Regulatory Guide 1.59, Revision 2. The flood resulting from the breaching of Grand Coulee Dam is considered in lieu of a seismically induced flood.

In 1951, the Seattle District Corps of Engineers made a confidential study (now declassified) to determine artificial flood hydrographs and the flood profile in the Columbia River Valley resulting from breaching the Grand Coulee Dam by enemy attack. The studies covered a spectrum of conditions in terms of breach openings and hydrologic conditions that might prevail at the time of attack. A postulated seismic failure of Grand Coulee Dam could result in displacement of part of the structure, but it would still act as a restriction or weir and minimize the hydraulic failure. For this reason, the explosion-induced artificial flood represents an upper limit to seismically induced failures. The failure of Grand Coulee Dam would initiate a catastrophic flood, which would be augmented by the failure of the earth portions of downstream dams and subsequent release of the storage pools. Figure 2.4-5 shows water surface profiles for RM 323 to RM 358 for various river flows, including Artificial Flood No. 1. This flood provides a "limiting case" assessment of the conservatism of CGS elevation. This flood would have an outfall peak of 8,800,000 ft^3/sec at Grand Coulee Dam at the moment of breaching and a peak discharge at RM 338 (Richland) of 4,800,000 ft^3/sec.

A base flow of 50,000 ft^3/sec was assumed above the mouth of the Snake for this flood (Reference 2.4-12).

An arbitrarily assumed dramatic 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 Corps of Engineers states (Reference 2.4-13) that the river channel restrictions at Trail, British Columbia, would restrict river flow to about 3.1×10^6 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 Bureau of Reclamation (Reference 2.4-14) 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.

Various studies (References 2.4-12, 2.4-15, 2.4-16 and 2.4-17) made by the Corps of Engineers, and others, since 1951 have considered that breaching of Grand Coulee Dam would represent the worst catastrophic event for downstream Columbia River occupants. Although these studies bear no relationship to flooding from natural causes, they have been used as the basis for a very conservative, limiting case approach.

Figure 2.4-5 shows water surface profiles for RM 323 to RM 395 for artificial and real stage flows, one of which corresponds to Artificial Flood No. 1 noted earlier, which has been established (Reference 2.4-18) as conservative (limiting case) criteria for Columbia River flooding. Since the base flow used to develop these curves was 50,000 ft^3/sec, an additional 570,000 ft^3/sec is added to account for simultaneous occurrence of the regulated SPF.

2.4.4.1 Dam Failure Permutations

The effect of potential dam failure on the water levels at the site is determined using the following assumptions:

a. The Columbia River is at flood stage, with a SPF (570,000 ft^3/sec regulated);

b. The reservoirs in each storage pool are full;

c. A massive hydraulic failure occurs at Grand Coulee Dam, releasing 8,800,000 ft^3/sec;

d. Following the above assumed failure, all downstream dams between CGS site and Grand Coulee Dam suffer some degree of failure and release their storage reservoirs to the flood. [The result of a stability analysis (Reference 2.4-15) showed that all mass concrete portions of the dams will resist sliding and overturning with the possible exception of part of Rock Island Dam.];

e. The explosion-induced failure of Grand Coulee Dam represents a more severe failure than any seismic event because of the failure mechanism;

f. Failure of Arrow and/or Mica Dam could result in greater release of storage volume than Grand Coulee Dam; however, the peak flow is limited to 3,100,000 ft^3/sec due to channel restrictions at Trail, British Columbia; and

g. Overtopping of Grand Coulee Dam would occur with failure of Arrow and/or Mica Dams in Canada. The failure of Grand Coulee, as a result of overtopping, is not considered to be a credible event in view of its concrete construction and rock abutments.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

The flood hydrographs developed by the Corps of Engineers are based on the results of extensive studies of the physical characteristics of the flood route (References 2.4-12 and 2.4-15). Subsequent studies made by the Corps of Engineers verify these results (Reference 2.4-17). Water levels following such a flood would depend on the status of reservoir storage upstream from Grand Coulee Dam but, without regulation of some dams, would approximate the natural seasonal flow conditions.

2.4.4.3 Water Level at Plant Site

The water elevations associated with limiting case flood (LCF) levels are shown in Figure 2.4-5. RM 350 provides the control for backwater flow to the plant area which is sheltered by higher ground east of WNP-1 and WNP-4.

Elevation at RM 350 (dam breach flood 4,800,000 ft^3/sec plus SPF, 570,000 ft^3/sec)	=	422 ft msl
Allowance for simultaneous wind and wave action	+	2 ft
Elevation:		424 ft msl = LCF

An adequate margin exists between the resultant flood elevation and the plant elevation of 441 ft msl.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

The location of the CGS site is not close to any water body which experiences seiche flooding. Thus the site is not vulnerable to such flooding.

2.4.6 PROBABLE MAXIMUM TSUNAMI FLOODING

The location of the CGS site is in south-central Washington and it is not adjacent to any coastal area. It is not, therefore, vulnerable to tsunami flooding.

2.4.7 ICE EFFECTS

Historically, the Columbia River has never experienced complete flow stoppage or significant flooding due to ice blockage. Periodic ice blocking has caused reduced flows and limited flooding for only relatively short periods of time. *The most significant icing in recent recorded history occurred during the winter of 1936-37 prior to the construction of the upstream regulating dams. A relatively thick sheet of ice formed across the river. The minimum flow recorded near the Priest Rapids Dam site during this winter was 20,000 cfs. However, the ice forming on the river was caused primarily by the low flow rather than the reverse. The deltaic mouths of many of the tributaries to the Columbia River are frequently blocked by ice causing backup of flood waters. No instance of complete stoppage is known to have occurred.*

Ice blockage is most likely to occur when water temperatures are already low, when flows are small, and when a significant cold spell occurs. With the completion of Grand Coulee and other dams on the Columbia River main stream, the seasonal temperature and flow cycles have drastically changed. These changes will further aid to reduce the intensity and timing of the conditions which may contribute to potential ice blockage and flooding situations. Also average river flow rates, during the winter months, have been increased significantly. The water temperatures have shown a shift in time such that the peak temperatures occur 30-45 days later than formerly. In addition, the low extreme temperatures measured have risen over the years.

The long term trends of temperatures in the Columbia River been studied (Reference 2.4-19) using a 37 year record of measured temperatures. The trends for the maximum, average and minimum temperatures are shown in Figure 2.4-12. The erection of dams on the upper Columbia River has caused the extreme high and low river temperatures measured at Rock Island Dam (Columbia RM 453, 101 miles above the CGS site) to converge toward the average. Winter water temperatures are considerably warmer and summer temperatures cooler with a slightly lowered average of less than 1°F occurring during the 37 years.

On the basis of these studies and the recorded observation of 25 years of operation of the Hanford plutonium production plants, it is concluded that the potential for ice blockage or the combination of blockage and flooding behind ice dams is so low as to be considered insignificant. The erection of Mica, Arrow, and Libby Dams in the Columbia River Basin headwaters is expected to further raise winter water flows and also to increase winter water temperatures somewhat.

In any event, ice flooding will not effect the capability to shut down the reactor in a safe and orderly manner. Also, the daily fluctuating stage of the river at the intake location will discourage formation of sheet ice as well as ice jams. Ice flows, should they occur, will normally pass over intake structure due to relatively high winter discharge in the river.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

There are no cooling water canals. The two spray ponds located southeast of the reactor building designed as Seismic Category I structures, have reinforced concrete side walls, and reinforced concrete base mats at el. 420 ft msl. The finished grade at the spray ponds is approximately at el. 434 ft msl and have top of wall elevations of 435 ft msl. The spray ponds are the ultimate heat sink for normal reactor cooldown and for emergency cooling.

The spray ponds are a part of the standby service water system which is discussed in Section 9.2.7. See also Section 2.4.11.6.

During normal reactor operation, the cooling water necessary for the plant is supplied from the cooling tower basins.

2.4.9 CHANNEL DIVERSIONS

The Columbia River flow in the Hanford reach is controlled to a large extent by regulation of the upstream reservoir projects. The riverbed in the vicinity of the site is well defined and it is very unlikely that the riverbed 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.

As discussed in Section 2.4.7, the river has not frozen over in Hanford reach during at least the past 25 years, and icing on the river has not been a problem at pump house or outfall structures associated with the plutonium production plants.

2.4.10 FLOODING PROTECTION REQUIREMENTS

The design considerations of safety-related facilities to withstand floods and flood waves are described in Section 2.4.2.2. The PMF is discussed in Section 2.4.3.

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. Flood considerations are described in Section 3.4 and the design of Seismic Category I structures, for all conditions including flood, is described in Section 3.8.

In the event of a flood at the site, it will be possible to place the plant in a safe shutdown condition.

All non-safety-related facilities with the exception of the makeup water pump house, are above the LCF elevation. The flooding of the makeup water pump house would not affect safety-related equipment and would not affect the safe shutdown of the plant. The approximate finished grade at all Seismic Category I structures except the spray ponds is at elevation 440 ft msl. The finished grade of the spray ponds is 434 ft msl.

The PMF elevation of the Columbia River (described in Section 2.4.3), at the site, is estimated to be 390 ft msl.

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 that due to wind and wave action.

2.4.11 LOW WATER CONSIDERATIONS

As described in Section 2.4.1.1, plant water needs are supplied through an intake structure in the Columbia River. The top of the makeup water intake screens (at RM 352) are set below the water surface elevation that would be associated with the minimum allowable flow (36,000 cfs) at the federally licensed Priest Rapids Dam (at RM 397). Water levels at the CGS intake are not influenced by backwater from the downstream McNary Dam (RM 292). The Columbia River Basin upstream of CGS has in excess of 35 million acre-ft of usable reservoir storage capacity. Because of this storage and highly regulated river flows, it is improbable that flows below the licensed minimum will occur. Based on data for 1961 through 1994, 7-day low flow with a recurrence interval of 100 years has been estimated at 44,500 cfs.

Even if some event (e.g., very severe drought) caused the makeup water system to be inoperable, the loss of water would not compromise the safe shutdown of the plant. As is discussed in Sections 9.2.5 and 9.2.7, shutdown cooling water is supplied by the ultimate heat sink which contains a 30-day supply of water in two spray ponds. The only scenario in which the makeup water pump house is called on to supply water in an emergency situation is when a tornado removes a significant quantity of spray pond water (see Section 9.2.5.3). Therefore, the low river water condition is not a situation requiring safety-related features and procedures.

2.4.11.1 Low Flow in Streams

Reservoir projects on the Columbia River Basin upstream of the proposed site have a total usable storage capacity in excess of 35 million acre-ft. This capacity is sufficient to maintain a flow in the Columbia River, at the proximity of CGS, of 36,000 ft^3/sec for over 1 year with absolutely no inflow from other sources. Because of this regulation, the anticipated minimum

3.4.1.2.2 Breach of the Grand Coulee Dam

The effects on the groundwater table due to a breach of the Grand Coulee Dam is discussed in Section 2.4.4. This flood would peak for a short time and would not affect the design-basis groundwater level of 420 ft msl due to its short duration.

3.4.1.2.3 Design Basis Flood Probable Maximum Precipitation

As stated in Section 3.4.1.1.1, the worst DBF condition results from the PMP event. Due to the short duration of this flood and its confines, groundwater level at the site would not be affected.

3.4.1.3 Identification of Structures, Systems, and Components

Seismic Category I structures and safety-related systems and components are identified in Section 3.2. Figures 1.2-1 through 1.2-24 show locations and elevations for systems and components. See Section 2.4.2 for discussion of flood protection of safety-related systems and components.

3.4.1.4 Description of Structures, Systems, and Components

3.4.1.4.1 Flood Protection Requirements

3.4.1.4.1.1 External Flood Protection Requirements. The plant site elevation at Seismic Category I structures and safety-related systems and components is approximately 441 ft msl, except at the spray ponds where the finish grade elevation is 434 ft msl and the top of spray ponds walls is 435 ft msl. These elevations are sufficient to protect the plant site and, therefore, Seismic Category I structures and the safety-related systems and components housed therein against the DBF. Exterior and access openings to all Seismic Category I structures are located above the plant site grade and, therefore, above the DBF level. Flood protection measures are not provided since they are not required.

3.4.1.4.1.2 Internal Flood Protection Requirements. Section 3.4.1.5.2 discusses internal flood protection measures provided for safety-related systems, equipment, and components.

Figures 1.2-2, 1.2-7 through 1.2-17, and 1.2-22 illustrate plant arrangement and layout and show that safety-related equipment is located within individual rooms or compartments. The pump rooms located on the 422 ft 3 in. elevation are enclosed by reinforced-concrete walls. Penetrations and doors in these walls are provided with seals that minimize the effects of flooding between rooms should a break occur in one of the rooms.