EA-12-049
10 CFR 50.54(f)

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102-06967-DCM/TNW
December 12, 2014

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

References:
1. NRC Letter, *Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident*, dated March 12, 2012
2. NRC Letter, *Palo Verde Nuclear Generating Station, Units 1, 2, and 3 – Relaxation of Response Due Dates Regarding Flooding Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of the Insights from the Fukushima Dai-ichi Accident,* dated July 29, 2014

Dear Sirs:

Subject: **Palo Verde Nuclear Generating Station (PVNGS)**
**Units 1, 2, and 3**
**Docket Nos. STN 50-528/529/530**
**Flood Hazard Reevaluation Report**

In accordance with the NRC request for information documented in Reference 1, enclosed please find the *Flood Hazard Reevaluation Report for Palo Verde Nuclear Generating Station Units 1, 2, and 3*.

Subsequent to the issuance of the request for information, the NRC issued a prioritization of the due dates for the submittal of the flood hazard reevaluation report for all sites. The NRC set the reevaluation due date for PVNGS as March 12, 2014. The initial flood hazard reevaluation report for PVNGS concluded that the following events and corresponding results were found to be comparable with the current licensing basis:

- Probable maximum flood (PMF) for Winters Wash
- Sitewide inundation as a result of local intense precipitation (LIP)
- Dam failure

However, the flood levels predicted by the initial flood hazard reevaluation were somewhat greater than expected for the following:

- Areas adjacent to the powerblock structures due to LIP
- East Wash embankment due to PMF



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NRR

As a result, Arizona Public Service Company (APS) requested and received approval for an extension of the reevaluation report due date to December 12, 2014, in Reference 2. The purpose of the extension was to complete a reanalysis by an independent contractor using refined analysis techniques and a room-by-room internal flooding analysis by APS.

The enclosure to this letter contains the results from the initial and refined flood hazard reevaluation, as well as the room-by-room internal flooding analysis completed by APS during the extension period.

No operator or mitigation actions are needed to ensure safe shutdown capability as a result of the flood hazard reevaluation. As no additional actions to protect against the reevaluated flood hazards are needed and the results are comparable to the licensing basis, APS believes that an integrated assessment is not needed or warranted.

No new commitments are being made to the NRC by this letter. Should you need further information regarding this submittal, please contact Thomas N. Weber, Licensing Department Leader, at (623) 393-5764.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on 12/12/14
(Date)

Sincerely,

D.C. Mims

DCM/TNW

Enclosure: *Flood Hazard Reevaluation Report for Palo Verde Nuclear Generating Station Units 1, 2, and 3*

| | | |
|---|---|---|
| cc: | M. L. Dapas | NRC Region IV Regional Administrator |
| | B. K. Singal | NRC NRR Project Manager for PVNGS |
| | M. M. Watford | NRC NRR Project Manager |
| | D. R. Reinert | NRC Acting Senior Resident Inspector for PVNGS |

# FLOOD HAZARD REEVALUATION REPORT

for

PALO VERDE NUCLEAR GENERATING STATION
UNITS 1, 2, AND 3

REVISION 0
DECEMBER 12, 2014

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## ACRONYMS

| | |
|---|---|
| ADWR | Arizona Department of Water Resources |
| AGS | Arizona Geological Survey |
| ANS | American Nuclear Society |
| ANSI | American National Standards Institute |
| APS | Arizona Public Service Company |
| AWA | Applied Weather Associates |
| CAP | Corrective Action Program |
| CEF | combined effects flooding |
| CEM | Coastal Engineering Manual |
| CFR | Code of Federal Regulations |
| CLB | current licensing basis |
| DAD | depth-area-duration |
| ESP | essential spray pond |
| ESRI | Environmental Systems Research Institute |
| EW | East Wash |
| FCDMC | Flood Control District of Maricopa County |
| FEMA | Federal Emergency Management Agency |
| HEC-HMS | Hydrologic Engineering Center-Hydrologic Modeling System |
| HEC-RAS | Hydrologic Engineering Center-River Analysis System |
| HHA | hierarchical hazard assessment |
| HMR | Hydrometerological Report |
| LIP | local intense precipitation |
| msl | mean sea level |
| MSW | maximum sustained wind |
| NCDC | National Climatic Data Center |

## ACRONYMS (CONTINUED)

| | |
|---|---|
| NEXRAD | next generation radar |
| NGVD | National Geodetic Vertical Datum |
| NRC | Nuclear Regulatory Commission |
| NRCS | National Resource and Conservation Service |
| NWS | National Weather Service |
| PMF | probable maximum flood |
| PMP | probable maximum precipitation |
| PVNGS | Palo Verde Nuclear Generation Station |
| REI | Riada Engineering, Inc. |
| RG | Regulatory Guide |
| SCS | U. S. Soil Conservation Service |
| SOCA | Security Owner Controlled Area |
| SSC | structures, systems, and components |
| UFSAR | Update Final Safety Analysis Report |
| USACE | United States Army Corps of Engineers |
| USBR | United States Bureau of Reclamation |
| USDA | United States Department of Agriculture |
| USGS | United States Geological Survey |
| VBS | vehicle barrier system |
| WW | Winters Wash |

## EXECUTIVE SUMMARY

This report provides a reevaluation of potential flood causing mechanisms at Palo Verde Nuclear Generating Station, (PVNGS) Units 1, 2, and 3, with consideration of the present-day regulatory guidance and methodologies being used for combined license reviews, including current techniques, software, and methods used in present-day standard engineering practice. The flood hazards considered include local intense precipitation, flooding from the nearby washes, and potential dam failure flooding. Other flood causing mechanisms, such as tsunami, storm surge, seiche, ice-induced flooding, and channel diversion effects, were excluded as not being applicable based on the characteristics of the site.

Subsequent to the issuance of the Nuclear Regulatory Commission (NRC) request for information on March 12, 2012, (NRC, 2012a) pursuant to Title 10 of the Code of Federal Regulations (CFR), Section 50.54(f), the NRC issued a prioritization of the due dates for the submittal of the flood hazard reevaluation report for all sites. The NRC set the reevaluation due date for PVNGS as March 12, 2014. The initial flood hazard reevaluation report was performed by Paul C. Rizzo Associates, Inc. under a subcontract with Westinghouse Electric Company.

The initial flood hazard reevaluation report concluded that the following events and corresponding results were found to be comparable with the current licensing basis:

- Probable maximum flood (PMF) for Winters Wash
- Sitewide inundation as a result of local intense precipitation (LIP)
- Dam failure

However, the flood levels predicted by the initial flood hazard reevaluation were somewhat greater than expected for the following plant locations:

- Areas adjacent to the powerblock structures due to LIP
- East Wash embankment due to PMF

As a result, Arizona Public Service Company (APS) requested and received approval for an extension of the reevaluation report due date to December 12, 2014, to complete a reanalysis by an independent contractor using refined analysis techniques and a room-by-room internal flooding analysis by APS.

This report contains the results from the initial and refined flood hazard reevaluation, as well as the room-by-room internal flooding analysis completed by APS during the extension period.

The analyses completed during the extension period included the following specific areas:

- Utilized FLO-2D software to model the impacts on the site caused by a PMF in East Wash
- Updated the combined effects analysis in this report to reflect the refined analysis of East Wash
- Performed a room-by-room internal flooding analysis of the potential impact on safe shutdown equipment due to water intrusion from a LIP event. Subsequent refined analysis of LIP in the powerblock validated that the depth and duration of water accumulation (hydrographs) used in the room-by-room internal flooding analysis were conservative.

The results of the refined PMF analysis of East Wash showed flood levels comparable to the current licensing bases with adequate freeboard to contain the PMF including wave runup. The combined effects analysis of the PMF event in Winters Wash and East Wash was bounded by the individual PMF event in each individual wash. The room-by-room internal flooding analysis conservatively utilized the results from the initial LIP analysis and concluded there was no impact to safe shutdown equipment.

No operator or mitigation actions are needed to ensure safe shutdown capability as a result of the flood hazard reevaluation. As no additional actions to protect against the reevaluated flood hazards are needed and the results are comparable to the licensing basis, APS believes that an integrated assessment is not needed or warranted.

# 1.0 INTRODUCTION

## 1.1 Purpose and Scope

The Nuclear Regulatory Commission (NRC) issued a request for information on March 12, 2012, (NRC, 2012a) pursuant to Title 10 of the Code of Federal Regulations (CFR), Section 50.54(f), related to the implementation of Recommendations 2.1, 2.3, and 9.3 from the Near Term Task Force, a portion of which calls for performing flood hazard reevaluations at all nuclear power plants in the United States. This Flood Hazard Reevaluation Report for the Palo Verde Nuclear Generating Station (PVNGS) Units 1, 2, and 3 provides the information required to address NRC Recommendation 2.1 with consideration of the present-day regulatory guidance and methodologies being used for combined license reviews, including current techniques, software, and methods used in present-day standard engineering practice.

The site (geographic location shown in *Figure 1-1* is licensed for the operation of three Combustion Engineering System 80 pressurized water reactor nuclear generating units. The original operating licenses for Units 1, 2, and 3 were issued June 1, 1985, April 24, 1986, and November 25, 1987, respectively. The operating licenses for all three units were renewed on April 21, 2011, and will expire June 1, 2045, April 24, 2046, and November 25, 2047, respectively.

Revision 17 to the Updated Final Safety Analysis Report (APS, 2013) for PVNGS Units 1, 2, and 3 was issued to the NRC in June 2013.

### 1.1.1 Flood Hazard Reevaluation Extension

Subsequent to the issuance of the NRC request for information, the NRC issued a prioritization of the due dates for the submittal of the flood hazard reevaluation report for all sites. The NRC set the reevaluation due date for PVNGS as March 12, 2014.

The initial flood hazard reevaluation report concluded that the following events and corresponding results were found to be comparable with the current licensing basis:

- PMF for Winters Wash
- Sitewide inundation as a result of LIP
- Dam failure

However, the flood levels predicted by the initial flood hazard reevaluation were somewhat greater than expected for the following plant locations:

- Areas adjacent to the powerblock structures due to LIP
- East Wash embankment due to PMF

As a result, Arizona Public Service Company (APS) requested and received approval for an extension of the reevaluation report due date to December 12, 2014, to complete *a reanalysis by an independent contractor using refined analysis techniques and a* room-by-room internal flooding analysis by APS

This report contains the results from the initial and refined flood hazard reevaluation, as well as the room-by-room internal flooding analysis completed by APS during the extension period.

The analyses completed during the extension period included the following specific areas:

- Utilized FLO-2D software to model the impacts on the site caused by a PMF in East Wash
- Updated the combined effects analysis in this report to reflect the refined analysis of East Wash
- Performed a room-by-room internal flooding analysis of the potential impact on safe shutdown equipment due to water intrusion from a LIP event. Subsequent refined analysis of LIP in the powerblock validated that the depth and duration of water accumulation (hydrographs) used in the room-by-room internal flooding analysis were conservative.

## 1.2 Hydrologic Description of Study Area

The site is located in Maricopa County, AZ at approximately 33°23′ North latitude and 112°52′ West longitude. The site is isolated from maritime bodies of water and is approximately 46 miles west of the center of Phoenix (*Figure 1-1*). Two desert streams, Winters Wash and East Wash, are located to the west and east of the site, respectively, as shown in *Figure 1-2*.

The site is located in a dry, desert region adjacent to the Palo Verde Hills. The terrain has very little topographic relief and slopes gently southward. Palo Verde is considered a "dry site" in accordance with the definition contained in Regulatory Guide (RG) 1.102, Revision 1 (NRC, 1976). As defined in the RG, a dry site is a site where the plant is built above the Design Basis Flooding Level, and therefore safety-related structures, systems and components (SSCs) are not affected by external flooding. The grade elevations [mean sea level (msl)] of Seismic Category I structures are 957.5 ft for Unit 1, 954.5 ft for Unit 2, and 951.5 ft for Unit 3 (UFSAR Figure 2.4-4).

The vertical datum used in the UFSAR is msl. At the site, the msl datum is equated with the National Geodetic Vertical Datum of 1929 (NGVD29), which, prior to 1973, was referred to as the Sea Level Datum of 1929 (USGS, 2013a). Equivalency between the msl datum and NGVD29 is apparent in a 1962 USGS map (USGS, 1962) covering the site area. In this Flood Hazard Reevaluation Report, all elevations are provided in NGVD29, unless stated otherwise.

Additional dry rivers and washes in the vicinity of the site include the Gila River and two of its tributaries, the Hassayampa River and the Centennial Wash. The Gila River's nearest approach is approximately six miles southeast of the site. The Hassayampa

River and Centennial Wash are located approximately five miles east and five miles south of the site, respectively. Luke Wash is further east than East Wash and the discharge from its watershed is conservatively combined in this study with the Hassayampa River.

## 1.3 Site Flood Hazard Background and History

East Wash and Winters Wash discharge to Centennial Wash (UFSAR Figure 2.4-1), which discharges into the Gila River upstream of the Gillespie Dam. The Hassayampa River also discharges into the Gila River. Although there is no stream gage on East Wash, a study of the available United States Geological Survey (USGS) stream gage data for the other four watercourses found no published information to indicate that flooding on any of these conveyances has resulted in water depths that would constitute a flood hazard at the site.

Additional details concerning historic flooding along these rivers and washes are provided in the following discussion. The stream gage and discharge rate information is provided through the USGS (USGS, 2013b).

Gila River Flooding

The Gillespie Dam is approximately 12 miles southeast of the site. The Gila River watershed upstream of the Gillespie Dam has an area of approximately 50,000 square miles (sq mi) (USGS, 2013b), including watersheds of East Wash, Winters Wash, Centennial Wash, and the Hassayampa River.

Systematic reporting of estimated discharges on the Gila River upstream of the Gillespie Dam began in 1888 (USACE, 1957). The largest flood of record is for February 1891 with an estimated discharge of 250,000 cubic feet per second (cfs) at the site of Gillespie Dam. Peak annual flood flows for this gage (USGS gage 09519000) through 1977 are reported in UFSAR Table 2.4-5. Significant flood events reported since 1977 and the discharge rates include the following:

- 1979 125,000 cfs
- 1984 95,200 cfs
- 1989 178,000 cfs
- 1993 130,000 cfs

A second gage along the Gila River is USGS Gage 09514100 for the Gila River at Estrella Parkway, near Goodyear, AZ. This gage is approximately 30 miles upstream of the Gillespie Dam along the Gila River and five miles downstream of inflows along the Salt River. The period of record is from 1993 to 2013, with the two highest flows recorded as 162,000 cfs in 1993 and 74,900 cfs in 1995.

Hassayampa River Flooding

The Hassayampa River discharges to the Gila River at a location approximately 9 miles east of the site. USGS Gage 09517000 for the Hassayampa River near Arlington, AZ is located approximately 1.8 miles upstream of the confluence with the Gila River. The period of record for this gage is 1961 through 2012. The two largest annual peak floods for this period are 39,000 cfs in 1970 and 22,000 cfs in 2001.

Peak flows for USGS Gage 09516500 for the Hassayampa River near Morristown, AZ are reported in UFSAR Table 2.4-4. The peak flow rate for this location of 47,500 cfs on September 5, 1970, remains the largest flood recorded at the gage station.

Centennial Wash Flooding

The Centennial Wash discharges to the Gila River just upstream of the Gillespie Dam, approximately six miles south of the site. USGS Gage 09517490 is located on this wash at the Southern Pacific Railroad Bridge near Arlington, AZ, providing flow records from 1983 to the present. The two largest annual peak floods recorded at this gage are 15,600 cfs in 1984 and 9,210 cfs in 1993.

A second gage along the Centennial Wash (USGS Gage 09517500) near Arlington, AZ has flow records for the period of 1961 through 1978. The largest flood for this gage is 14,500 cfs occurring in 1961. The next largest flood for this gage is 11,900 cfs in 1970. This gage was the source for the flows reported in UFSAR Table 2.4-2.

Winters Wash Flooding

USGS Gage 09517400 on Winters Wash near Tonopah, AZ is located approximately 8 miles northwest of the site. The two largest annual peak floods of record for this gage are 3,640 cfs in 1976 and 2,100 cfs in 1972. This gage was used for the flows reported in UFSAR Table 2.4-3.

## 1.4 Design Basis of the Plant

The onsite drainage system is designed such that runoff due to probable maximum precipitation (PMP) will not inundate safety-related structures, equipment, and access to those facilities. Areas adjacent to the powerblock are sloped away at 0.5% to 1%, resulting in a minimum drop of 5 to 7 feet at the peripheral drainage system. The design basis calculated maximum water surface elevations due to local PMP storm are 955.5 ft, 952.5 ft, and 949.5 ft at Units 1, 2, and 3, respectively. These maximum flood elevations are 2.0 feet below the grade elevations at the respective units (UFSAR Section 2.4.2.3).

## 1.5 Basic Approach of the Flood Hazard Reevaluation

As stated earlier in this report, the initial flood hazard reevaluation report was conducted by Rizzo, a subcontractor to Westinghouse Electric Corporation. The scope of this initial reevaluation is shown in *Figure 1-3* and includes the following analyses that were performed in accordance with NUREG/CR-7046 (NRC, 2011):

- PMF for both Winters Wash and East Wash
- Sitewide inundation as result of LIP
- Dam failure
- Combined effects

In this report, the first reevaluation done by Rizzo is referred to as the "initial flood hazard reevaluation." The initial flood hazard reevaluation predicted flood levels for a LIP that were somewhat greater than expected for areas adjacent to the powerblock structures and also for PMF flood levels for East Wash. APS requested an extension from the NRC in order to have additional analyses done using specific refinements. One of the elements of the refined analysis was to use FLO_2D software (FLO-2D, 2012) to model the impact on the site caused by a PMF in East Wash.

Another element of the refined analysis was to utilize a modified version of the FLO-2D software model that specifically accounted for roof detention (parapet walls), scupper inlets, scupper outlets, and downspouts. This enhancement to the FLO-2D program was done by the FLO-2D developers specifically for this project. The flow to the ground for the LIP event is attenuated on the roof and discharged to specific locations, in lieu of directing runoff directly to the ground. This was intended to address the unexpected results for the higher flood levels for areas adjacent to the powerblock structures.

The final action was for APS to perform a room-by-room internal flooding analysis. Due to the time constraints of having to complete the flood hazard revaluation by December 12, 2014, the room-by-room internal flooding analysis had to be performed before completion of the refined analyses. Therefore, the results from the second element of the refined analysis (LIP) were not available to use as an input to the room-by-room internal flooding analysis. The lower flood levels obtained for the second refinement were used to validate margin for the depth and duration of water accumulation in the areas adjacent to the powerblock structures.

### 1.5.1 Probable Maximum Precipitation

The original design basis PMP was used to determine the PMF levels in the watercourses near the site that might contribute to flooding of the site, specifically Winters Wash and East Wash. The design basis PMP for Winters Wash was calculated using the Hershfield method based on the statistics of extreme events (UFSAR Section 2.4.3.1.1), while the PMP for East Wash was obtained using extreme summer thunderstorm rainfall for the southwest (USFAR Section 2.4.3.1.2).

The LIP[1] was calculated for the current design basis using the 1972 Preliminary Probable Maximum Thunderstorm Precipitation Estimates Southwest States Report (NWS, 1972) by the National Weather Service (NWS) (UFSAR Table 2.4-6).

For the initial and refined flood hazard reevaluation, the most up-to-date PMP estimation methodology for the state of Arizona was applied to develop the LIP hyetograph using a PMP evaluation tool developed by Applied Weather Associates (AWA) for the Arizona Department of Water Resources (ADWR) (AWA, 2013).

## 1.6 Probable Maximum Flood Reevaluation

The design basis 24-hour PMP produced the most severe design basis PMF for Winters Wash, while the design basis 6-hour thunderstorm produced the most severe design basis PMF for East Wash (UFSAR Section 2.4.3.1). For the initial and refined reevaluation analyses, PMP hyetographs were developed and applied over their corresponding watersheds to characterize the design basis PMF within the two desert watersheds adjacent to the site. The hyetographs were determined using the AWA tool, which provides PMP values for three different storm types: local storms (i.e., thunderstorms), general winter storms, and tropical storms.

Winters Wash and East Wash Reevaluation

For East Wash, the depth and duration of flooding, maximum flood velocities, and hydraulic forces associated with flooding at the site were determined in the initial analysis using the FLO-2D flood routing program (FLO-2D, 2012).

Inputs to the model included the flood hydrographs for each wash using the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) and rainfall based on the PMP hyetograph for each watershed.

The initial analysis calculations were refined to include additional detailed data and methodology in accordance with the hierarchical hazard assessment (HHA) approach in NUREG/CR-7046 (NRC, 2011) utilizing a two-dimensional hydraulic model (FLO-2D, 2014a) to calculate the impacts on the site caused by PMF in East Wash. A two-dimensional model simulates the flow of water more accurately than a one-dimensional model such as HEC-HMS (USACE, 2010a) in combination with the Hydrologic Engineering Center-River Analysis System (HEC-RAS) (USACE, 2010b). It had been determined that flooding in Winters Wash does not affect the site, so the analysis for Winters Wash was not refined.

[1] The NRC defines LIP as a 1 hour, 1 sq mi PMP event located at the site. The UFSAR uses the term "local intense precipitation," but not the abbreviation "LIP." The UFSAR also uses the term point-value PMP when referring to the rainfall associated with the LIP (UFSAR 2.4.3.2).

### 1.6.1 Local Intense Precipitation in the Powerblock

The most up-to-date PMP estimation methodology for the state of Arizona was applied to develop the LIP hyetograph for the powerblock using the PMP evaluation tool developed by AWA. The hyetograph was developed for the Security Owner Controlled Area (SOCA), which is approximately 1 sq mi and encompasses safety-related SSCs at the site. It was assumed to rain evenly over the FLO-2D model domain (approximately four sq mi). The resulting cumulative 6-hour rainfall depth was 12.80 inches following the methodology of the AWA tool. The maximum rainfall in one hour within the LIP hyetograph was 10.73 inches.

### 1.6.2 Effects of LIP on Safety-Related SSCs in the Powerblock

UFSAR Section 2.4.2.3 states that areas adjacent to the powerblock are sloped away at 0.5 to 1%. This results in a minimum drop of 5 to 7 feet at the peripheral drainage system, as compared to the grade elevation at each unit. Computer and hand calculation methodologies used during initial design did not have the capability to predict minor accumulation adjacent to the entrances of buildings. Therefore, the licensing bases did not provide a specific value for the transient water accumulation phenomenon. However, the design of powerblock structures did include sufficient capability to mitigate internal flooding resulting from high and moderate energy line breaks, which was implicitly assumed to bound the effects of external flooding from localized transient water accumulation during the LIP event. These design features include independent four-inch drain headers, pedestals, curbs, check valves and room train separation, and a large holdup capacity at the lower elevations of each building. These passive design features provide an inherently safe design for localized transient water accumulation during a LIP event and the corresponding internal flooding of buildings, as further discussed in *Section 3.2.1.6*.

Today, numerical models and software such as FLO-2D can predict a conservative estimate of local transient water accumulation adjacent to the powerblock structures. Appendix B to NUREG/CR-7046 utilizes a one-dimensional flow model and does not provide a method to predict the accumulation of water within the powerblock complex.

The initial flood hazard reevaluation for LIP was performed using FLO-2D. A byproduct of the analysis was hydrographs for the perimeters of each powerblock building. A comprehensive room-by-room internal flooding analysis of water infiltration from LIP was performed using the hydrographs and existing design features for internal flooding. The analysis concluded that infiltrating water from LIP does not impact safe shutdown equipment within these buildings due to the various plant features such as curbs, pedestals, train separation, drains, stairwells, and trenches that redirect or limit water flow into the critical areas of the plant.

Additionally, to understand the margin of the room-by-room internal flooding analysis, a refined model of each powerblock was developed using modified FLO-2D (FLO-2D, 2014b) software that included roof, gutters and downspouts. The refinement provided a more accurate representation of the flow characteristics of roof runoff and consequential transient water accumulation adjacent to structures. This refined LIP for the powerblock provided additional insight to the complex nature of sheet flows surrounding powerblock structures and confirmed that additional margin can be realized by more complex modeling.

# 2.0 FLOOD HAZARDS AT THE SITE

*Section 2.0* has been prepared in response to Item 1.a. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter. This section documents current design basis results, as well as pertinent site information related to the applicable flood hazards.

The current flood hazards are identical to the flood hazards that existed during the initial licensing phase and documented in UFSAR Sections 2.4.2 through 2.4.7, except for the addition of combined effects flooding (CEF) as required by the NRC.

## 2.1 Detailed Site Information

*Section 2.1* has been prepared in response to Item 1.a.i. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter. Relevant site data presented for consideration include the present-day site layout, elevation of pertinent SSCs important to safety, site topography, as well as pertinent spatial and temporal data sets.

### 2.1.1 Design Site Information

Design site information describes characteristics considered for the original licensing basis of the site. Changes to the site layout and SSCs related to flooding protection were discussed and evaluated as part of the *Flooding Walkdown Report* (APS, 2012). These changes were evaluated as part of this hazard reevaluation report with respect to the new guidance and methodologies.

The topographic mapping and site layout are provided in *Figure 2-1* and supplemented by aerial photography *(Figures 2-2 and 2-3)*. The powerblock arrangement is shown in *Figure 2-2.*

Site Topography

Ground surface elevations range from 890 ft at the southern site boundary to nearly 1,030 ft at the northern site boundary (UFSAR Section 2.1.1.2). Protection of safety-related facilities from inundation by offsite flood sources is achieved by the location of the facilities beyond the extent of flooding (UFSAR Section 2.4.2.2.1). The onsite drainage system is designed so that runoff due to PMP will not inundate the safety-related structures, equipment, and access to these facilities (UFSAR Section 2.4.2.3). The existing flooding design bases found in the UFSAR, the structure elevations, and the flood levels are presented in *Table 2-1*. Plant grades for Units 1, 2, and 3 are all 951 ft or above (UFSAR Section 2.4.2.2.2).

East Wash was realigned from its natural course to a location east of the site during site grading and construction activities (UFSAR Section 2.4.10). Flood calculations documented in the UFSAR were based upon the realigned position of East Wash.

Safety-Related SSCs

The locations of the safety-related structures are shown in *Figure 2-2*. A list of the existing flooding elevations found in the UFSAR is presented in *Table 2-1*.

Ultimate Heat Sink

The ultimate heat sink for each unit consists of two independent Seismic Category I essential spray ponds (ESPs) (UFSAR Section 9.2.5) located adjacent to the unit *(Figure 2-2)*. The ESPs for each unit are rectangular reinforced concrete structures able to remain functional following any external event as required by 10 CFR 50 Appendix A, General Design Criterion 2 (UFSAR Section 9.2.5.2).

### 2.1.2 Present-Day Site Information

Site Topography

Changes to site and surrounding topography since licensing have been identified and documented. The site topography was confirmed by aerial mapping as part of the flood hazard reevaluation and recent surrounding topography information was obtained from governmental agencies for use in the PMF reevaluation.

Present-day topographic mapping for the site includes the detailed APS 2013 aerial topographic mapping and digital topographic maps provided by the Flood Control District of Maricopa County (FCDMC). The 2013 site aerial topographic mapping was used to obtain ground and pond surface elevations for the area within the site. The aerial data were supplemented with GPS survey data where shadows caused inaccuracies in the aerial data. The map data obtained from the FCDMC includes:

- Palo Verde mapping (2-foot vertical contours from June 2007)
- Luke Wash and Arlington mapping (2-foot vertical contours from September and December 2005)
- Countywide mapping (10-foot vertical contours from December 2000)

The two-foot vertical contour mapping provided by the FCDMC was used for delineation of the watersheds adjacent to the site for the flood hazard reevaluation. The vertical datum for the topographic contour data is the North American Vertical Datum of 1988 (NAVD88), which was converted to NGVD29. The FCDMC topographic data provides sufficient detail to support the simulation of floods in East Wash and Winters Wash. The topographic data reflects the bottom of the stream beds, not a water surface, because the data was collected when the stream beds were dry.

The topographic data for the on-site ponds and reservoirs reflects the water levels at the time of the 2013 site aerial topographic mapping. The starting water surface elevations used in the flooding analyses were higher than the water levels recorded in the topographic data. Therefore, detailed bathymetric data for these impounded water bodies was not required to conduct the flooding analyses described in *Sections 3.2.1* and *3.2.2*. The topographic data for the areas around the impounded water bodies was sufficient for evaluating potential flooding at the site.

The rivers and washes in the vicinity of the site are the Hassayampa River, Gila River, Winters Wash, Centennial Wash, and East Wash, as shown in *Figure 2-4*.

Safety-Related SSCs

Changes to the site layout and SSCs related to flooding protection were noted as part of the *Flooding Walkdown Report*. Based on field observations, the alterations to the topography by the modifications do not adversely affect the runoff assumed in the current licensing basis (CLB) to the point where it could affect Seismic Category I structures.

The external flooding walkdowns identified some conditions related to features that protect Seismic Category I structures from the effects of PMP and PMF as well as groundwater intrusion. These items were entered into the Corrective Action Program (CAP) and actions are being taken to correct the conditions. The conditions have been addressed to ensure the affected SSCs continue to be functional or operable, as applicable.

Ultimate Heat Sink

The design and design criteria for the ESPs have not changed and were verified by the walkdowns performed in support of NTTF 2.3 as reported in the *Flooding Walkdown Report.*

## 2.2 Current Design Basis Flood Elevations

*Section 2.2* has been prepared in response to Item 1.a.ii. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter. Relevant site data to be considered includes the current design basis flood elevations for all flood-causing mechanisms.

### 2.2.1 Point-Value Probable Maximum Precipitation

For the current design basis, the point-value PMP (equivalent to the new NRC definition of LIP) was calculated using National Weather Service data (NWS, 1972), which was eventually issued as Hydrometeorological Report No. 49 (HMR 49) (NOAA, 1977). The rainfall depth was computed to be 11.8 inches for a duration of 1-hour and 15.53 inches for a duration of six hours (UFSAR Table 2.4-6).

The current design basis point-value PMP calculations assumed zero infiltration losses and complete blockage of the drainage culverts. The occurrence of snow and ice accumulation coincident with the point-value PMP was not considered to be a probable event. The maximum local flooding water surface elevations due to the point-value PMP event were 955.5 ft, 952.5 ft, and 949.5 ft at Units 1, 2, and 3, respectively, which are two feet below the floor elevations of the respective units (UFSAR Section 2.4.2.3).

### 2.2.2 Probable Maximum Flood on Rivers and Streams

PMP hyetographs were developed and applied over the corresponding watersheds to characterize the design basis PMF within the two desert watersheds adjacent to the site, Winters Wash and East Wash (UFSAR Figure 2.4-1).

Winters Wash

The PMP for the Winters Wash watershed was calculated using the Hershfield method. The PMP that produces the most severe PMF for the Winters Wash watershed was the 24-hour PMP, which has a cumulative rainfall of 14.6 inches. (UFSAR Section 2.4.3.1.1). The current design basis PMF flow rate calculated for Winters Wash is 172,400 cfs at cross-section D (UFSAR Table 2.4-16). The maximum water surface elevations for the PMF on Winters Wash at cross-sections near the site (UFSAR Figure 2.4-2) range from 929.5 ft at cross-section D to 956.4 ft at cross-section AA, including wind-wave run-up. These flood levels do not adversely affect the site as important to safety SSCs are not inundated by the PMF in Winters Wash (UFSAR Section 2.4.3).

The current design basis combined wind setup and run-up heights are provided in *Table 2-2*.

East Wash

The UFSAR states that flood protection will be achieved by site grading such that all Seismic Category I facilities will be located beyond the extent of PMF (UFSAR Section 2.4.10). It further states that:

> *East Wash has been realigned along the eastern edge of the site to maximize use of the site for other facilities and to limit the extent of the PMF. The normal channel of East Wash has been blocked by an embankment between the two hills on the northern edge of the site. This embankment forces flood flows around the small hill in the northeast corner of the site and cuts off any flow through the old channel. An additional embankment has been constructed along the eastern edge of the site to prevent flooding of the site proper.*

The PMP for the East Wash watershed was calculated using National Weather Service data (NWS, 1972). The 6-hour PMP with a cumulative rainfall of 14.44 inches caused the most severe PMF for East Wash. The current design basis PMF flow rate calculated for East Wash is 16,600 cfs (UFSAR Table 2.4-7). The water surface elevations due to PMF on East Wash at cross-sections near the site range from 926.6 ft at cross-section F to 978.8 ft at cross-section G2 (UFSAR Figure 2.4-2, and UFSAR Table 2.4-16). These flood levels do not adversely affect the site at the associated cross-sections. Accordingly, the UFSAR concluded that all Category I facilities are safe from inundation by the PMF on (or from) East Wash (UFSAR Section 2.4.3).

The maximum flood elevation for the current design basis combined wind setup and run-up heights is provided in *Tables 2-2, 4-3* and *4-4*.

Hassayampa River, Gila River, and Centennial Wash

A topographic ridge between the plant site and the Hassayampa River, five miles east of the plant site, provides a natural barrier against site flooding that is approximately 33 ft above the river's PMF level. The nearest approach of the Gila River to the site six miles to the southeast, where the PMF stage is 175 ft below the lowest plant grade elevation of 951 ft at Unit 3. Centennial Wash is approximately five miles south of Unit 3, with a PMF level approximately 63 ft below the lowest plant grade (UFSAR Section 2.4.2.2.1). An evaluation of the PMF similar to that of East and Winters Washes determined that flood events on these watercourses do not reach the site (UFSAR Section 2.4.3).

### 2.2.3 Potential Dam Failures (Seismically Induced)

According to the UFSAR, the floodwater surface elevation due to dam failure does not adversely affect the plant. Using the cross-section data and inundation maps of the Salt, Verde and Agua Fria river systems, a floodwater surface elevation of 900 ft would accommodate a peak discharge of 7.6 million cfs at the selected point in the Gila River, 51 ft lower than the plant grade for Unit 3. Accordingly, a peak discharge of 7.6 million cfs resulting from domino-type failure of dams in the Gila River system upstream from the site with timing such that the peaks from each river arrive simultaneously at the point in the Gila River nearest to the plant site during a standard project flood has been determined to not impact the site. Wind-waves superimposed upon these water surface elevations will also not affect the site (UFSAR Section 2.4.4.3).

### 2.2.4 Probable Maximum Surge and Seiche Flooding, Probable Maximum Tsunami Flooding, and Ice Effects

Storm surge and seiche flooding, tsunami flooding, and ice effects were screened out as potential flooding mechanisms in the UFSAR:

- Probable maximum surge and seiche flooding (UFSAR Section 2.4.5)
- Probable maximum tsunami flooding (UFSAR Section 2.4.6)
- Ice effects (UFSAR Section 2.4.7)

### 2.2.5 Channel Diversion

The source of cooling water for PVNGS, including a source of makeup for the ESPs, is treated sewage effluent primarily from the city of Phoenix. The effluent is conveyed to the site through approximately 35 miles of pipeline and treated in the onsite water reclamation facility to meet plant water quality requirements. Onsite storage reservoirs provide for a continuous water supply in the event of scheduled or unscheduled interruptions or reductions in the normal water source (UFSAR Section 2.4.9).

Since the conveyance line, water reclamation plant, and reservoirs are not specifically designed against failure under extreme environmental conditions, the normal water source is subject to possible interruption. However, the ESPs are designed to provide storage of safety-related water necessary for safe shutdown, and the ponds will not be subject to loss of function due to any interruptions in the water source (UFSAR Section 2.4.9).

Therefore, channel diversion is not applicable in the current design basis from the perspective of interruption of cooling water supply.

### 2.2.6 Operating Water Surface Elevations

There are two cooling water makeup reservoirs at the site (*Figure 2-3*). These reservoirs are called the "45-Acre Reservoir" and the "85-Acre Reservoir" because at the normal operating capacity (water surface elevation at 951 ft), the associated surface areas are approximately 45 acres and 85 acres for the two reservoirs, respectively.

The pumps in the intake structure of each reservoir require a minimum water surface elevation of 922.5 ft for operation. The normal operating level in both reservoirs is 951 ft with a maximum operating level in both reservoirs of 952.5 ft to accommodate emergencies and power plant outages. Operational procedures provide for the control of water levels in the ponds so that they are only raised above 951 ft when there is no large storm in the weather forecast. A freeboard of 1.5 ft (between 951 ft and 952.5 ft) is provided to contain the 6-hour PMP and to accommodate occasional excess flows from the reclamation plant in emergencies. An additional minimum 2.5 ft of freeboard is provided to accommodate waves and run-up (UFSAR Section 2.4.8.2.2) so the minimum embankment elevation is 955 ft.

There are currently three evaporation ponds in service near the site southern boundary *(Figure 2-3)*. Pond No. 1, with a surface area of approximately 250 acres, was constructed initially to provide sufficient capacity for approximately four years from the startup of Unit 1. Pond No. 2, with a surface area of approximately 235 acres, was constructed in 1988, along the east side of Pond No. 1. Pond No. 2 was eventually divided with internal embankments into three segments; Pond 2A (117 acres), Pond 2B (87 acres) and Pond 2C (30 acres). In 2009, Pond No. 3, with a surface area of approximately 180 acres, was constructed as an earth embankment structure to the south of Pond No. 1, and is divided into two near-equal halves.

The maximum operating water surface elevation for all of the evaporation ponds is 937 ft. The maximum operating elevation provides 1.5 ft of freeboard above the normal operating level of 935.5 ft to allow for the 6-hour PMP and occasional plant wastewater discharge during startup. An additional minimum 5 ft of freeboard is provided to accommodate waves and run-up (UFSAR Section 2.4.8.2.3) such that the minimum embankment elevation is 942 ft for all three ponds.

The ESPs are operated with a maximum static water level of 1.1 ft below the top of the vertical walls, which is maintained by an overflow weir (UFSAR Figure 9.2-1). This arrangement provides adequate freeboard for high wind conditions i.e., waves generated within the ESPs could not spill out. The ESP walls are rated for full capacity (water levels up to the top of each wall), which accounts for hydrostatic loads and wave run-up for flood events.

## 2.3 Flood-Related Changes to the Licensing Basis

*Section 2.3* has been prepared in response to Item 1.a.iii of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter. Relevant site data to be considered include flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance.

### 2.3.1 Description of Hydrological Changes and Flood Elevations

Hydrologic changes since the initial license issuance with a potential to impact flood elevations at the site include changes in the rainfall-runoff response of the Winters Wash and East Wash watersheds due to natural geomorphologic processes and anthropogenic (i.e., man-made) changes.

Natural geomorphologic processes with the potential to change the rainfall-runoff response of the watersheds include severe erosion and channel down-cutting and migration. There has been no report of processes of this nature that would impact flood elevations at the site.

Anthropogenic forces with the potential to change the rainfall-runoff response of the watersheds include urbanization, road construction, and channelization. There has been no report of urban development with an aerial extent sufficient to change runoff in the Winters or East Wash watersheds. However, the construction of the Interstate 10 (I-10) embankment and associated drainage ditches and culverts more than six miles upstream of the site has potentially impacted drainage patterns and peak discharges associated with rainfall events. The flood hazard reevaluation includes the effects of I-10 (*Section 2.4.2*).

Channelization and realignment of East Wash associated with the construction of PVNGS had a beneficial impact on flood elevations at the site. These changes were incorporated in the hydrologic studies and flooding calculations associated with the original license application.

The design basis flood elevations for the flood-causing mechanisms that are applicable to the site are summarized in *Table 2-2*. A description of changes to flood-related protection implemented since license issuance is provided in *Sections 2.3.2* and *2.4.2*. Up-to-date information and data regarding topography, buildings, structures, and hydrologic controls were utilized in the flood hazard reevaluation analysis.

### 2.3.2 Description of Flood Protection Changes (Including Mitigation)

The flood protection system described in the UFSAR and observed and documented in the *Flooding Walkdown Report,* is specifically relevant to the flood hazard reevaluation analyses. Changes to the site layout and SSCs related to flood protection were noted as part of Recommendation 2.3, Flooding Walkdown.

Changes to flood protection related to site layout are discussed further in *Section 2.4.2*. Safety-related SSCs that are credited in the CLB with protection of the plant from external flood hazards were identified, inspected, and evaluated. Observations of nonconforming conditions were entered into the CAP.

## 2.4 Changes to the Watersheds and Local Site Area

*Section 2.4* has been prepared in response to Item 1.a.iv of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter. Relevant site data to be considered includes changes to the watershed and local area since license issuance. Descriptions of the watersheds at the time of license issuance and pertinent changes to the watersheds since license issuance are presented in the following two subsections.

### 2.4.1 Description of the Watersheds and Local Area at the Time of License Issuance

The site is located within a broad valley or basin surrounded by a series of low hills with a maximum relief of less than 250 ft. The average elevation of the basin floor is approximately 950 ft and the adjacent hills rise to about 1,200 ft elevation. The basin floor slopes to the south with a gradient of about 28 ft per mile and is dissected by a number of stream channels that converge and flow toward the Gila River, about 10 miles to the south. *Figure 2-4* provides an aerial view with the various rivers and watersheds annotated.

The site is bordered by Winters Wash on the west and East Wash on the east. Buckeye Salome Road is north of the site and runs in a northwest-southeast direction. A paved county road, Wintersburg Road, runs north-south along the west edge of the site, and Elliot Road (also referred to as Ward Road) runs east-west along the southern boundary of the site (UFSAR Figure 1.2-2).

A Union Pacific railroad line runs on a southwest-northeast alignment approximately two miles south of Elliot Road. A spur from that rail line heads north across Elliot Road and forms a peripheral ring around most of the site; thereby, providing rail access at a number of points within the powerblock, near the cooling towers, and other areas of the site.

### 2.4.2 Description of Changes to the Watersheds and Local Area since License Issuance

Changes at the site since the development of the original design basis include the addition of the 45-Acre Reservoir, construction of a vehicle barrier system (VBS) creating a SOCA boundary (*Figure 2-3*), and non-safety-related building expansion outside the Protected Area. In addition, Evaporation Ponds No. 2 and No. 3 were constructed in 1988 and 2009, respectively. These changes were accounted for in the flood hazard reevaluation.

Excavated soils from the 45-Acre Reservoir were initially placed in East Wash, east of the embankment. Later, this spoils pile was removed from East Wash and was placed in several locations around the 45-Acre and 85-Acre Reservoirs, where it would not affect flow of water in East Wash. The impact of this earthwork activity on site topography was accounted for in the flood hazard reevaluation.

A section of I-10 was constructed across Winters Wash and East Wash watersheds north of the site (*Figure 2-4*). The impact of the embankment and associated drainage ditches and culverts on flow patterns within the watersheds were accounted for in hydrologic modeling associated with the flood hazard reevaluation. The flood hazard reevaluation uses newer and higher resolution topographic data than the flooding evaluation documented in the UFSAR. The analysis in the UFSAR was based on USGS quadrangle maps which, due to the date of the survey and/or the low resolution, may not account for the presence of I-10. The higher resolution data used in the flood hazard reevaluation leads to a more detailed delineation of watershed boundaries. In the case of the East Wash watershed, the updated delineation indicates a larger watershed than was delineated for the UFSAR analysis (UFSAR Section 2.4.3).

South of the site, Elliot Road was paved to accommodate new industrial development south of the road. There have been some minor developments north of the site with an insignificant impact on watershed runoff or site drainage.

Changes in the Gila River watershed include replacement (i.e., submergence) of the Waddell Dam by construction of the New Waddell Dam, which was completed in 1994 (USBR, 2011a). Additionally, the Theodore Roosevelt Dam and the Bartlett Dam storages have been augmented since license issuance. The increased storage volume was accounted for in the screening of dam failure for the flood hazard reevaluation.

The impact of the changes on regional and site drainage have been taken into account in the hydrologic and hydraulic analysis performed in support of this flood hazard reevaluation report, as described in *Section 3.0*.

## 2.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

*Section 2.5* has been prepared in response to Item 1.a.v of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter. Relevant site data to be considered include CLB flood protection and pertinent flood mitigation features at the site.

The flood protection features credited in the CLB were identified in the *Flooding Walkdown Report* and include the East Wash embankment, the East Wash riprap, the Winters Wash embankment, Seismic Category I building exterior walls, basemats, roof drainage systems, the 45-Acre and 85-Acre Reservoir berms, drainage ditches, compacted fill near cooling towers, vaults, and site grading. Two embankment structures were designed to realign East Wash around the site. The north-facing embankment was constructed between two hills on the northern edge of the site, and completely diverts flood flows of East Wash from its old channel to the east to prevent flooding of the site proper. The eastern embankment extends south toward a point about even with the edge of the powerblock area and continues to divert the water in East Wash away from the site. Both embankments were included in the original site design to provide protection from the design-basis PMF flood with two feet of freeboard.

The ground elevation along the west side of the site was raised to limit the extent of PMF on the site. Approximately 10 feet of compacted fill was placed in the cooling tower areas, such that ground between the peripheral road and the powerblock areas is above the PMF levels (UFSAR Section 2.4.10).

## 2.6 Additional Site Details

*Section 2.6* has been prepared in response to Item 1.a.vi of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter. Relevant site data to be considered includes additional site details necessary to assess relevant flood hazards (i.e., bathymetry, walkdown results, etc.).

### 2.6.1 Bathymetry

Storage of floodwater in the Winters Wash and East Wash channels and floodplains has a significant impact on peak discharges and flood levels adjacent to the site. Detailed bathymetric data (i.e., channel topography) was obtained from recent topographic mapping, as discussed in *Section 2.1.2*, for hydrologic and hydraulic modeling executed in support of the flood hazard reevaluation.

### 2.6.2 Recommendation 2.3 Walkdown Results

Flood protection features that are credited in the CLB to protect the plant from external flood hazards were identified, inspected, and evaluated as reported in the *Flooding Walkdown Report*. The results of the walkdown observations were reviewed using site processes in accordance with NRC Inspection Manual Chapter 326 (NRC, 2014) and entered into the CAP.

Topographic mapping with aerial photography taken in 2013 captured the conditions of the site for incorporation in the flood hazard reevaluation.

### 2.6.3 Site Visits

Reevaluation team representatives have investigated the area outside the PVNGS property, including: hydraulic controls at some bridges and roads; the site layout; the East Wash embankment; and hydraulic structures along the entrance road. Additionally, photographs of flood mitigation features were taken and reviewed during the development of the flood hazard reevaluation analyses.

# 3.0 FLOOD HAZARD REEVALUATION ANALYSIS

*Section 3.0* has been prepared in response to Item 1.b of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter and provides the results of the flood hazard reevaluation for the site, addressing each applicable flood-causing mechanism based on present-day methodologies and regulatory guidance. The flood-causing mechanisms potentially impacting the site include local intense precipitation and site drainage, flooding in rivers and streams (including combined effects flooding (CEF) scenarios), dam breaches and failures, and channel migration or diversion. Storm surge and seiche, tsunami, and ice-induced flooding were screened out as credible sources of flooding at the site. The site-specific LIP and the PMP on the washes use the AWA study in lieu of HMR 49 for rainfall distribution. *Appendix C* provides the basis for using the AWA study for the flood hazard reevaluation.

## 3.1 Software Used

The following software was used to perform the flood hazard reevaluation analyses. The descriptions and capabilities of the software are provided in *Appendix D.*

- FLO-2D Pro 2012 (FLO-2D, 2012)
- FLO-2D Pro Release 14.03.07 (FLO-2D, 2014a)
- FLO-2D Pro Release 14.03.07.URS (FLO-2D, 2014b)
- ArcGIS 9.3.1 (ESRI, 2009)
- ArcGIS 10.1 (ESRI, 2012)
- ArcHydro 10.1 (ESRI, 2011)
- United States Army Corps of Engineers (USACE) HEC-HMS 3.5 (USACE, 2010a)
- USACE HEC-GeoHMS (USACE, 2009)
- USACE HEC-RAS 4.1 (USACE, 2010b)
- AWA PMP Evaluation Tool (AWA, 2013)

## 3.2 Flood-Causing Mechanisms

NUREG/CR-7046 recommends using an HHA method for evaluating the safety of SSCs. The HHA method is a progressively refined, stepwise estimation of site-specific hazards that starts with the most conservative plausible assumptions consistent with available data. The HHA process is used for each flood-causing mechanism to be reanalyzed. This method can be summarized as follows:

1. Develop a conservative estimate of the hydrologically relevant site-related parameters using simplifying assumptions for the flood-causing mechanism and estimate new flood elevations using the appropriate modeling approach.

2. Compare the reevaluated flood hazard elevation (from step 1) with the original design flood elevation for the selected flood-causing mechanism. If the newly calculated flood elevation is lower, it is used for comparison against the current design basis for the reevaluation of this causal mechanism.

3. If not lower, determine if the parameterization of site hydrology can be further refined. If yes, perform reevaluation (repeat steps 1, 2). If not, use the flood elevation from the previous step for this causal mechanism for comparison of reevaluation against the current design basis.

4. If all flood-causing mechanisms have not been addressed, select another flood-causing mechanism and proceed to step 1.

For each flood-causing mechanism, the final flood elevations from the hazard reevaluation were compared with the current design basis flood elevations to determine whether the current design basis flood bounds each reevaluated hazard.

The methodology described above was used to reevaluate the potential flooding effects resulting from each potential flood-causing mechanism relevant to the site using present-day methodologies and regulatory guidance. Details regarding the considerations and results of the analyses regarding each flood-causing mechanism are presented in the following subsections of this report.

### 3.2.1 Local Intense Precipitation

*Sections 3.2.1.1* through *3.2.1.3* address the effects of LIP at the site. A flow chart of the HHA screening methodology for the site overall LIP flooding analysis, based on guidance developed in NUREG/CR-7046, is presented in *Figure 3-1*.

#### 3.2.1.1 Local Intense Precipitation Hyetograph

A LIP hyetograph (graphical representation of rainfall over time) was developed as part of the flood hazard reevaluation to support analysis of the flooding effects associated with intense rainfall on the overall site drainage system. The most up-to-date PMP estimation methodology for the state of Arizona was applied to develop the LIP hyetograph. This methodology utilizes a PMP evaluation tool developed by AWA under the direction/funding of the ADWR, Arizona Game & Fish Department, FCDMC, Navajo County Flood Control District, National Resource and Conservation Service (NRCS), and the Federal Emergency Management Agency (FEMA). Refer to *Appendix C* for information about the AWA PMP evaluation tool for determining rainfall in Arizona.

The LIP hyetograph was developed for the SOCA (approximately 1 sq mi), which encompasses all safety-related SSCs at the site. The resulting cumulative 6-hour rainfall depth was 12.80 inches. This rainfall was distributed in 10-minute increments over a 6-hour period, following the methodology of the AWA PMP Evaluation Tool. The maximum rainfall in one hour within the LIP hyetograph was 10.73 inches. The incremental and cumulative distributions of the 6-hour PMP are shown in *Figure 3-2*.

#### 3.2.1.2 Effects of LIP

In accordance with the guidance presented in NUREG/CR-7046, the considerations addressed in the analysis of site overall flooding resulting from the LIP were:

- Depth of flooding
- Duration of flooding
- Maximum velocities
- Hydrodynamic and hydrostatic loads
- Sedimentation
- Debris loading

Each of these considerations was evaluated based on the results of two-dimensional flow modeling in FLO-2D to simulate runoff from the site. The output of the FLO-2D model includes water surface elevations, water depths, maximum water velocities, and the duration of flooding. FLO-2D also computes the hydrostatic and hydrodynamic forces that the floodwater could exert on obstacles (e.g., buildings) within flooded areas. These results from FLO-2D directly address requirements of NUREG/CR-7046. The potential for sedimentation and debris loading was qualitatively evaluated, based on the interpretation of FLO-2D output of depths, maximum velocities, and flow directions.

The FLO-2D model boundaries were established away from the powerblock area and safety-related SSCs in order to ensure the stability of the model. The domain of the FLO-2D model was developed to represent site conditions reflected in 2013 site aerial topographic mapping. The boundaries of the FLO-2D domain were primarily established along drainage divides (e.g., roads, berms, and embankments). The FLO-2D domain for the LIP analysis is shown in *Figure 3-3*.

Consistent with established FLO-2D methodology, boundary conditions include mechanisms through which water enters or leaves the model domain. These mechanisms include lateral outflow through the model boundaries, rainfall applied directly to the FLO-2D grid cells, and infiltration that removes water from the model domain. Lateral outflow conditions along all boundaries allow runoff to drain from the FLO-2D domain in a natural manner. The rainfall hyetograph discussed in *Section 3.2.1.1* was applied as direct rainfall in FLO-2D, and infiltration was characterized in the more refined cases for pervious areas surrounding the powerblock using the Green-Ampt method (FLO-2D, 2012). Site-specific soil properties and land use classifications were also used.

The FLO-2D model characterized topographic and man-made features that affect runoff from the site, including the VBS. As a modeling assumption, spaces between VBS blocks were assumed to be closed (i.e., water was not allowed to flow between adjacent blocks). This effect is intended to simulate obstruction by potential debris carried away from the powerblock due to runoff during the LIP simulation. Thus, any backwater effects of debris jams during the LIP event are accounted for in the FLO-2D model.

Additionally, buildings, tanks, and other structures were characterized within FLO-2D as flow obstructions.

The HHA methodology for evaluating LIP flooding is shown in *Figure 3-1.* It consists of iterative calculations starting with conservative modeling assumptions and progressively refining the inputs and assumptions. Four basic cases were developed for the whole site, and an additional simulation was conducted for sensitivity analysis. These cases are summarized as follows:

- Case 1 was a steady state simulation (i.e., constant rainfall intensity) with 25x25 ft grid cells, and assumed high Manning's roughness coefficients and no infiltration losses.
- Case 2 included infiltration losses and a time-varying LIP distribution was applied.
- Case 3 reflected the same characteristics as Case 2, but had a finer grid cell size of 15x15 ft.
- Case 4 included the 15x15 ft grid resolution and lower, more representative Manning's roughness coefficients. Also, the roof slopes were simulated within FLO-2D to more closely reflect the plant configuration.
- Case 5 was a sensitivity analysis to demonstrate the effect of varying the rainfall distribution.

A sensitivity study was performed to determine the effects of lower Manning's roughness coefficients. It was found that lowering the Manning's roughness coefficients below those used in Case 4 had negligible impact on the maximum transient water accumulation depths. Cases 1 through 4 applied the rainfall distribution recommended by the AWA PMP evaluation tool.

Case 4 was considered the most representative for the LIP event because of the refined grid size, Manning's roughness coefficients, and roof slopes and was used for the development of the flood hazard reevaluation. An inundation map developed with the output from Case 4 is provided in *Figure 3-3*.

#### 3.2.1.3 Sedimentation and Debris Loading Coincident with LIP

Sedimentation and debris loading during a LIP event were screened out qualitatively as hazards at the site based on the results of the LIP analysis. This screening was based on flow depths, flow velocities, and flow directions predicted by the FLO-2D model for the powerblock area. Flow depths and velocities near safety-related structures were generally small and did not constitute a credible hazard for erosion, sedimentation, or debris loading. Additionally, predicted flow directions were away from safety-related SSCs, which are surrounded by predominantly paved areas, precluding any impact on the SSCs from sedimentation or debris loading. While it is not expected that debris could impact safety-related structures, potential debris blockage is accounted for in the model by assuming the spaces between VBS blocks are closed.

### 3.2.1.4 Wind-waves and Run-up Coincident with LIP

Wave run-up is a process whereby wind-generated waves impinge on a structure or embankment and cause intermittent flow of water up the side of the structure or embankment. In general, the impact of wave action increases with the speed of the wind, the depth of the water over which it acts, and the length over which the wind blows (i.e., the "fetch").

The two-year, 10-minute overland MSW speed used at the site was calculated to be 39.35 mph. The sustained overland wind speed was converted to an overwater wind speed of 47.28 mph using procedures outlined in the USACE CEM (USACE, 2008).

Wave action coincident with the LIP analysis was evaluated for the various water bodies at the site as follows (*Figure 3-4*):

- 45-Acre and 85-Acre Reservoirs
- Evaporation ponds
- ESPs

The results of the wave run-up analysis indicated a maximum water level of 955.08 ft within the 45-Acre Reservoir due to run-up from wind-waves. This run-up level cannot cause spillover toward the powerblock area because the 45-Acre Reservoir is separated from the powerblock area by a ridge that has a minimum elevation of 961.0 ft. The wave run-up level computed for the 85-Acre Reservoir was 953.97 ft, which is 1.03 ft lower than the minimum reservoir embankment elevation of 955.0 ft. Consequently, wave run-up in the 85-Acre Reservoir during the LIP event does not affect water levels in the powerblock area.

The maximum run-up level computed for the evaporation ponds was 938.76 ft. This elevation is below the top of the surrounding berms (942 ft). Consequently, run-up in the evaporation ponds does not affect water levels in the powerblock.

Wave run-up was not computed for the ESPs. Any waves generated by wind would result in water spilling out from the ESPs onto the powerblock area. However, any potential spill of water from the ESPs during a LIP event was screened out as a hazard because site grading will direct the flow away from safety-related SSCs, as confirmed by the maximum LIP water depths modeled in FLO-2D.

Water levels within the SOCA were too shallow for significant wave development and there were many obstructions that disrupt fetches over the standing water. A fetch was developed for this area, but wave effects were screened out due to the short length of the fetch and the intervening obstructions. The longest potential fetch was defined for each water body listed above as shown in *Figure 3-4*.

The growth of wind-waves on the transient LIP runoff in the powerblock area was screened out as a flood hazard because of the relatively shallow depth of transient water accumulation and the limits on fetch lengths resulting from the number of tall structures inside the SOCA. All potential wave paths approaching safety-related SSCs were blocked by shallow water or transverse flows in drainage ditches. Consequently, wave action on the maximum water surface elevations experienced at safety-related SSCs has no effect.

#### 3.2.1.5 LIP Accumulation at Safety-Related SSCs

On-site LIP accumulation depths (Case 4) at entrances to safety-related structures were calculated and were found to be higher than the inlet elevations of some doors and hatches for limited durations. Potential pathways for water intrusion into buildings/structures through gaps in doors and hatches were evaluated for each unit. APS conducted an evaluation of the effects of these flood depths in the room-by-room internal flooding analysis described in the next section.

#### 3.2.1.6 Effects of LIP on Safety-Related SSCs

A room-by-room internal flooding analysis of the critical areas of the plant was performed to assess the potential impact to safe shutdown equipment when water from a LIP event enters buildings through door thresholds and gaps in hatches (pathways). These transient flood evaluations were performed utilizing methodology in the Design Basis Flooding Calculations per the Standard Review Plan, NUREG-0800, Section 3.6.1 (NRC, 1981).

Protection of essential equipment against the postulated water inflow through gaps in doors and hatches, and corresponding internal room flood levels is provided by plant design features such as compartmentalized train separation, curbs, pedestals, check valves, level sensors and drainage systems. There are typically two drain headers at the ground floor elevation of each building which discharge the inflow water to the sump in the lowest elevation of the building. Additional drainage for the ground floor elevation is provided by stairwells that communicate to the lower floors of the building and trenches or seismic gap cavities between adjacent buildings.

Based on the existing passive plant features and the room-by-room internal flooding analysis, it has been determined that there is no effect on safe shutdown equipment.

Methodology

The path water would take once it entered the buildings through gaps in doors and hatches was investigated by review of plant layout drawings, design basis internal flooding calculations and walk downs. These activities helped determine the water flow characteristics and the configuration of passive plant features, such as walls, curbs, doors, door transoms, equipment pedestals, penetrations, drains and check valves, that limit the effects of internal flooding. All of these compartment features that redirect or limit water flow are used to generate various simulations to determine the bounding water levels in the compartments where safe shutdown equipment is located.

The analyses of the internal flooding within the safety related structures of the powerblock were subdivided into the following sections to determine the water heights in the critical areas of the following buildings:

- Diesel Generator Building
- Control Building
- Auxiliary Building
- Fuel Building
- Main Steam Support System Building
- Essential Pipe Density Tunnel
- Condensate Storage Tank Tunnel
- Internal Flooding at the Perimeter Yard Areas and Hatches

The internal flooding design basis was initially prepared in 1979 and then further updated to demonstrate compliance with the GDC 2 requirements as well as the BTP ASB 3-1 requirements for pipe breaks outside containment.

The major assumptions of the internal flooding transient analysis consist of the following:

1. The amount of localized transient water accumulation around the powerblock buildings, due to a LIP (or PMP) event is based on the analysis as described in *Section 3.2.1.2.*

2. The FLO-2D computer program provides conservative results (flood time histories, hydrographs) because it does not credit the roofs, parapet walls, scuppers and gutters that hold up water and distribute it to specific locations, which has a lagging effect as well as inventory reduction, in the calculation of water accumulation in the areas adjacent to building pathways.

3. Inflow of water from outside into the building pathways through gaps in doors is assumed to not occur when the predicted depth of transient water accumulation is equal to or less than 0.6 inch. This criterion is based on plant operating experience where normal rain storm runoff typically results in no flooding at the ground level of the buildings.

4. Accumulation of water around the buildings in the powerblock starts approximately one to two hours after the beginning of the LIP event based on the hydrographs. Thus, the flood evaluations start one hour into the LIP event, considered as time zero in the simulation.

5. The water height inside the buildings was determined using optimal time steps to accurately model the hydrographs. For instance, the flood height inside the buildings is determined in small time increments of one or two minutes during the

increased initial inflow rate of the outdoor water accumulation and larger time intervals thereafter for the duration of the 6-hr LIP event. The inflow rate can be input as a constant rate in the source room or a combined variable rate from various pathways at discreet time steps into the event.

6. Since the outflow of water through drains, floors, and doors is dependent on the flood height in the room, the flood height for the previous time period was used to determine the outflow for the current time step.

7. Proper visualization of the water flow path is critical in establishing appropriate boundary conditions between compartments to provide a bounding flood depth for each critical compartment. Walkdowns of the modeled flooded areas were performed to confirm the water flow characteristics, and the configuration of passive plant features, such as walls, curbs, doors, door transoms, equipment pedestals, penetrations, drains and check valves, that limit the effects of internal flooding were properly accounted for in the model.

8. The drain systems were assumed to function at a reduced capacity of 75% to account for debris or blockage in the pipe.

9. The amount of drainage flow allowed was based on the amount of hydraulic head provided by water accumulation on the floor, the number of drains, and the capacity of the header serving the area and/or multiple floors in the building.

10. The flow through the drains was determined using Darcy's Formula taking into account the head and the total resistance coefficient of the drain pipe and fittings up to the discharge location at the sump.

11. The flow through the floor openings and door gaps was determined using the equation for a rectangular weir with a discharge flow coefficient ($C_d$) value of 0.6:

$$Q = (\tfrac{2}{3})C_d(L)(G)\sqrt{2g\Delta h}$$

This equation provides equivalent results to that of a sluice gate model when taking into account the free flow and submerged conditions of the hydraulic jump wave experienced across the rectangular opening of the door as the room is flooded. This limits the amount of flow into the room. For conservatism, the inflow of water through the door gaps was determined without considering the differential head across the door as the room was being flooded.

12. Where applicable and for conservatism, the transoms at the bottom of doors are credited to restrict flow out of the source room to maximize the water depth in the source room.

13. Where applicable, the seals and gaskets were credited in precluding ingression of water during a LIP event.

Margin Evaluation

The room-by-room internal flooding analysis provided a conservative and reasonable internal water level for each unit. A refined FLO-2D PRO (FLO-2D, 2014b) model was developed that accounts for roof rain inventory distribution and localized grade around the access doors. Use of this model yielded lower time duration of water accumulation and lower water levels, resulting in a significant reduction of the inflow into the SSCs based on the APS simulations (URS, 2014). The lower values for duration of water accumulation and for water level were used to show the existence of margin and were not used as input for the room-by-room internal flooding analysis.

Conclusion

Localized accumulation of water, adjacent to structures in the powerblock as a result of a LIP event does not impact safe shutdown equipment. No operator action is required as a result of the LIP event.

### 3.2.2 Flooding in Rivers and Streams

River flooding hazards at the site were evaluated using the HHA method presented in NUREG/CR-7046, which is shown schematically in *Figure 3-5*. The initial flood hazard reevaluation identified the following watercourses near the site for evaluation of flood hazards due to river flooding: the Hassayampa and Gila Rivers, and Centennial, East, and Winter Washes (*Figure 2-4*).

#### 3.2.2.1 Screening Out Watersheds, Rivers, and Washers

Flooding due to the PMF on Centennial Wash, Hassayampa River, and Gila River was evaluated using steady state HEC-RAS models. Each of these watercourses was evaluated to determine whether the PMF could cross watershed divides and reach the site.

Sufficient historic stream flow datasets from the USGS or any other source were not available for estimating PMF discharge rates for these streams with a sufficient level of confidence for screening purposes. Consequently, the PMF flow rates for the screening analysis were estimated from a regression equation relating PMF discharge to watershed area for locations in this region. The regression equation was developed based on data in USACE and United States Bureau of Reclamation (USBR) reports (USACE, 1957; USBR, 2011a, 2011b, 2012).

Maximum water surface elevations were obtained for each watercourse using steady state HEC-RAS models. These models indicated that the site was not susceptible to flooding associated with PMF along Centennial Wash, and the Hassayampa and Gila Rivers.

Luke Wash lies between East Wash and the main branch of the Hassayampa River (*Figure 2-4*) and was treated as part of the Hassayampa River watershed for this evaluation. This is a conservative approach that results in higher estimates of peak flows nearer to the East Wash watershed.

#### 3.2.2.2 PMP for East Wash and Winters Wash Watersheds

The PMP hyetographs for the East Wash and the Winters Wash watersheds were determined in the initial analysis using the AWA PMP Evaluation Tool, which provides PMP values for three different storm types: local storms (i.e., a thunderstorm), general winter storms, and tropical storms. The AWA PMP Evaluation Tool limits the applicability of local storms to relatively small areas (approximately 50 sq mi or less).

The tropical storm PMP event was identified as the PMP event with the most intense potential rainfall for Winters Wash. The duration of the PMP on Winters Wash was 72 hours, with a cumulative rainfall of 11.21 inches. The peak rainfall intensity was 4.16 inches in six hours, occurring 42 hours after the start of the rainfall event.

A local storm PMP was identified as the critical PMP for the East Wash watershed. The East Wash PMP has a six-hour duration, with a cumulative depth of 10.09 inches. The peak rainfall intensity was 2.11 inches in 10 minutes, occurring three hours after the start of the rainfall event. The hyetographs for the East and Winters Wash watershed PMPs are illustrated in *Figure 3-6*.

#### 3.2.2.3 PMF for Winters Wash Watershed

A hydrologic response model (HEC-HMS) was developed to determine the runoff rates for the PMP events in the Winters Wash watershed. The Winters Wash watershed was divided into thirteen sub-basins for the river flooding analysis (*Figure 3-7*).

A schematic of the HEC-HMS model is illustrated in *Figure 3-8*. The runoff rate outputs from the HEC-HMS model were used as input to the FLO-2D model used to compute the PMF flood levels at the site.

PMF hydrographs were calculated for the Winters Wash watershed using HEC-HMS models following the HHA process. Following the guidance in (FCDMC, 2011), S-graph unit hydrographs developed for watersheds of similar characteristics to Winters Wash were used to transform excess rainfall to runoff.

Rainfall losses were calculated using the Green-Ampt method, as recommended by the FCDMC. Varying levels of soil moisture conditions were also evaluated. *Table 3-1* provides the key hydrologic input parameters and the peak discharges for each model sub-basin.

The nonlinearity effects of the unit hydrograph process were reviewed using a 33% reduction in the lag time and a 5% increase in the peak of the unit hydrographs. As stated in NUREG/CR-7046, the recommended adjustments are a 5% to 20% increase for the peak discharge and a 33% reduction in the lag time.

The paper by Pilgrim and Cordery referenced in NUREG/CR-7046 indicates that the channel morphology will have an impact on the extent of non-linearity (P & C, 1993):

> *...drainage basins where the design flow is retained within channels that are formed or have small floodplains are likely to respond in a highly nonlinear manner. In drainage basins with large floodplains and vegetation or other obstructions within high banks and on overbank areas, average velocities are likely to remain fairly constant or even to decrease to some extent as flow rates increase.*

In other words, the peak discharge of the PMF for a watershed with large floodplains will tend to be less than the peak from a watershed with flow contained within well-defined channels. As indicated by modeling results for the flood hazard reevaluation, the PMF event within the Winters Wash watershed would not be contained within channel banks and the majority of runoff would flow in the floodplain area, which is vegetated with desert brush. Therefore, following Pilgrim and Cordery, the nonlinear effects were expected to be small, so a 5% increase in peak discharge was applied.

<u>Effects of PMF on Winters Wash Watershed</u>

The following considerations were addressed for the watershed PMF analysis per the guidance of NUREG/CR-7046:

- Depth of flooding
- Duration of flooding
- Maximum flood velocities
- Hydrodynamic and hydrostatic loads associated with flooding
- Sedimentation associated with flooding
- Debris loading associated with flooding

The depth and duration of flooding, maximum flood velocities, and hydraulic forces associated with the flooding at the site were determined for the flood hazard reevaluation using the FLO-2D flood routing program (FLO-2D, 2012). The model domain used for the PMF evaluation of Winters Wash is shown in *Figure 3-8*.

A diagram of the HHA approach for the analysis of flooding from rivers and streams is provided in *Figure 3-5*. It consists of iterative model runs starting with conservative assumptions and progressively refining the inputs and assumptions.

A series of simulations with increasingly refined inputs and a verification run with hydrographs from UFSAR Figure 2.4-11, were run to evaluate the effects of PMF flooding on the site. The FLO-2D cases considered are summarized in *Table 3-2*. Case 2 is the most representative simulation of the PMF in the Winters Wash. Flooding from Winters Wash was screened out as a flood hazard due to differences in topographic grade and the significant conveyance capacity of the Winters Wash floodplain.

Wind-waves and Run-up Coincident with PMF

The potential for flooding due to wind-wave run-up coincident with the PMF was analyzed for Winters Wash, the ESPs, the evaporation ponds, the 45-Acre and 85-Acre Reservoirs, and the powerblock area. The run-up was calculated by identifying critical fetches within areas of floodwater on the site and calculating wave run-up using the procedures described in the USACE CEM. The two-year MSW (overwater) of 47.28 mph was used for the wind-wave action coincident with the PMF.

The critical fetch length used for Winters Wash is shown in *Figure 3-9*. The run-up on Winters Wash was 0.37 ft. As indicated in the figure, the floodwaters from Winters Wash did not reach the powerblock area. The run-up was computed for Winters Wash at a point closest to the powerblock area (cross-section B). The final flood elevation for Winters Wash including run-up was 940.4 ft (FLO-2D Case 2), which does not reach the minimum grade elevation of the powerblock, which is 951.0 ft.

The effects of wave run-up in the ESPs and the evaporation ponds were bounded by the wind-wave effects considered in the LIP analysis (*Section 3.2.1.4*) because the precipitation depths and corresponding water levels were higher in the LIP analysis. The design of the 45-Acre and 85-Acre Reservoirs accommodates wave run-up. Run-up on the transient flows on the powerblock area was screened out as a flood hazard because of the numerous obstacles that prevent formation of substantial waves. Additionally, water levels in the powerblock area were lower for the river flooding analysis than for the LIP analysis because of lower precipitation rates. Consequently, any small waves that could form were bounded by the waves associated with the LIP analysis.

#### 3.2.2.4 PMF for East Wash Watershed

The initial flood hazard reevaluation hydrologic response model using HEC-HMS (USACE, 2010a) was developed to determine the runoff rates for the PMP events in the East Wash watershed. The East Wash watershed was divided into five sub-basins for the river flooding analysis (*Figure 3-7*). The HEC-HMS model was set up and calibrated to represent the East Wash watershed, including a relatively detailed representation of I-10 within the East Wash watershed.

Refined Analysis

A refined flood hazard reevaluation, in conformance with the HHA, was subsequently performed to evaluate the two-dimensional flow characteristic associated with the East Wash watershed. The analysis modeled predominant flow direction along the watercourse. A description of the data, assumptions, methodology, and results for each portion of the refined analysis from the calculations is discussed in the following sections.

The initial analysis included a number of calculations to reevaluate the PMP event. The first task in refining the flood hazard reevaluation at the site was to develop a strategic process for modifying the initial analysis. The refinements are in conformance with

NUREG/CR-7046 and the HHA process, which allows for progressively refining the methods that increasingly use site-specific data to demonstrate whether the plant SSCs important to safety are adequately protected from the adverse effects of severe floods. General areas considered for refinement were:

- Precipitation amount and distribution
- Topography
- Site characteristics
- Site-specific hydrologic and hydraulic analysis
- Contributing watershed hydrology and hydraulics
- Existing site protection
- Wind-wave action

APS and contract subject matter experts met with or spoke to the following agencies and firms as part of the assessment of these areas for refinement:

- Flood Control District of Maricopa County - FCDMC is involved in identifying, regulating, and remediating regional flood hazards. As such, they have developed or supported the development of comprehensive tools for determining flood hazards. Their support of the development of the refined methodologies for calculating site specific PMP events in Arizona and using two-dimensional flow models (FLO-2D) for evaluating arid watershed floods are considered important to this study. The FCDMC also recently sponsored a floodplain delineation study of East Wash where the 100-year hydrology was accepted by the FEMA.
- Arizona Department of Water Resources – Meetings were held with ADWR to discuss the use of extreme rainfall events. In 1980, the ADWR was created to secure long- term dependable water supplies for Arizona communities (ADWR, 2014). One function they perform to support this mission is regulating dam safety. In 2013, AWA prepared the report Probable Maximum Precipitation Study for Arizona (AWA, 2013) that developed a procedure for calculating PMP storms throughout the state. The results of the study were developed to replace the historical results from HMR 49.
- Applied Weather Associates – Meetings were held with AWA to discuss the applicability of using Arizona site specific PMP information.

The refined 100-foot grid element analysis is an enhancement to the HEC-HMS model because it more accurately defines the hydrologic and hydraulic performance of the East Wash watershed during the PMP event. The entire East Wash, from the northern boundary approximately seven miles north of I-10 to the evaporation ponds at cross-section XS7 (*Figure 3-10*), was included in the 100-foot grid element analysis.

The FLO-2D 100-foot grid element hydrograph (*Figure 3-15*) has two flow paths. The discharges at cross-sections XS4 and XS5 (*Figure 3-10* were used as inflows in the 25-

foot grid element model, which extended in East Wash from cross-sections XS4 and XS5 down to XS7 (area shown in *Figure 3-14*). High points along the east embankment are modeled as levees to reflect the highest height in the 25-foot grid element in determining overtopping.

Topography

Considerations for the East Wash refined model include:

- The crown of the roadways at I-10 was included in the refined analysis in the 100-foot grid element FLO-2D model to simulate the potential overtopping elevation during the PMP in the East Wash. The data was obtained from the Arizona Department of Transportation As-Built plans for the specific section of I-10 (ADOT, 1969).
- High resolution contour mapping data were obtained from the FCDMC for delineating the East Wash watershed. The datasets used were the Palo Verde Mapping, 2-ft contours from June 2007, and Luke Wash and Arlington Mapping, 2-ft contours from September 2005 (FCDMC, 2011).
- Aerial mapping flown for APS on March 25, 2009, was used in the refined analysis (URS, 2013). The topographic survey information was only used where ground elevations and other sources of data were not available.

Watershed Characteristics

The HEC-HMS unit hydrograph rainfall runoff approach developed for the initial analysis of East Wash PMF divided the watershed into only five sub-basins. To account for the unit hydrograph approach, nonlinearity was accounted for by reducing lag time and increasing the peak discharge as required in Appendix I of NUREG/CR-7046. The HEC-HMS model has some inherent issues when modeling a watershed's hydrologic response in Maricopa County. Presently, the FCDMC has coordinated with the USACE to modify the program for use in Maricopa County. The USACE is modifying HEC-HMS for Maricopa County in the following areas:

- Point-area rainfall area reduction (similar to HEC-1 JD cards)
- Green-Ampt method with GIS (land use and soil shape file)
- Clark unit hydrograph (FCDMC time of concentration method)
- Normal depth channel routing
- Efficient handling and management of large models with hundreds of sub-basins

FCDMC uses either HEC-1 or FLO-2D to model both rural and urban watersheds. Selection of the model being used depends on the detail needed for the analysis and the project budget. FLO-2D is a more accurate model for simulating overland flow patterns in areas such as East Wash because the washes in the upper watershed, including the area north of I-10, are small and intermittent.

For the refined flood hazard reevaluation of East Wash, a significant portion of the flow occurs in the floodplain adjacent to the wash, especially for less frequent flood events including the PMF. When Entellus prepared a flood delineation study of East Wash, the FLO-2D model was still being tested by the FCDMC and HEC-1 was the model selected for the study. Since then the FCDMC has worked with Riada Engineering, Inc. (REI) in updating the FLO-2D model. The FCDMC has used the model on many of their recent watershed studies.

Use of a two-dimensional flow model (100-ft grid elements) provides a more representative simulation of hydrologic and hydraulic parameters for predicting peak discharges and water surface elevations in the watershed for severe rainfall events than a one-dimensional model such as HEC-HMS or HEC-1. No adjustments are needed for nonlinearity because FLO-2D is a physically based two-dimensional rainfall runoff model that explicitly accounts for the hydrologic effects that contribute to surface runoff generation. The refined analysis of East Wash utilizes a 100-ft grid element for benchmarking with the FEMA-accepted 100-year flood, a 100-ft grid element PMP model and, near the site, a 25-ft grid element to refine the analysis when estimating the water surface elevation at the east embankment.

Floodplain Cross-Sections

Floodplain cross-sections were added to the East Wash FLO-2D model to determine the peak flow rates at several locations. Five locations were chosen along the watershed:

- XS1 - north of I-10
- XS2 - south of I-10
- XS3 - north of PVNGS, at upper boundary of the initial model, and at the same location as HEC-HMS junction EJ16
- XS6 - at Water Reclamation Access Road, and at the same location as the initial HEC-HMS junction EJ11
- XS7 - south boundary of model
- Two more cross-sections, XS4 and XS5, were added at the XS3 location to quantify the flow going across the two primary flow paths in the East Wash at that point. *Figure 3-10* shows the locations of the seven floodplain cross-sections.

The discharges at cross-sections XS4 and XS5 were used as the East Wash inflows in the 25-ft grid element model.

Results for PMF in East Wash Without Wind Effects

The hydrographs for floodplain cross-sections XS1 through XS5 are shown in *Figure 3-11*. The results of the model show that the floodwaters from the PMP storm do not breach the East Wash or the north-facing embankments around the site. *Figures 3-12* and *3-16* show the flow depths near the site, and *Figure 3-13* shows velocity vectors representing the flow velocities and directions. Floodwaters shown inside the site are the result of direct rainfall onto the site only, since no floodwaters breach the embankments. *Table 3-3* summarizes the results at the floodplain cross-sections.

There were six hydraulic cross-sections evaluated as shown on *Figure 3-14*. Floodplain cross-sections X1, X2, and X3 correspond to locations defined in the initial analysis, and A1, B, and C are locations from the UFSAR. Placing floodplain cross-sections at these locations provides discharges and flood elevations that are easily extracted from the model. The model results for the six hydraulic cross-sections are shown in *Table 3-4*, along with the embankment elevations and calculated embankment freeboard.

<u>Wind-Wave Action Coincident with PMF</u>

The fetch lengths were re-calculated for the refined PMF analysis to determine the change in fetch length and wave height. The critical 2-year wave heights were calculated using the information from the initial analysis and the reduced fetch length for the lower water surface elevations. The results show that when the predicted wave heights are added to the PMF water surface elevation, the north and east embankments are not overtopped.

<u>Fetch Selection</u>

The refined analysis analyzed eight potential fetches, five at the north embankment and three at the east embankment, to select the worst case scenario fetches for this wind-wave analysis. The potential fetches were selected where deep water is adjacent to the embankments and at multiple directions that follow the deepest backwaters. They are shown in *Figure 3-17*.

The maximum water depth is used in the fetch selection process and will yield a conservative selection of the fetches used in the analysis. Potential fetches N-2, N-3, N-4, and E-1 were not selected because their extents reach into flow channels at lengths that do not accurately capture the backwater effect that contributes to the wind-wave generation and propagation as can be visually verified on *Figure 3-17*.

Sections were taken at potential fetches N-1, N-5, E-2, and E-3 and the maximum water depths were tabulated in spreadsheets to determine the fetch lengths contributing to wind-wave propagation based upon the same assumption as in the initial analysis, i.e., areas with water depth less than 0.5 foot are not considered while computing fetch lengths. Fetch N-5, referred to as north embankment fetch henceforth, and E-2, referred to as east embankment fetch are selected as the worst case scenario wind-wave propagating fetches for analysis. *Figure 3-18* shows the north fetch and east fetch in plan view, and *Figures 3-19* and *3-20* show the sectional views for the north and east fetches, respectively.

Wind-wave Analysis

The procedures used to determine the impacts of wind-wave actions and wave run-up on the embankments are similar to the procedures in the CEM. A 2-year return period MSW speed and maximum over water fetch is used to calculate the maximum wave height, and the same wind speed is used to calculate run-up at the embankments that could cause spillover.

Refined Flood Hazard Reevaluation Results for East Wash

*Tables 3-6* and *3-7* summarize the results of the calculation. The refined analysis determined that the static freeboard available at the north embankment is approximately 3.60 ft and at the east embankment is approximately 3.10 ft. The refined analysis also shows that the wave run-up freeboard available at the north embankment is approximately 2.09 ft and at the east embankment is approximately 1.72 ft. Neither embankment is overtopped by wave run-up during the PMF event.

## 3.3 Combined Effects Flooding

This section has been prepared in response to the Request for Information Item 1.b of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (NRC, 2012a). NUREG/CR-7046 provides guidance for the CEF analysis. This type of analysis was not considered or required during the initial licensing of PVNGS.

The CEF analyses are based on the initial flood hazard reevaluation. The East Wash evaluation also considered the refined analysis to establish that the embankment is not overtopped when subjected to the PMF in combination with wind wave action.

Both American Nuclear Society (ANS), ANSI/ANS-2.8-1992 (ANS, 1992), and NUREG/CR-7046 indicate that isolated flood-causing events are not adequate as a design basis for power reactors. Consequently, it is appropriate to postulate critical combinations of flood-causing events when reevaluating the flood hazard at the site. Characteristics of the combined effects addressed for this flood hazard reevaluation include:

- Depth of flooding
- Duration of flooding
- Maximum flood velocities
- Hydrodynamic and hydrostatic loads associated with flooding
- Sedimentation associated with flooding
- Debris loading associated with flooding

In accordance with NUREG/CR-7046 and ANSI/ANS-2.8-1992, combined effects flooding alternatives for the site included floods caused by precipitation events and floods caused by seismic dam failures. A schematic diagram of the HHA methodology for CEF is provided in *Figure 3-21*. The CEF alternatives considered for the site were as follows:

| | Alternative I | Alternative II | Alternative III |
|---|---|---|---|
| Precipitation Floods | Mean monthly (base) flow | Mean monthly (base) flow | Mean monthly (base) flow |
| | Median soil moisture | Probable maximum snowpack | 100-yr snowpack |
| | Antecedent (or subsequent) 40% of PMP or 500-yr rain, whichever is less | Coincident 100-yr snow season rain | Coincident snow season PMP |
| | 2-yr wind speed in the critical direction | 2-yr wind speed in the critical direction | 2-yr wind speed in the critical direction |
| | PMP | N/A | N/A |
| Seismic Dam Failures | 25-yr flood | 50% of PMP or 500-yr flood, whichever is less | N/A |
| | Dam failure caused by SSE coincident with peak of flood | Dam failure caused by OBE coincident with peak of flood | N/A |
| | 2-yr wind speed in the critical direction | 2-yr wind speed in the critical direction | N/A |

The effect of snow as part of the CEF analysis was screened out for this desert rangeland area. Additionally, the proposed dam failure scenarios were screened out as less conservative than the dam failure analysis completed in *Section 3.4*.

### 3.3.1 Characterization of Combined Effects Flooding

The remaining precipitation flood alternative that required investigation for the combined effects flooding analysis was Alternative I proposed for precipitation floods in ANSI/ANS-2.8-1992. Each of the effects listed for Alternative was addressed as follows:

> <u>Mean monthly base flow</u> - base flow was screened out as not applicable, since East Wash and Winters Wash are intermittent streams and their base flows are equal to zero.
>
> <u>Median soil moisture</u> - the soil moisture classification of "dry" was used as the median soil moisture as defined by the FCDMC in the *Drainage Design Manual for Maricopa County* (FCDMC, 2011).This classification was appropriate for the arid desert lands surrounding the site.
>
> <u>Antecedent or subsequent rainfall</u> - the 500-year return period rainfall events were computed for both the East Wash and the Winters Wash watersheds. However, for both watersheds, the 40% PMP was selected over the 500-year

rainfall depth. Consequently, 40% of the PMP was used as the antecedent storm for both the East Wash and the Winters Wash watersheds. Consistent with the guidance in ANSI/ANS-2.8-1992, the storms in Winters Wash were spaced with three days between peak rainfall intensities, and the storms in East Wash were spaced with one day (24 hours) between peak rainfall intensities.

PMP - the PMP hyetographs for the East Wash and the Winters Wash watersheds were developed as part of the PMF analysis (*Section 3.2.2.2, Figure 3-6*). The PMP rainfall along with antecedent rainfall was used as input for the FLO-2D model. The CEF simulations with the FLO-2D models included the PMP distribution both on the powerblock and the washes concurrently to account for the contribution to runoff from each source.

Wind-waves - the effect of wind-waves was included in the CEF using procedures similar to those described in *Section 3.2.2.4*. Note that for the East Wash credit is taken for the refined analysis as described in *Section 3.2.2.4* which results in lower wind wave action heights.

To simulate the CEF at the site, separate FLO-2D models were developed for four different watershed regions surrounding the site (*Figure 3-22*). Two domains were used to simulate runoff from the East Wash (EW) watershed and two were used to simulate the Winters Wash (WW) watershed, as follows:

EW-North - this domain characterized runoff from East Wash upstream of the site. Runoff from this domain was used as an inflow boundary condition for simulations on the EW-South domain.

EW-South - this domain characterized the flood levels near the site on East Wash.

WW-North - this domain characterized runoff from Winters Wash upstream of the site. Runoff from this domain was used as an inflow boundary condition for simulations on the WW-South domain.

WW-South - this domain characterized the flood levels near the site on Winters Wash.

A total of 12 FLO-2D simulations (*Table 3-8*) were developed for the CEF analysis. Six simulations (W1 through W3 and E1 through E3) were conducted on the northern watershed domains (EW-North and WW-North; *Figure 3-22*). Simulations W1 through W3 evaluated runoff from the Winters Wash watershed. Each simulation applied different Manning's roughness coefficients as a sensitivity analysis. Similarly, the simulations E1 through E3 evaluated runoff from the East Wash watershed, with a range of Manning's roughness coefficients. Simulations E1 through E3 included a representation of I-10 in East Wash to account for the flow through the large box culverts and over the highway as a weir.

Six FLO-2D simulations (Cases 1 through 6; *Table 3-8*) were also conducted on the southern watershed domains (EW-South and WW-South; *Figure 3-22*). Case 1 applied an inflow hydrograph from the WW-North domain to simulate flooding in Winters Wash.

Cases 2 through 6 evaluated flooding in East Wash near the site using a finer grid cell size of 25x25 ft. Cases 2 through 4 apply a range of Manning's roughness coefficients as a sensitivity analysis. Of Cases 2 through 4, Case 3 was the most representative of the site. Case 5 accounted for failure of the East Wash embankment and removal of the VBS, a simulation developed because wind-waves associated with Case 3 without embankment failure indicated overtopping of the East Wash embankment. Case 6 was a sensitivity simulation to demonstrate the effect of completely blocking the culverts under the Water Reclamation Access Road. Given the results of the refined analysis where it is shown that the embankments are not overtopped, Case 5 is not applicable but is included for completeness.

### 3.3.2 Combined Effects Flooding Results

The modeling for the CEF analysis, which utilizes a two-dimensional model of the entire watersheds in lieu of the hybrid one-dimensional and two-dimensional models (Rizzo, 2014), was a continuation (per the HHA approach) of the river flooding analysis.

<u>Winters Wash</u> - CEF modeling for Winters Wash (Case 2) indicated that flood levels were bounded by, but were similar to, the flood levels for the reevaluated PMF river flood model (*Section 3.2.2.3*). The slightly reduced water levels are the result of a more refined modeling approach (i.e., applying FLO-2D to simulate runoff from the watersheds instead of HEC-HMS).

<u>East Wash</u> - the results from Case 3 (*Table 3-8*) indicated that the freeboard between the maximum still water level and the East Wash embankment crest at a point immediately north of the Water Reclamation Facility Access Road is 1.48 ft. The transient water accumulation observed in the powerblock shown in *Figure 3-23* results from rainfall directly on the powerblock, not from river flooding. The effects reported for the powerblock area, including the water depth, duration of flooding (*Figure 3-24*), maximum velocities, as well as hydrostatic and hydrodynamic forces, were bounded by the levels reported in the reevaluated LIP analysis (*Section 3.2.1*).

In the initial analysis, water levels were analyzed in the ESPs, 45-Acre and 85-Acre Reservoirs, and evaporation ponds, with the following results:

- The ESPs filled and overflowed due to the rainfall associated with the CEF considered.
- The 45-Acre and 85-Acre Reservoirs were submerged by floodwater for the East Wash embankment failure scenario. Consequently, water levels were not computed for these impoundments.
- The water level in the evaporation ponds due to the CEF was approximately 937.0 ft. This water elevation does not overtop the berms of the evaporation ponds at elevation 942 ft

The Case 6 sensitivity simulation that evaluated the effect of completely blocking the culverts under the Water Reclamation Access Road indicated a negligible effect on water surface elevations within East Wash.

The CEF analysis determined that the static water level for Case 3 did not overtop the East Wash embankment. However, the initial flood hazard reevaluation of wind-wave effects showed potential embankment overtopping at cross-section C, south of Unit 3 (*Figure 3-14*) and cross-section X2 at the Water Reclamation Facility road.

The refined flood hazard reevaluation determined that the PMF levels in the East Wash were lower than those calculated in the initial analysis and critical wind-wave run-up was 1.38 ft at cross-section X2 rather than 2.0 ft. The refined analysis determined that the PMF level plus wind-wave effects allowed a freeboard of 1.72 ft at X2 (*Figure 3-16*) and no overtopping was postulated along the East Wash embankment.

Based on the above discussion, it is appropriate to use the results of CEF Case 3 and then add the wind wave action from the refined flood hazard reevaluation.

Using the above approach, the available static freeboard from CEF Case 3 at cross-section X2 is 1.48 ft and, when subtracting the wind wave height of 1.38 ft from the refined model, this demonstrates that the embankment is not overtopped.

Also at cross-section C (*Figure 3-16*) the available static freeboard is 1.04 ft. The fetch at this location in East Wash south of the Unit 3 ESPs is significantly shorter (less than 0.25 mile) than at the north boundary. This results in a wind-wave height that is smaller than the available freeboard, thus no overtopping of the embankment occurs. If overtopping at cross-section C were to occur, there would be no detrimental effect on the powerblocks given its location along the very southern edge of Unit 3.

### 3.3.3 Wind waves and Run-up coincident with Combined Effects Flooding

Wind-waves were evaluated for the following locations:

- Evaporation Ponds - the final run-up level was 939.06 ft, which is below the berm elevation of 942 ft.
- ESPs - the ESPs were filled completely so any waves would cause spill-over from the ESPs, which would drain away from the ESPs and not affect water levels near other safety-related SSCs.
- Powerblock area - wind-wave effects on the powerblock area were screened out due to the shallow water and intervening obstacles that reduced potential fetches and prevented waves from reaching safety-related SSCs. Additionally, water levels in the powerblock area were lower for the CEF analysis than for the LIP analysis because of lower precipitation rates. Consequently, any small waves that could form were bounded by the waves associated with the LIP analysis (*Section 3.2.1.4*).
- Winters Wash - the final run-up level for Winters Wash was 940.4 ft, which did not approach the powerblock.
- East Wash - the East Wash wind-wave effects for CEF are considered to be equivalent to the wind-wave effects described for PMF in *Section 3.2.2.4*.

## 3.4 Dam Breaches and Failures

The potential flooding of the site due to dam breaches and failures was evaluated using the method outlined in ISG-2013-01 (NRC, 2013a). A flowchart of the method is provided in *Figure 3-25*.

The Volume Method, which is shown schematically in *Figure 3-26*, consists of calculating the theoretical flood elevation that would be obtained by combining the predicted flood elevation from the failure of all upstream dams with the 500-year flood on the main watercourse starting from a cross-section located as close to the site as possible. This is a conservative assessment and represents a condition with all upstream dams breached simultaneously and translated to a point near the site without attenuation. The dam break hazard can be screened out if the results of the Volume Method show that the flood level does not reach the site grade.

The locations and storage volumes for each dam upstream of the site were obtained from the USACE National Inventory of Dams (USACE, 2013). The locations of these dams are shown in *Figure 3-27*. In addition to the dams in the inventory, the volumes of the on-site reservoirs were considered. The total storage volume for all of the off-site dams and the Evaporation Ponds is 7,897,049 acre-ft. The addition of 3,928 acre-ft to account for the combined storage of the 45-Acre and 85-Acre Reservoirs gives a total volume of 7,900,977 acre-ft of water to be superimposed on top of the antecedent water surface elevation based on a 500-year flood.

The antecedent water surface profile was derived in HEC-RAS for a PMF discharge of 730,000 cfs (USACE, 1957), which bounds the 500-year flood discharge rate. Based on an analysis of digital topographic data in ArcGIS, it was determined that the flood elevation associated with the 7,900,977 acre-ft storage volume superimposed on the antecedent water surface profile did not rise above elevation 935 ft. Because this is 16 ft below the minimum site grade elevation of 951 ft, the dam-break mechanism, including aspects related to potential debris and sediment loads, was screened out as a flood hazard.

Based on the approximately 16-ft difference in elevation between the maximum water surface derived from the Volume Method and the minimum site grade elevation, it was concluded that wind-waves associated with the two-year return period wind speed could not reach the site because maximum wave heights were no greater than three feet.

## 3.5 Storm Surge

The site lies more than 1,000 miles from both the Atlantic Ocean and Gulf of Mexico and is located approximately 258 miles inland from the Pacific coast *(Figure 1-1)*. Therefore, the site is located a sufficient distance inland from these water bodies to screen hurricane storm surge out as a potential flooding hazard so that the more detailed considerations contained in NUREG/CR-7134 (NRC, 2012b) were not applicable.

The site is also approximately 134 miles inland from the Gulf of California (*Figure 1-1*). While Pacific hurricane surges can be transmitted up into the gulf to some degree (ANS, 1992), and spring tides as high as 10 meters (approximately 33 ft) have been reported (Filloux, 1973), the potential for flooding due to hurricane surge from the Gulf of California was screened out because of the significant topographic barriers between the shore and the site, which is located approximately 920 ft above the highest high tide elevation at the head of the Gulf of California.

Because safety-related SSCs at the site were not affected from storm surge flood levels from any water body, the evaluations of hydrostatic and hydrodynamic forces, debris, and water-borne projectiles, and the effects of sediment erosion or deposition due to storm surge laid out in JLD-ISG-2012-06 (NRC, 2013b) were not necessary.

## 3.6 Seiche

A seiche is defined as an oscillation of the water surface in an enclosed or semi-enclosed body of water initiated by an external cause in NUREG/CR-7046. Where the amplitude of seiche action is sufficiently large, the areas adjacent to the effected water bodies are at risk of flooding.

Following NRC guidelines detailed in NUREG/CR-7046, the risk of flooding at the site due to seiche-motion was evaluated for seismic effects, free oscillation of the water body due to meteorological effects, and landslides.

The site is not located near any coastlines or large bodies of water from which flooding due to seiches can occur. Water bodies with a theoretical potential for seiche-induced flooding at the site include the reservoirs, evaporation ponds and ESPs.

The Arizona Geological Survey (AGS, 2000) indicates that the site is located in an area of low seismic hazards, so there is a minimal risk of seismic activity. However, if seismic motion, barometric effects, or landslides along the embankments adjacent to the reservoirs were to cause a seiche large enough to displace water from the 45-Acre Reservoir or the 85-Acre Reservoir, an evaluation of the 2013 aerial topographic mapping of the site indicates that any water spilling from the reservoirs would flow south and away from the powerblock area, following the natural topography and site grading. Thus, seiche action in the reservoirs was screened out as a potential source of flooding to the SSCs.

Due to the small fetch across the ESPs, any induced surge and waves would be relatively minor. However, any water originating from the ESPs would drain away from the powerblock following site gradients.

The evaporation ponds are south and downstream of the powerblock. Any water leaving the evaporation ponds would be conveyed to the Gila River and away from the powerblock.

Based on the evaluation of topography summarized above, potential seiches on the reservoirs, evaporation ponds, and ESPs at the site due to meteorological effects, seismic effects, or landslides were screened out as a potential source of flooding to SSCs.

### 3.7 Tsunami

A review of historic tsunamis impacting the west coast of the United States and Mexico was undertaken following the guidance of NUREG/CR-6966 (NRC, 2008). The maximum water level due to historic tsunami run-up recorded along the west coast of the United States and/or Mexico was 39.4 ft at Newport Beach, California, in 1934 (NOAA, 2013), approximately 292 miles from the site. Based on the distance, intervening topographic features and differences in elevation (the minimum grade level of safety related SSCs is 951 ft, over 910 ft above the maximum recorded tsunami run-up elevation), it was concluded that tsunami run-up from Pacific coast events cannot reach the site and, therefore, tsunami flooding at the site was screened out.

### 3.8 Ice-Induced Flooding

The risk of ice-induced flooding which could adversely impact safety-related structures at the site was assessed in accordance with the applicable guidelines of the NRC. Ice-induced flooding analysis for the site included an assessment of ice jams, frazil ice, and ice thickness. According to guidance in NUREG CR-7046, ice-induced flooding was only considered in the context of whether a collapse of an ice jam can cause water to propagate to the site and whether an ice jam can cause flooding via backwater effects. The analysis screened out ice-induced flooding at the site based on historical ice jam records and meteorological data.

### 3.9 Flooding Resulting from Channel Migration or Diversion

The reevaluation of flood hazards at the site included an evaluation of the potential for site flooding resulting from channel migration or diversion upstream and downstream of the site. The rivers and washes in the vicinity of the site are the Hassayampa and Gila River, and Winters, Centennial, and East Washes, as shown in *Figure 2-4*. A qualitative assessment of these watercourses, based on an evaluation of local and regional topography, current and future land use, and seismic, geological, and thermal (e.g., volcanic) processes in the region, determined that the possibility of channel migration causing a flood hazard to safety-related SSCs at the site was negligible.

# 4.0 COMPARISON OF CURRENT AND REEVALUATED PREDICTED FLOOD LEVELS

*Section 4.0* has been prepared in response to Item 1.c. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter. Item 1.c. requires a comparison of current and reevaluated flood causing mechanisms at the site, an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism, and how the findings from Enclosure 4 of the letter (i.e., Recommendation 2.3 flooding walkdowns) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, justification should be included for how this finding was determined.

## 4.1 Comparison of Current and Reevaluated Flood-causing Mechanisms

The flood-causing mechanisms evaluated under the current design basis were:

- Effects of local intense precipitation (UFSAR Section 2.4.2.3)
- PMF on rivers and streams (UFSAR Section 2.4.3) including coincident wind-wave activity (UFSAR Section 2.4.3.6)
- Potential dam failures (seismically induced) (UFSAR Section 2.4.4)
- Probable maximum surge and seiche flooding (UFSAR Section 2.4.5)
- Probable maximum tsunami flooding (UFSAR Section 2.4.6)
- Ice effects (UFSAR Section 2.4.7)
- Channel diversions (UFSAR Section 2.4.9).

The flood hazard reevaluation includes the same flooding mechanisms as presented in the current design basis with some differences between the terminology in current NRC guidance and the UFSAR. Additionally, CEF has been evaluated as part of the reevaluated flood hazard.

The conditions for which the flooding analyses were performed and the methods used to perform the analyses varied between CLB analyses and the reevaluation analyses. The differences are summarized in *Tables 4-1* and *4-2.*

## 4.2 Assessment of Differences between Current Design Basis and Reevaluated Flood Elevations and Effects

A comparison of CLB and reevaluated flood levels and effects at the site for each flood mechanism is provided in the following subsections of this report. A summary comparison is provided in *Tables 4-3* and *4-4.*

### 4.2.1 Local Intense Precipitation Flooding

For the current design basis, the maximum calculated water levels near the safety-related structures due to the LIP event are two feet below the plant floor elevations at each unit. As a result, the current design basis does not include hydrostatic or hydrodynamic forces associated with flooding at safety-related SSCs.

The initial flood hazard reevaluation for sitewide inundation determined that water around the powerblock runs off during the LIP to the peripheral drainage system with some accumulation in localized areas approximately 1.0 to 1.75 ft. below the plant floor elevation at each unit *(Tables 4-3* and *4-4*). The areas of water accumulation are limited in size and depth and are primarily due to localized grade depressions as depicted in *Figures 3-3, 3-23* and *3-24*.

The 1-hour, 1 sq mi LIP depth is 11.8 inches, with a cumulative 6-hour rainfall of 15.53 inches. The cumulative 6-hour rainfall depth for the LIP reevaluation analysis obtained using the AWA PMP evaluation tool was 12.80 inches, with a 1-hour rainfall depth of 10.73 inches.

The current licensing bases presented in the UFSAR recognizes that some transient water accumulation could occur. The onsite drainage system is designed such that runoff due to PMP will not inundate safety-related structures, equipment, and access to those facilities. Areas adjacent to the powerblock are sloped away at 0.5% to 1%, resulting in a minimum drop of 5 to 7 feet at the peripheral drainage system (UFSAR Section 2.4.2.3).

The initial flood hazard reevaluation for LIP identified transient localized water accumulation adjacent to the powerblock structures, which could result in water ingress into the structures. This was addressed by the room-by-room internal flooding analysis, which determined that there was no impact to safe shutdown equipment. No operator action is required as a result of the LIP event.

The room-by-room internal flooding analysis provided a conservative and reasonable internal water level for each unit. A refined FLO-2D PRO (FLO-2D, 2014b) model was developed to account for roof rain inventory distribution and localized grade around the access doors was developed. Use of this model yielded lower time duration of water accumulation and lower water levels resulting in a significant reduction of the inflow into the SSCs based on APS simulations (URS, 2014).

The potential for sedimentation and debris loading on safety-related SSCs due to LIP was screened out qualitatively in the reevaluation analysis because of the low flood flow velocities. In addition, the flood flows were not in directions that would carry sediment from any potential sediment source into the powerblock area. Similarly, debris loading was screened out as a potential hazard for the site, because flows were shallow and could not carry larger debris into the powerblock area.

The effect of wave action on the maximum water surface elevations experienced at safety-related SSCs during the reevaluation LIP event was determined to be negligible.

### 4.2.2 Flooding in Rivers and Streams

Floodwater elevations were analyzed in terms of the impact of backwater from riverine flooding (i.e., "stillwater" or "static" flood levels) and the superposition of wave action as discussed below.

PMP Distributions

The current design basis PMP for the Winters Wash watershed was the 24-hour PMP of 14.6 inches (UFSAR Table 2.4-10) with a peak rainfall intensity of 5.20 inches in 1.15 hours. The reevaluated PMP for the Winters Wash watershed was determined using the AWA PMP evaluation tool. The PMP for the Winters Wash watershed was a 72-hour tropical storm PMP of 11.21 inches with an associated peak rainfall intensity of 4.16 inches in 6 hours.

For the East Wash watershed, the current design basis 6-hour PMP of 14.44 inches (UFSAR Table 2.4-11) with a peak rainfall intensity of 6.65 inches in 0.32 hours caused the most severe PMF. Using AWA, a local storm PMP was identified as the critical PMP for the East Wash watershed, with a 6-hour cumulative depth of 10.09 in. and an associated peak rainfall of 2.11 inches in 10 minutes.

PMF Discharges

The current design basis peak discharge from the Winters Wash watershed of 172,400 cfs occurs approximately 5 hr and 45 min after the start of the PMP (UFSAR Table 2.4-7). The reevaluated peak discharge determined by using the HEC-HMS is 33,260 cfs.

The current design basis peak discharge from the East Wash watershed of 16,600 cfs occurs approximately 2 hr and 10 min after the start of the PMP (UFSAR Table 2.4-7). The refined flood hazard reevaluation results are shown in *Table 3-4*. The maximum PMF discharge rate of 12,830 cfs occurs at cross-section X3 (*Figure 3-16*) in East Wash just south of the north embankment.

Stillwater Levels

The current design basis PMF water level along Winters Wash for the watershed PMP is 944.7 ft at cross-section B (UFSAR Table 2.4-16). The flood hazard reevaluation determined the peak flood elevation along Winters Wash would be 940.0 ft. using FLO-2D.

The current design basis PMF water levels for East Wash are 962.8 ft, 954.7 ft, and 944.0 ft for cross-sections A1, B, and C, respectively (UFSAR Figure 2.4-2, UFSAR Table 2.4-16). The refined flood hazard reevaluation results for the six hydraulic cross-sections are shown in *Table 3-4*, along with the embankment elevations and calculated embankment freeboard. The locations of these six floodplain cross-sections are shown in *Figure 3-14*. The analysis determined that East Wash PMF flows do not breach the north or east embankments at any point in the simulation. *Figure 3-16* shows the maximum water depths determined by the model.

<u>Wind-waves and Run-up Coincident with PMF</u>

The design basis wave run-up and set-up height in Winters Wash (at cross-section B) is 5.6 ft, (i.e., run-up 4.8 ft + set-up 0.8 ft. height)(UFSAR Table 2.4-16). The flood hazard reevaluation wave run-up on Winters Wash was 0.37 ft at the same location (*Figure 3-9*). The maximum flood elevation for the current design basis was obtained by summing the PMF water surface elevation, the wind setup, and the wave run-up height, which was evaluated for waves associated with a sustained overland wind velocity of 40 mph. The current design basis water level along Winters Wash for the watershed PMP is 950.3 ft at cross-section B (UFSAR Table 2.4-16). The flood hazard reevaluation maximum water surface elevation at this location, including run-up, was determined to be 940.4 ft, which does not reach the minimum elevation of the powerblock. *Table 4-3* summarizes these results.

The design basis maximum water levels, including wind-waves, for East Wash are 964.6 ft, 956.5 ft, and 945.8 ft for cross-sections A1, B, and C, respectively (UFSAR Table 2.4-16). The refined flood hazard reevaluation (*Table 3-5*) determined the water levels at these locations to be 964.78 ft., 956.58 ft., and 947.58 ft, respectively. Both the design bases and the refined flood hazard reevaluation show sufficient freeboard.

The design basis wave run-up and set-up height for the East Wash embankment at cross section X2 is 1.8 ft (UFSAR Table 2.4-16), which results in a freeboard of 1.75 ft. The run-up and setup for the north embankment is 4.0 ft (UFSAR Table 2.4-16), which results in a freeboard of 3.0 ft at UFSAR cross-section G1, which is equivalent to cross section X1 in the refined flood hazard reevaluation report (*Table 3-5*). At the critical fetch length cross-sections (*Figure 3-18*), the refined flood hazard evaluation wave run-up freeboard is 2.09 ft for the north embankment (at X1) and 1.72 ft for the east embankment (at X2) (*Table 3-7*). These values are comparable to the design basis.

Thus, the East Wash east and north embankments that realign the PMF flood around the site have sufficient freeboard to contain the PMF coincident with a wave height induced by a 2-year wind event. East Wash PMF flows do not overtop the north or east wash embankments at any point.

### 4.2.3 Combined Effects Flooding

No CEF analysis is documented in the current design basis.

As described in *Section 3.3*, the CEF for Winters and East Wash concluded that the PMF event is contained within the washes and the embankments are not overtopped.

### 4.2.4 Dam Breaches and Failures

The current design basis states that the site is not exposed to flooding due to dam failure using a domino failure analysis method and that peak flood levels only reach elevation 900 ft, which is below the plant grade elevations (UFSAR Section 2.4.4.3).

The reevaluation analysis for dam breaches and failures used the more conservative Volume Method screening analysis of ISG-2013-01, which utilizes 100% of the dam capacity and does not account for attenuation of flood waves. The reevaluation determined that peak flood levels did not exceed 935 ft as compared to the lowest grade elevation at the powerblock of 951 ft, which screened out dam failure at the site.

### 4.2.5 Storm Surge and Seiche, Tsunami, and Ice-Induced Flooding

Storm surge and seiche, tsunami, and ice-induced flooding were screened out as potential flooding events in the current design basis in the UFSAR and in the initial flood hazard reevaluation report.

### 4.2.6 Channel Diversion

The UFSAR confirmed that the plant and essential water supplies will not be adversely affected by natural stream channel diversion or, that in such an event, alternate water supplies are available for safety-related equipment.

The flood hazard reevaluation identified potential inflows from the Jackrabbit Wash into the Winters Wash watershed, but also determined that flood waters in Winters Wash did not impact the site. The reevaluation analysis excluded potential diversion across watershed divides into East Wash based on a series of simplified HEC-RAS simulations. Therefore, channel diversion was screened out at the site in the initial flood hazard reevaluation analysis.

## 4.3 Supporting Documentation

The reevaluated flood levels presented in this report are based on detailed calculations developed in support of the flood hazard reevaluation at the site. The calculations were prepared and reviewed by the responsible organizations and a client review was performed by APS. The critical calculations were also peer reviewed by an independent organization (*Table 4-5*). The *Flooding Walkdown Report* provides additional information regarding the current design basis flood hazard levels, as well as flooding protection and mitigation features. APS determined through the flooding walkdowns that the flood protection features were capable of providing the level of protection credited in the licensing basis. Nonconforming conditions discovered during the walkdowns were entered into the CAP.

### 4.3.1 Technical Justification of the Flood Hazard Analysis

The flood hazard reevaluation analyses described in this report utilized techniques, software, and methods used in present-day standard engineering practice. The technical basis for the various scenarios modeled under the HHA method and the key assumptions utilized in determination of the reevaluated flooding levels for each flood-causing mechanism are discussed individually in *Section 3.0* and are summarized in *Tables 4-1* through *4-4.*

### 4.3.2 Technical Justification by the Recommendation 2.3 Walkdown Results

The results from the *Flooding Walkdown Report* were taken into consideration during the implementation of the flood hazard reevaluation and when drawing conclusions from the analyses. Specifically, it was found that site modifications have not adversely affected the ability of safe shutdown equipment to perform their safety function.

## 4.4 Conclusions

No operator or mitigation actions are needed to ensure safe shutdown capability as a result of the flood hazard reevaluation. As no additional actions to protect against the reevaluated flood hazards are needed and the results are comparable to the licensing basis, APS believes that an integrated assessment is not needed or warranted.

### 4.4.1 Effects of LIP

The current licensing basis flood elevations for LIP are two feet below plant grade at the periphery of the powerblock. As described in *Section 2.2.1*, the current analysis used a methodology that accounted for water levels in ditches, but not transient water accumulation and sheet flow adjacent to buildings. Specific values for transient water accumulation adjacent to powerblock structures were not stated in the LIP licensing basis.

Using present-day regulatory guidance and methodologies, including current techniques, software, and methods used in present-day standard engineering practice, the flood hazard revaluation quantified localized water accumulations adjacent to safety-related SSCs. Specifically, these accumulations from LIP were determined to be of limited duration, although possibly reaching peak accumulations of 1 to 7 inches. These projected levels would exceed entrance elevations of a limited number of safety-related SSCs (i.e., doors or hatches). APS has determined in a room-by-room internal flooding analysis that localized accumulation of water adjacent to structures in the powerblock does not require operator action and does not impact safe shutdown equipment. Since the LIP flood levels for the current licensing bases do not require operator action, just as the reevaluated LIP flood levels do not require operator action to ensure the capability for safe shutdown, APS believes that no further action is required to address LIP.

### 4.4.2 PMF in Nearby Watercourses

The reevaluated PMF static flood levels in Winters Wash were approximately 5 ft lower than the static flood levels computed in the current licensing basis. Therefore, the licensing basis bounds the PMF static water levels and wave run-up height computed in the reevaluation analysis for Winters Wash.

The refined analysis determined that PMF flood levels in East Wash were comparable to the flood levels computed in the current design basis. The refined analysis determined that both the north embankment and east embankment of East Wash were not overtopped by wave run-up during the PMF event, which is consistent with the current licensing basis.

No CEF analysis is documented in the current licensing basis. As described in *Section 3.3*, the CEF for Winters and East Wash concluded that the PMF event is contained within the washes and there is no impact to safe shutdown equipment.

The reevaluated PMF flood levels were found to be comparable to current licensing basis flood levels. The results showed that there is no impact to the site from Winters Wash, the East Wash embankments are not overtopped, there is no new operator action required, and there is no impact to safe shutdown equipment. Therefore, APS believes that no further action is required to address PMF.

### 4.4.3 Remaining Flood-Causing Mechanisms

Both the current licensing basis and the reevaluation analysis dismiss flooding as a result of storm surge and seiche, tsunami, and ice-induced flooding.

Both the current licensing basis and the reevaluation analysis concluded that channel diversion is not a hazard for the site.

Both the current licensing basis and the reevaluation analysis screened out dam failure as a potential hazard to the site.

Based upon the reevaluation results for storm surge, seiche, tsunami, ice-induced flooding, channel diversion, and dam failure, no further action is required.

## 5.0 INTERIM EVALUATION AND ACTIONS

*Section 5.0* has been prepared in response to Item 1.d. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter: "Provide an 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."

At this time, there are no additional actions which are planned to address flooding hazards at the site.

## 6.0 ADDITIONAL ACTIONS

*Section 6.0* has been prepared in response to Item 1.e. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter: "Provide additional actions beyond Request for Information item 1.d taken or planned to address flooding hazards, if any."

At this time, there are no additional actions beyond Item 1.d. of NRC Recommendation 2.1 (*Section 5.0*) to address flooding hazards at the site.

## 7.0 REFERENCES

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# APPENDIX A

# TABLES

**Table 2-1: Existing Design Parameters**

| Parameter | Value | UFSAR |
|---|---|---|
| LIP due to thunderstorm PMP | 11.8" (1 hr)<br>15.53" (6 hr) | Table 2.4-6 |
| Maximum ponding depth on roof | ≤ 6" during 6-hr thunderstorm PMP | Section 2.4.2.3 |
| Maximized Storm PMP for the Winters Wash watershed | 14.6" (24.15 hr) | Table 2.4-10 |
| Maximized Storm PMP for the East Wash watershed | 14.44" (6.06 hr) | Table 2.4-11 |
| East Wash flow velocity during PMP | 6 fps (estimated) | Section 2.4.10 |
| East Wash embankment maximum local boundary shear for PMP | 0.8 psf | Section 2.4.10 |
| Maximum run-up level<br>Winters Wash<br>East Wash north embankment<br>East Wash east embankment | <br>4.8 ft<br>3.8 ft<br>1.7 ft | Tables 2.4-19 through 2.4-21 |
| Protection of safety-related facilities from inundation by offsite flood sources is achieved by the location of the facilities beyond the extent of flooding | Structure elevation<br>Unit 1 - 957.5 ft<br>Unit 2 - 954.5 ft<br>Unit 3 - 951.5 ft | Section 2.4.2.2.1<br>Figure 2.4-4 |
| Wind speed for wave run-up (over land) | 40 mph | Tables 2.4-19 through 2.4-21 |
| Freeboard in the Essential Spray Ponds | 4.0 ft | Design basis calculation |
| The onsite drainage system is designed so that runoff due to PMP will not inundate the safety-relates structures, equipment, and access to these facilities | Maximum surface water<br>Unit 1 - 955.5 ft<br>Unit 2 - 952.5 ft<br>Unit 3 - 949.5 ft | Section 2.4.2.3 |

**Table 2-2: Current Design Basis Flood Elevations Due to All Flood Mechanisms**

| Parameters | Value | UFSAR |
|---|---|---|
| Lowest Exterior Entrance Elevation for any Safety-Related Building | Unit 1 – 957.5 ft<br>Unit 2 – 954.5 ft<br>Unit 3 – 951.5 ft | Section 2.4.2.3 |
| Point-Value PMP (LIP) Flooding Level at the Periphery of the Powerblock | Unit 1 – 955.5 ft<br>Unit 2 – 952.5 ft<br>Unit 3 – 949.5 ft | Section 2.4.2.3 |
| PMF levels on East Wash | 926.6 ft at "F"<br>978.8 ft at "G2" | Section 2.4.3<br>Table 2.4-16 Sh 1 |
| East Wash Run-up + Wind Setup:<br>North Facing Embankment<br>East Facing Embankment | <br>4.0 ft<br>1.8 ft | Table 2.4-16 |
| PMF levels on Winters Wash | 929.5 ft at "D"<br>956.4 ft at "AA" | Section 2.4.3<br>Table 2.4-16 Sh 1 |
| Winters Wash Run-up + Wind Setup | 5.6 ft | Table 2.4-16 |
| Maximum Water Level with Upstream Dam Failure | 900 ft | Section 2.4.4.3 |
| Storm Surge and Seiche Flooding | N/A | Section 2.4.5 |
| Tsunami Flooding | N/A | Section 2.4.6 |
| Ice Flooding | N/A | Section 2.4.7 |
| Channel Diversion Flooding | N/A | Section 2.4.9 |

**Table 3-1: Characteristics of Winters Wash Sub-Basins and HEC-HMS Model Results [1]**

| Drainage Sub-Basin[2] | Sub-Basin Area (sq mi) | Green-Ampt Parameters[3] | | | | Impervious Area (%) | Lag Time[4] (hr) | PMP Depth (in.) | PMP Duration (hr) | Peak Discharge (cfs) | Time to Peak Discharge |
|---|---|---|---|---|---|---|---|---|---|---|---|
| WW-1 | 35.628 | 0.13 | 0.36 | 5.47 | 0.36 | 11.1 | 2.54 | 11.21 | 72 | 6,819 | 42:25 |
| WW-2 | 29.399 | 0.13 | 0.38 | 5.22 | 0.37 | 11.6 | 1.77 | 11.21 | 72 | 5,767 | 42:00 |
| WW-3 | 49.722 | 0.13 | 0.37 | 5.44 | 0.34 | 10.7 | 2.37 | 11.21 | 72 | 10,155 | 42:00 |
| WW-4 | 14.265 | 0.11 | 0.36 | 5.06 | 0.39 | 3.1 | 1.45 | 11.21 | 72 | 2,332 | 42:00 |
| WW-5 | 49.200 | 0.10 | 0.35 | 4.67 | 0.48 | 14.2 | 2.09 | 11.21 | 72 | 6,902 | 42:00 |
| WW-6 | 15.169 | 0.11 | 0.36 | 4.89 | 0.43 | 2.9 | 1.24 | 11.21 | 72 | 2,076 | 42:00 |
| WW-7 | 15.903 | 0.10 | 0.35 | 4.75 | 0.48 | 3.5 | 1.52 | 11.21 | 72 | 1,664 | 42:00 |
| WW-8 | 19.419 | 0.12 | 0.36 | 5.00 | 0.41 | 5.4 | 0.99 | 11.21 | 72 | 3,101 | 42:00 |
| WW-9 | 13.415 | 0.10 | 0.35 | 4.38 | 0.54 | 18.1 | 1.79 | 11.21 | 72 | 1,637 | 42:00 |
| WW-10 | 8.557 | 0.11 | 0.35 | 5.08 | 0.39 | 13.1 | 1.42 | 11.21 | 72 | 1,654 | 42:00 |
| WW-11 | 1.982 | 0.10 | 0.35 | 4.94 | 0.41 | 18.2 | 0.89 | 11.21 | 72 | 392 | 42:00 |
| WW-12 | 11.547 | 0.11 | 0.35 | 5.05 | 0.39 | 4.0 | 1.38 | 11.21 | 72 | 1,933 | 42:00 |
| WW-13 | 17.278 | 0.11 | 0.36 | 4.71 | 0.48 | 12.8 | 1.05 | 11.21 | 72 | 2,402 | 42:00 |

<u>Notes:</u>

[1] Results presented are for the transient most refined case (Case W8). which included normal soil conditions, rainfall losses, and reach routing

[2] Refer to *Figure 3-7* for a wash sub-basin map.

[3] Values from left to right are: initial soil moisture as a % volume, saturated soil moisture as a % volume, suction in inches, and soil vertical hydraulic conductivity (inches/hour)

[4] Lag time is reduced by 33% and the peak of unit hydrographs were increased by 5% to account for non-linearity effects (NUREG/CR-7046).

**Table 3-2: Summary of FLO-2D Simulations for Winters Wash Flooding**

| Case | PMF Hydrograph [1] | FLO-2D Domain | Domain Grid Size (ft) | Manning's Roughness [2] | Sedimentation Evaluation | With VBS and East Wash Embank Removed | Simulated Period (hr) |
|---|---|---|---|---|---|---|---|
| 1 | Constant at Peak PMF from downstream of PVNGS | Large | 50 x 50 | Higher | No | No | 11 |
| 2 | Downstream hydrograph applied upstream | Large | 50 x 50 | Higher | No | No | 96 |
| 3 | Downstream hydrograph applied upstream | Large | 50 x 50 | Higher | Yes | No | 96 |
| 9 | Time-Varying PMF from UFSAR (PVNGS, 2013) | Large | 50 x 50 | Lower | No | No | 48 |

Notes:

[1] Hydrographs are from HEC-HMS simulations for the most refined cases Case W8 for the Winters Wash.

[2] Manning's roughness coefficients are for the higher and lower ends of the recommended ranges

**Table 3-3: Floodplain Cross-Section PMF Results (100-ft Grid Element Model)**

| FLO-2D | Peak Flowrate (cfs) | Time to Peak (hr) | Total Discharge (acre-ft) |
|---|---|---|---|
| XS1 | 10,630 | 4.2 | 2,470 |
| XS2 | 10,090 | 4.4 | 2,500 |
| XS3 | 12,070 | 3.9 | 4,280 |
| XS4* | 1,400 | 3.2 | 120 |
| XS5* | 11,700 | 4.0 | 4,170 |

Note:

* XS4 and XS5 discharges were used as the hydrologic inputs into the 25-ft grid element model to represent flows from the East Wash watershed.

**Table 3-4: Floodplain Cross-Section PMF Results (25-ft Grid Element Model)**

| Floodplain Cross-Section * | Peak Flowrate (cfs) | Time to Peak (hr) | Maximum Water Surface Elevation** (ft) | Embankment Elevation (ft) | Embankment Freeboard (ft) |
|---|---|---|---|---|---|
| X1 | 11,770 | 4.53 | 979.5 | 983.0 | 3.5 |
| X2 | 12,530 | 4.79 | 974.9 | 978.0 | 3.1 |
| X3 | 12,830 | 4.85 | 969.2 | 973.1 | 3.9 |
| A1 | 12,280 | 4.95 | 963.4 | 967.5 | 4.1 |
| B | 11,040 | 5.13 | 955.2 | 961.6 | 6.4 |
| C | 10,860 | 5.30 | 946.2 | 948.4 | 2.2 |

Notes:

* Cross-sections X1, X2, and X3 are locations defined in *Figures 3-14* and *3-16*, and cross-sections A1, B, and C are cross-sections from the UFSAR report.

** Maximum water surface elevation does not include effects from wind-wave action.

**Table 3-5: Comparison of PMF Levels**

| Floodplain Cross-Section | X1 (G1) | A1 | B | C |
|---|---|---|---|---|
| Embankment elevation (ft) | 983.1 | 967.5 | 961.6 | 948.4 |
| UFSAR static flood level / (freeboard) (ft) | 976.1 (7.0) | 962.8 (4.7) | 954.7 (6.9) | 944.0 (4.4) |
| Wave run-up + set-up (ft) | 4.0 | 1.8 | 1.8 | 1.8 |
| Wind-wave flood level / (freeboard) (ft) | 980.1 (3.0) | 964.6 (2.9) | 956.5 (5.1) | 945.8 (2.6) |
| Refined static flood level / (freeboard) (ft) | 979.5 (3.6) | 963.4 (4.1) | 955.2 (6.4) | 946.2 (2.2) |
| Wave run-up + set-up (ft) | 1.51 | 1.38 | 1.38 | 1.38 |
| Wind-wave flood level / (freeboard) (ft) | 981.01 (2.09) | 964.78 (2.72) | 956.58 (5.02) | 947.58 (0.82) |

Note:

Cross-sections A1, B, and C are cross-sections from UFSAR Figure 2.4-2. Cross-section X1 is defined in *Figures 3-14* and *3-16*. Cross-section G1 is equivalent to cross-section X1 as shown in UFSAR Figure 2.4-2. The crest elevations of the embankments are based on survey data taken as part of the refined flood hazard reevaluation.

**Table 3-6: Fetch Data**

| Fetch | Depth (m) | Depth (ft) | Fetch Length (mi) | Fetch Length (ft) | Fetch Length (m) |
|---|---|---|---|---|---|
| North Embankment | 2.68 | 8.78 | 1.02 | 5,400 | 1,646 |
| East Embankment | 3.14 | 10.29 | 0.87 | 4,600 | 1,402 |

**Table 3-7: Wave Run-Up and Freeboard**

| Fetch | North Embankment | East Embankment |
|---|---|---|
| Surf Similarity, $\xi_0$ | 1.14 | 1.13 |
| Wave Run-up, $R_{2\%}$ (ft) | 1.51 | 1.38 |
| Antecedent Static Water Level (NGVD29) | 979.5 | 974.9 |
| Final Run-up Water Level (NGVD29) | 981.01 | 976.28 |
| Embankment Crest (NGVD29) | 983.1 | 978.0 |
| Static Freeboard (ft) | 3.60 | 3.10 |
| Run-up Freeboard (ft) | 2.09 | 1.72 |

**Table 3-8: Summary of FLO-2D Simulation Cases for Combined Effects Flooding**

| Simulation | Grid Cell Size (ft) | Manning's Roughness Coefficients | Detailed Representation of I-10 | Culverts Under WRF Access Road | Reference Simulation From FLO-2D PMF | East Wash Embank Removed |
|---|---|---|---|---|---|---|
| W1 [1] | 200 x 200 | 0.09 | No | N/A | N/A | N/A |
| W2 [1] | 200 x 200 | 0.06 | No | N/A | N/A | N/A |
| W3 [1] | 200 x 200 | 0.04 | No | N/A | N/A | N/A |
| E1 [2] | 50 x 50 | 0.09 | Yes | N/A | N/A | N/A |
| E2 [2] | 50 x 50 | 0.06 | Yes | N/A | N/A | N/A |
| E3 [2] | 50 x 50 | 0.04 | Yes | N/A | N/A | N/A |
| Case 1 [3] | 50 x 50 | 0.09 [4] | No | N/A | Case 2 | No |
| Case 2 [5] | 25 x 25 | 0.09 | Yes | Partially Unblocked | Case 4 | No |
| Case 3 [5] | 25 x 25 | 0.06 | Yes | Partially Unblocked | Case 5 | No |
| Case 4 [5] | 25 x 25 | 0.04 | Yes | Partially Unblocked | Manning's roughness sensitivity | No |
| Case 5 [5] | 25 x 25 | 0.06 | Yes | Partially Unblocked | Case 7 | Yes |
| Case 6 [5] | 25 x 25 | 0.06 | Yes | Blocked | N/A | No |

Notes:

[1] This is a model of the north domain of the Winters Wash watershed.

[2] This is a model of the north domain of the East Wash watershed.

[3] This is a model of the south domain of the Winters Wash watershed.

[4] These Manning's roughness coefficients correspond to the desert rangeland areas within the FLO-2D model. Appropriate coefficients were used for other land cover types near the site.

[5] This is a model of the south domain of the East Wash watershed.

**Table 4-1: Comparison of Modeling Approaches for CLB and Reevaluation**

| Modeling Approach | Reevaluated Hazards | Current Licensing Basis |
|---|---|---|
| Station used for Wind Analysis | Phoenix Sky Harbor International Airport | Phoenix Sky Harbor International Airport |
| Local Intense Precipitation (LIP) | AWA PMP Evaluation Tool | Calculated based on (*NWS, 1972*)[1] |
| LIP Flooding Characterization | A 2D flood routing model (FLO-2D) is used to simulate runoff from the powerblock and surrounding areas. | The powerblock areas were divided into small tributary areas, and runoff rates were computed for each area. These rates were supplied to periphery of powerblock. Flood elevations were computed using elevation-volume relationships developed for each tributary area. |
| PMP Calculation (River Flooding) | AWA PMP Evaluation Tool | Calculated based on Hershfield Method for Winters Wash; PMP Estimates for East Wash[2] |
| PMP Rainfall Hyetograph East Wash Watershed | Time period of 6 hrs with 10-min increments | Time period of 6.06-hr PMP, with 19.2-minute increments (UFSAR Table 2.4-11) |
| PMP Rainfall Hyetograph Winters Wash Watershed | Time period of 72 hrs with 6-hr increments | Time period of 24.15 hrs with 1.15-hr increments (UFSAR Table 2.4-10) |
| Rainfall-Runoff Model | USACE HEC-HMS[2] | HEC-HMS was not used for the UFSAR analysis; PMF was computed based on SCS method. |
| Transformation Method | Maricopa County S-graph[3] | SCS Type II Unit Hydrograph |
| PMF loss, and routing | Green-Ampt Method | SCS Curve Number Method |
| River Hydraulic Model | FLO-2D | Cross-sectional data was used to compute the PMF water level. |
| Dam Break Flooding | Volume Method used to determine water surface elevation | Seismically induced domino failure with an antecedent 500-yr flood event |
| Combined Effects Flooding | NUREG/CR-7046 and ANS, 1992 methods | Combined effects flooding analysis was not required and is not documented in the current design basis |

## Table 4-1: Comparison of Modeling Approaches for CLB and Reevaluation (Continued)

Notes:

[1] The local intense precipitation analysis in the UFSAR is based on a point value PMP distribution at the site (UFSAR Section 2.4.3.2).

[2] HEC-HMS is only used for modeling discharge upstream of the site for the River and Streams PMF analysis, but not the CEF analysis.

[3] Rainfall is directly applied to the FLO-2D models for the LIP, PMF, and CEF analysis.

**Table 4-2: Comparison of Analytical Inputs for CLB and Reevaluation**

| Analytical Input | Reevaluated Hazards | Current Licensing Basis |
|---|---|---|
| Local Intense Precipitation [1] | 10.73" (1 hr)<br>12.80" (6 hrs) | 11.8" (1 hr)<br>15.53" (6 hrs)<br>(UFSAR Table 2.4-6) |
| PMP for Winters Wash watershed (72-hr PMP) | 11.21" | A 72-hour PMP was not computed. The 24.15-hr PMP is 14.60"<br>(UFSAR Table 2.4-10) |
| PMP for East Wash watershed (6-hr value) | 10.09" | 14.44" (6.06-hr PMP) [2]<br>(UFSAR Table 2.4-11) |
| PMF Model, peak flow rate north of the site | 12,830 cfs<br>(East Wash, cross-section X3 from refined model)<br><br>33,260 cfs (Winters Wash) [3] | 16,600 cfs (East Wash)<br>(UFSAR Table 2.4-7)<br><br>172,400 cfs (Winters Wash)<br>(UFSAR Table 2.4-7) |
| CEF model, peak flow rates north of the site | 15,842 cfs (East Wash)<br>29,585 cfs (Winters Wash) [4] | CEF not required |
| Maximum Sustained Overland 10-min, 2-yr Wind Speed | 39.35 mph | 40 mph<br>(UFSAR Section 2.4.3.6) |
| Maximum Sustained Overwater 10-min, 2-yr Wind Speed | 47.28 mph | 48.8, 43.2, and 46.8 mph [5]<br>(UFSAR Tables 2.4-19 through 2.4-21) |

Notes:

[1] The local intense precipitation analysis in UFSAR Section 2.4.3.2 is based on a point value PMP distribution at the site.

[2] The value of 14.44 inches is obtained from summing the incremental rainfall values presented in UFSAR Table 2.4-11. The area reduction for the 6-hour PMP is reported as 93% (UFSAR Section 2.4.3.1.2), which reduces 15.53 inches to 14.44 inches.

[3] The listed flows are from the HEC-HMS simulations.

[4] The listed flows are from the FLO-2D simulations; the CEF analysis is more detailed than the HEC-HMS simulations.

[5] Values presented are for Winters Wash, East Wash east-facing embankment, and East Wash north-facing embankment, respectively.

**Table 4-3: Comparison of Flood Levels for CLB and Reevaluation**

| Flooding Mechanism | Reevaluated Water Level (ft) | Current Licensing Basis Water Level (ft) |
|---|---|---|
| Maximum Transient Water Accumulation Depths at Safety-Related Structures for the LIP | 0.19 to 0.63 (transient water accumulation)[1]<br><br>1.0 to 1.75 ft below plant grade at localized sections near the powerblock [3] | Did not specify [2]<br><br>2 ft below plant grade at each unit [4] (UFSAR Section 2.4.2.3) |
| River Flooding: East Wash (Stillwater) | 979.5 at "X1" [5]<br>963.4 at "A1"<br>955.2 at "B"<br>946.2 at "C"<br>(Refined model) | 976.1 at "G1" [5]<br>962.8 at "A1"<br>954.7 at "B"<br>944.0 at "C"<br>(UFSAR Table 2.4-16) |
| River Flooding: Winters Wash (Stillwater) | 940.0 at "B" | 944.7 at "B"<br>(UFSAR Table 2.4-16) |
| River Flooding East Wash Wave Run-up + setup: North Facing Embankment | 1.51<br>Cross-section X1(Refined model) | 4.0 at "G1"<br>(UFSAR Table 2.4-16) |
| River Flooding East Wash Wave Run-up + setup: East Facing Embankment | 1.38<br>(Refined model)<br>Cross-sections A1, B and C | 1.8<br>(UFSAR Table 2.4-16) |
| River Flooding Wave Run-up + setup: Winters Wash | 0.37 | 5.6<br>(UFSAR Table 2.4-16) |
| River Flooding Flood Elevations with Wave Run-up, East Wash | 981.01 at "X1"<br>964.78 at "A1"<br>956.58 at "B"<br>947.58 at "C"<br>(Refined model) | 980.1 at "G1"<br>964.6 at "A1"<br>956.5 at "B"<br>945.8 at "C"<br>(UFSAR Table 2.4-16) |
| River Flood Freeboard Wave Run-up + setup, East Wash | 2.09 at "X1"<br>2.72 at "A1"<br>5.02 at "B"<br>0.82 at "C"<br>(Refined model)[6] | 3.0 at "G1"<br>2.9 at "A1"<br>5.1 at "B"<br>2.6 at "C"<br>(UFSAR Table 2.4-16) |

## Table 4-3: Comparison of Flood Levels for CLB and Reevaluation (Continued)

| Flooding Mechanism | Reevaluated Water Level (ft) | Current Licensing Basis Water Level (ft) |
|---|---|---|
| River Flood Elevations with Wave Run-up, Winters Wash | 940.4 | 950.3 (UFSAR Table 2.4-16) |
| River Flood Freeboard Wave Run-up + setup, Winters Wash | 10.6 [7] | 0.7 [7] |
| Dam Failure Flooding | Screened Out | Screened Out |
| Storm Surge & Seiche Flooding | Screened Out | Screened Out |
| Tsunami Flooding | Screened Out | Screened Out |
| Ice Flooding | Screened Out | Screened Out |
| Channel Diversion Flooding | Screened Out | Screened Out |

<u>Notes:</u>

[1] APS room-by-room internal flood analysis evaluated the localized transient water accumulation adjacent to structures and determined that there was no impact to safe shutdown equipment.

[2] The licensing bases did not provide a specific value for the transient water accumulation phenomenon. However, the design of powerblock structures did include sufficient capability to mitigate internal flooding resulting from high- and moderate-energy line breaks which was implicitly assumed to bound the effects of external flooding from the localized transient water accumulation during the LIP event.

[3] The initial flood hazard reevaluation for sitewide inundation determined that water around the powerblock runs off during the LIP to the peripheral drainage system with some accumulation in localized areas approximately 1.0 to 1.75 ft below the plant floor elevation at each unit. The areas of water accumulation are limited in size and depth and are primarily due to localized grade depressions as depicted in *Figures 3-3, 3-23* and *3-24*.

[4] CLB values from UFSAR Section 2.4.2.3. The levels reported in the current design basis refer to water surface elevations at the periphery of the powerblock and do not account for sheet flow and transient water accumulation adjacent to buildings.

[5] Cross-section X1 in the refined model is equivalent to G1 from UFSAR Figure 2.4-2.

[6] The crest elevations of the embankment are based on survey data taken as part of the refined Flood Hazard Reevaluation Report.

[7] Freeboard for Winters Wash with wave-runup was calculated with respect to the lowest plant grade elevation (951 ft) (UFSAR 2.4.2.2.2).

**Table 4-4: Comparison of Flooding for CLB and Reevaluation**

| **Flood Condition** | **Reevaluated Flood Hazard** | **Current Licensing Basis** |
|---|---|---|
| Local Intense Precipitation | | |
| Flood Depth (ft) | 0.19 to 0.63 (transient water accumulation) [1]<br><br>1.0 to 1.75 ft below plant grade at localized sections near the powerblock [3] | Did not specify [2]<br><br>2 ft below plant grade at each unit [4] (UFSAR Section 2.4.2.3) |
| Flood Duration (hr) | 2 to 5 hours [5] | Did not specify [6] |
| Maximum Flow Velocity (ft/sec) | 1.7 [7] | Did not specify [6] |
| Maximum Hydrostatic Loading (lb/ft) | 10.2 [8] | Did not specify [9] |
| Maximum Hydrodynamic Loading (lb/ft) | 3.2 [8] | Did not specify [6] |
| Flood Elevation with Debris Effects | Screened Out [10] | Did not specify [6] |
| Flood Elevation with Sedimentation Effects | Screened Out [11] | Did not specify [6] |

**Table 4-4: Comparison of Flooding for CLB and Reevaluation (Continued)**

| Flood Condition | Reevaluated Flood Hazard | Current Licensing Basis |
|---|---|---|
| Flooding in Rivers and Streams[12] | | |
| Flood Elevation along East Wash with wind-wave effects (ft) | 981.01 at "X1"<br>964.78 at "A1"<br>956.58 at "B"<br>947.58 at "C"<br>(Refined model) | 980.1 at "G1"<br>964.6 at "A1"<br>956.5 at "B"<br>945.8 at "C"<br>(UFSAR Table 2.4-16) |
| Flood Elevation along Winters Wash with wind-wave effects (ft) | 940.4 | 950.3<br>(UFSAR Table 2.4-16) |
| Flood Duration (hr) for PMF | Footnote 14<br>(Refined model) | Duration of PMF is 13 hours [6]<br>(UFSAR Figure 2-4-13 and Figure 2.4-14) |
| Maximum Flow Velocity in East Wash (ft/sec) | 3 to 7 [13]<br>(Refined model) | 6 [6, 13]<br>(UFSAR Section 2.4.10) |
| Debris Effects | Screened Out | N/A |
| Sedimentation Effects | Screened Out | Scour due to sediment transport during river flooding was evaluated. (UFSAR Section 2.4.10) |

## Table 4-4: Comparison of Flooding for CLB and Reevaluation (Continued)

Notes:

[1] APS room-by-room internal flooding analysis evaluated the localized transient water accumulation adjacent to structures and determined that there was no impact to safe shutdown equipment.

[2] The licensing bases did not provide a specific value for the transient water accumulation phenomenon. However, the design of powerblock structures did include sufficient capability to mitigate internal flooding resulting from high- and moderate-energy line breaks, which was implicitly assumed to bound the effects of external flooding from the localized transient water accumulation during the LIP event.

[3] The initial flood hazard reevaluation for sitewide inundation determined that water around the powerblock runs off during the LIP to the peripheral drainage system with some accumulation in localized areas approximately 1.0 to 1.75 ft below the plant floor elevation at each unit. The areas of water accumulation are limited in size and depth and are primarily due to localized grade depressions as depicted in *Figures 3-3, 3-23* and *3-24*.

[4] CLB values from UFSAR Section 2.4.2.3. The levels reported in the current design basis refer to water surface elevations at the periphery of the powerblock and do not account for sheet flow and transient water accumulation adjacent to buildings.

[5] Flood duration transient, where water enters the building, lasts on average two to three hours and a maximum of five hours depending on the door and adjacent grade and curb features.

[6] Flooding does not reach safety-related SSCs.

[7] Maximum flood velocity near doors within powerblock.

[8] Maximum load at safety-related structures.

Hydrostatic Loading - Detailed structural analysis regarding the effects of these forces is not required because the flow vectors that are computed in this analysis indicate that the flows are away from Seismic Category I structures.

Hydrodynamic Forces- act in the direction of flow velocity. Consequently, the reported hydrodynamic forces should be interpreted as a conservative estimate. In cases where flow velocity is directed away from or parallel to the door, the hydrodynamic force acting on the door is zero.

[9] Flooding does not reach safety-related SSCs. Hydrostatic loads due to groundwater were considered for the current design basis, but not in the reevaluation study.

[10] Simulated water levels were too shallow, velocities were too low, and flow directions were not toward buildings.

[11] Simulated low velocities at the powerblock were too low and flow directions were not toward buildings.

[12] Includes the CEF analysis as the most refined cases. CEF was not considered in the design bases because it was not required at the time of license issuance.

[13] Maximum hydrodynamic and hydrostatic loads associated with river flooding were computed for all inundated areas with the FLO-2D model domain in the CEF evaluation. Forces associated with flooding on the powerblock during river flooding were bounded by the forces associated with the LIP flooding.

Using a velocity of 6 fps, 10 ft maximum water depth, and an average stone diameter of 12 inches, the value of local boundary shear is 0.8 psf. Since the range of velocity in the wash is 3 to 7 fps, then the corresponding loading on the embankment is less than the design values of 3.6 and 3.1 psf (UFSAR 2.4.10).

[14] Flood durations in East Wash and Winters Wash are not provided since the PMFs in the washes do not result in overtopping the embankment or inundating the site.

**Table 4-5: List of Supporting Documents**

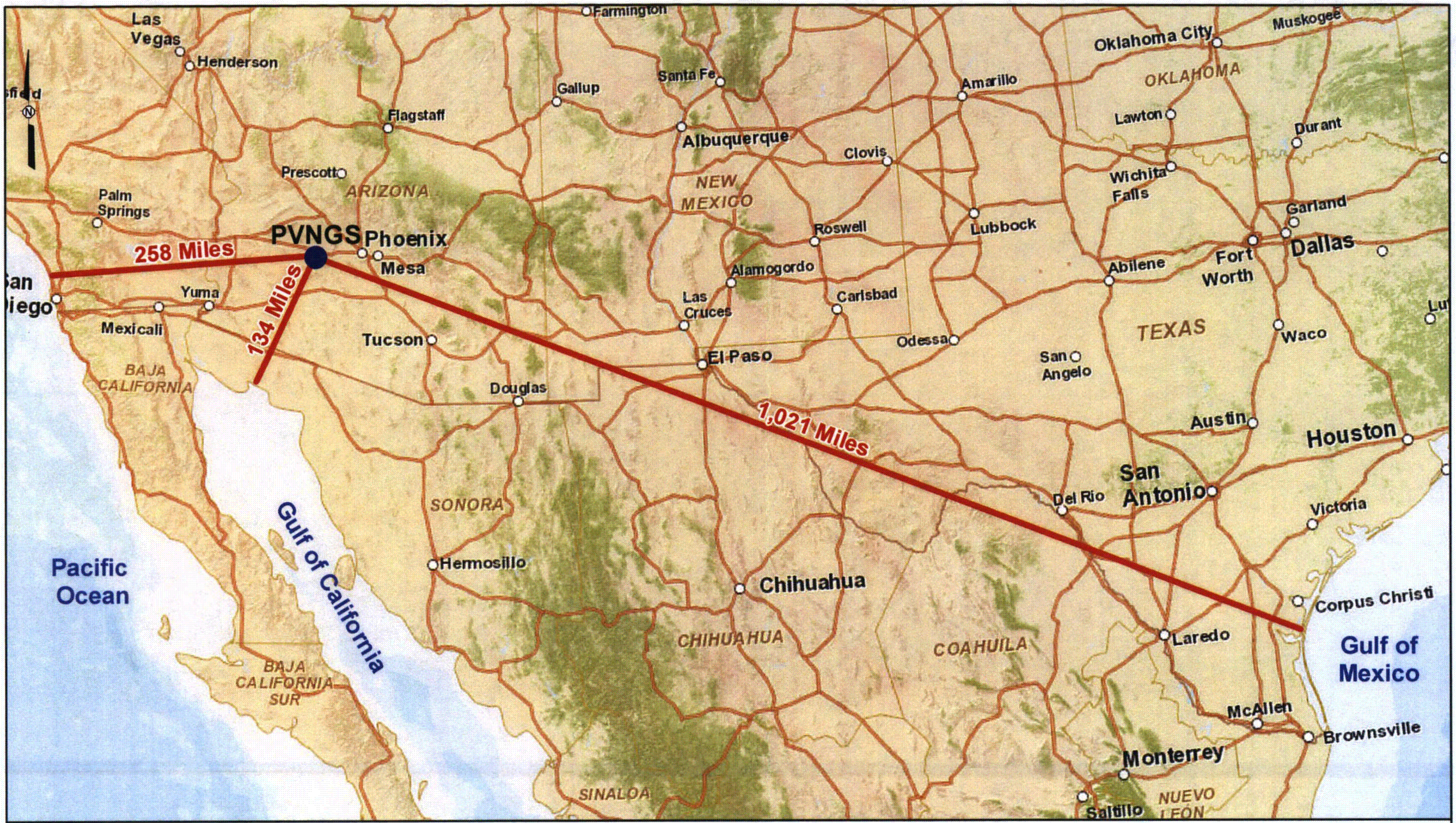
| Document Title | Originating Organization | Peer Review Organization |
|---|---|---|
| Local Intense Precipitation | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| Effects of Local Intense Precipitation Using FLO-2D | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| Wind-Wave Activity Coincident with the LIP | Westinghouse Electric Company/ Paul C. Rizzo Associates | Deemed Unnecessary |
| Watershed Delineation for East Wash and Winters Wash | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| PMP Estimation for the East Wash and Winters Wash Watershed | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| Probable Maximum Flood in Rivers | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| Water Level Estimation Due to PMF Using FLO-2D | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| Potential Dam Breaches and Failures | Westinghouse Electric Company/ Paul C. Rizzo Associates | Deemed Unnecessary |
| Screening of Coastal Flooding | Westinghouse Electric Company/ Paul C. Rizzo Associates | Deemed Unnecessary |
| Channel Diversion Flooding | Westinghouse Electric Company/ Paul C. Rizzo Associates | Deemed Unnecessary |
| Wind-Wave Activity Coincident with the PMF | Westinghouse Electric Company/ Paul C. Rizzo Associates | Deemed Unnecessary |

**Table 4-5: List of Supporting Documents (Continued)**

| Document Title | Originating Organization | Peer Review Organization |
|---|---|---|
| Low Water Considerations | Westinghouse Electric Company/ Paul C. Rizzo Associates | Deemed Unnecessary |
| Ice Flooding | Westinghouse Electric Company/ Paul C. Rizzo Associates | Deemed Unnecessary |
| Combined Effects | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| Transmittal of Palo Verde Nuclear Generating Station Flood Hazard Reevaluation Closeout Documentation & Third Party Review | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| Palo Verde Nuclear Generating Station - Procedure for Aerial Photography of the Owner Controlled Area (OCA) as a basis of Topographical Mapping | Westinghouse Electric Company | Deemed Unnecessary |
| AeroTech Aerial Photographic Coverage Report | AeroTech Mapping | Deemed Unnecessary |
| URS Review of Palo Verde Nuclear Generating Station Flood Hazard Reevaluation Calculation | Westinghouse Electric Company/ Paul C. Rizzo Associates | URS Corporation |
| Evaluation of Internal Flooding in Safety Related Structures as a Result of Localized Ponding at the Power Block During a LIP Event in support of NRC 50.54(f) letter and the PVNGS Flood Hazard Reevaluation Report | APS | Sargent & Lundy |

# APPENDIX B

# FIGURES




## Legend

- Site Center Point
- Shortest Distance from Water Body to Site

Coordinate System: NAD_1983_StatePlane_Arizona_Central_FIPS_0202_Feet
Projection: Transverse_Mercator

160 80 0 160 320 Miles

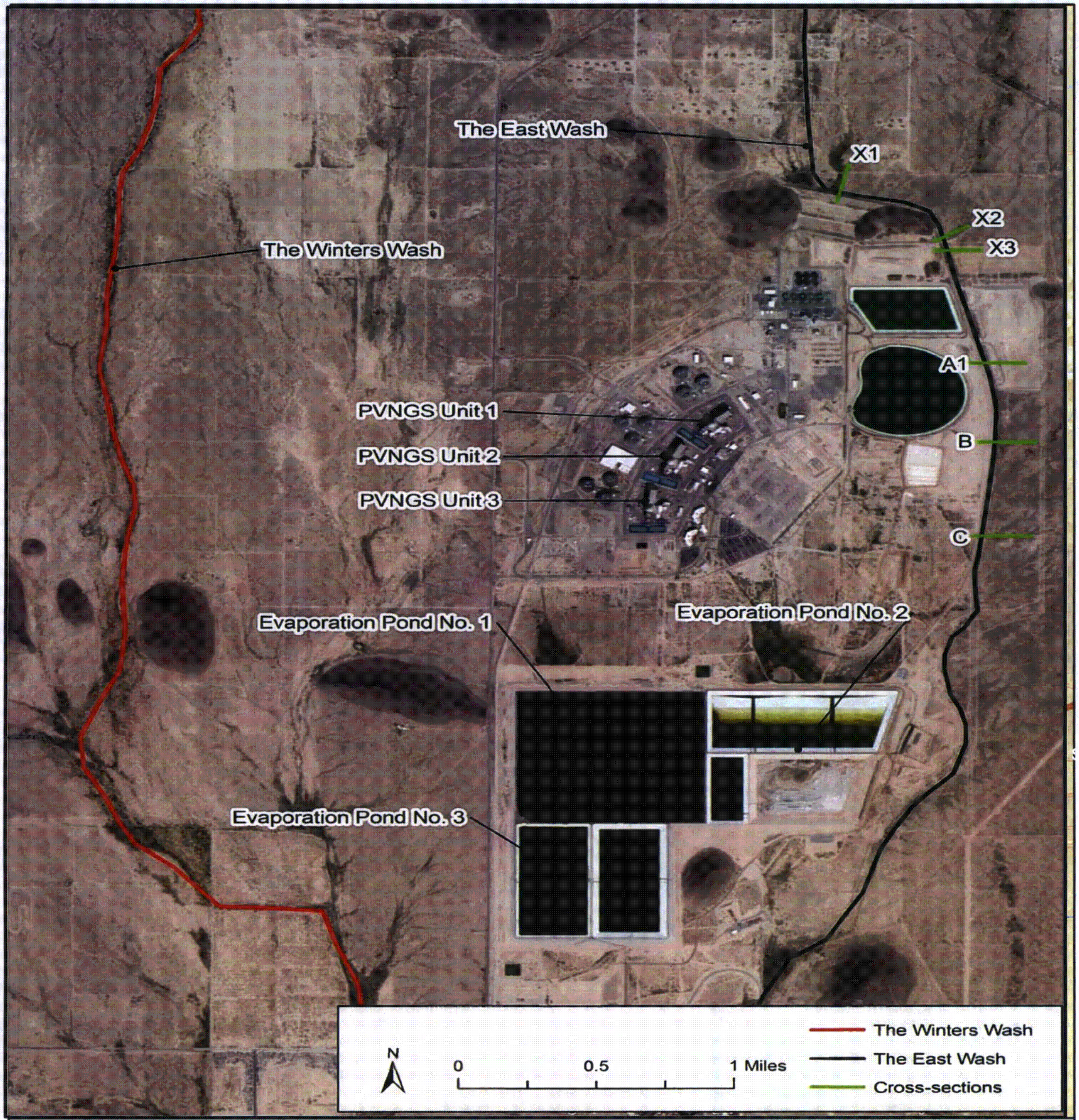
RF: 1:8,320,000

REFERENCE:
Background Image: ESRI, 2013a, Environmental Systems Research Institute (ESRI), "World Street Map"
Website: http://goto.arcgisonline.com/maps/World_Street_Map
Date Accessed: January 14, 2014

**FIGURE 1-1**

**GEOGRAPHIC LOCATION OF PALO VERDE NUCLEAR GENERATING STATION**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**




**Legend**

Site Center Point

Watershed Boundaries

USGS Gauge Stations

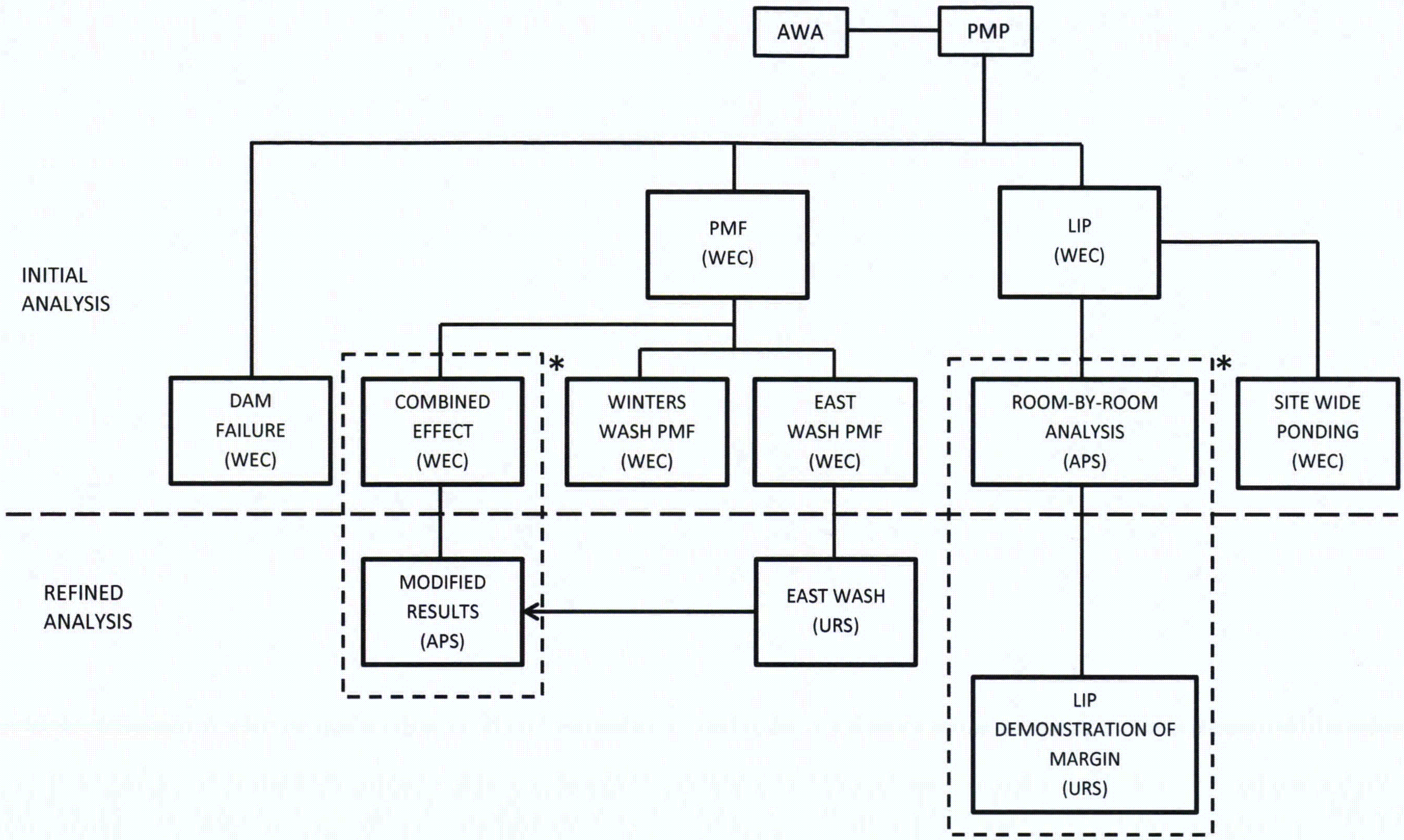
Rivers and Streams

REFERENCE:
Background Image: ESRI, 2013a, Environmental Systems Research Institute (ESRI), "World Street Map"
Website:http://goto.arcgisonline.com/maps/World_Street_Map
Date Accessed: January 14, 2013
USGS, 2013c, United States Geological Survey (USGS)
"USGS Water Data for the Nation,' Website <http://waterdata.usgs.gov/nwis>,
Date Accessed: October 9, 2013

**FIGURE 1-2**

**GENERAL LOCATION MAP OF THE SITE**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**




* NOT IN DESIGN BASIS

**FIGURE 1-3**

**FLOOD HAZARD REEVALUATION FLOWCHART**

**PALO VERDE NUCLAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**

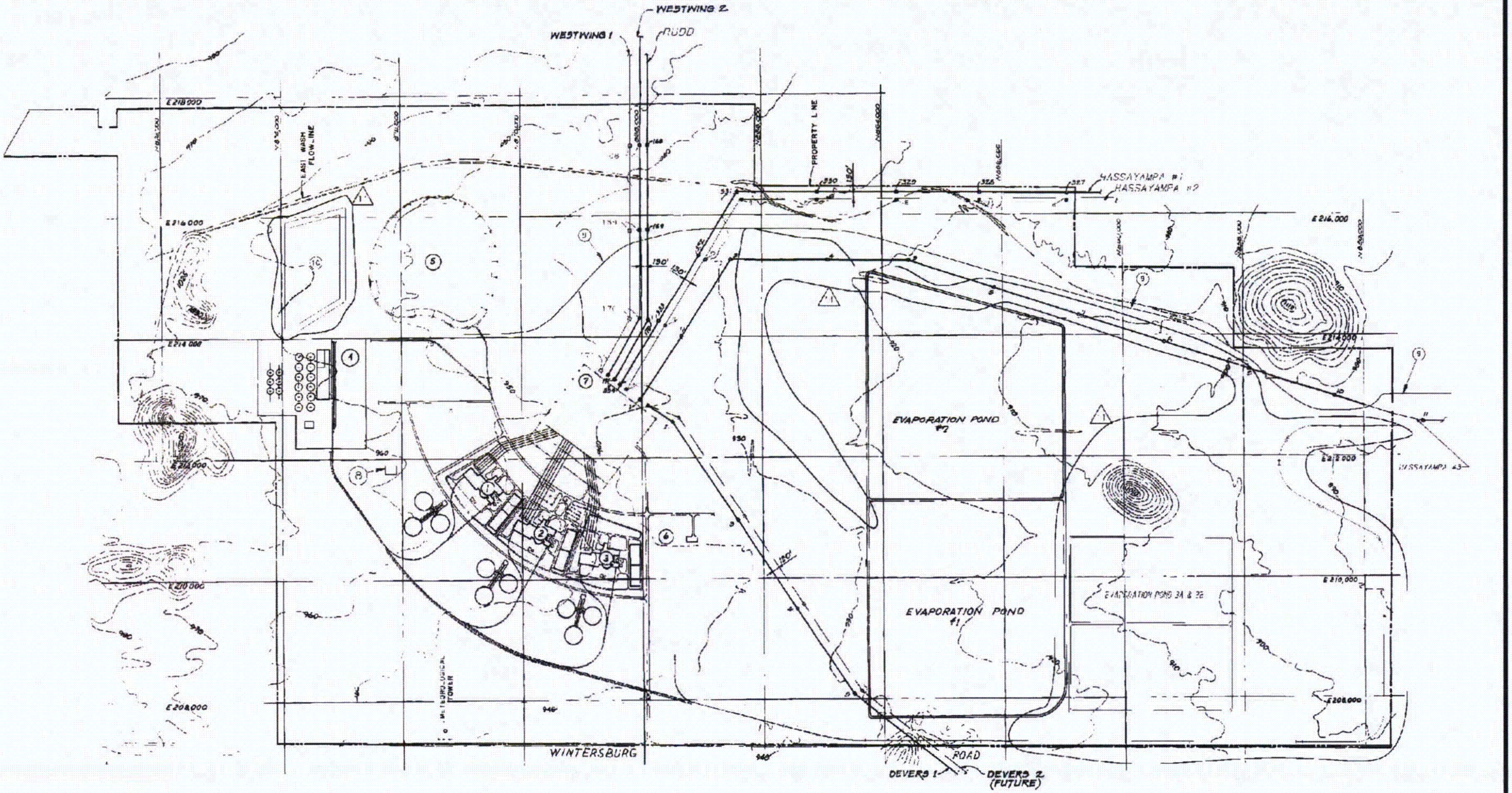



**FIGURE 2-1**

**SITE LAYOUT TOPOGRAPHY**

**PALO VERDE NUCLEAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**


LEGEND:

1. AUXILIARY BUILDING
2. CONDENSATE STORAGE TANK
3. CONTROL BUILDING
4. DIESEL GENERATOR BUILDING
5. EMERGENCY FUEL OIL TANKS
6. FUEL BUILDING
7. CONTAINMENT BUILDING
8. ESSENTIAL SPRAY POND
9. MAIN STEAM SUPPORT STRUCTURE
10. REFUELING WATER TANK

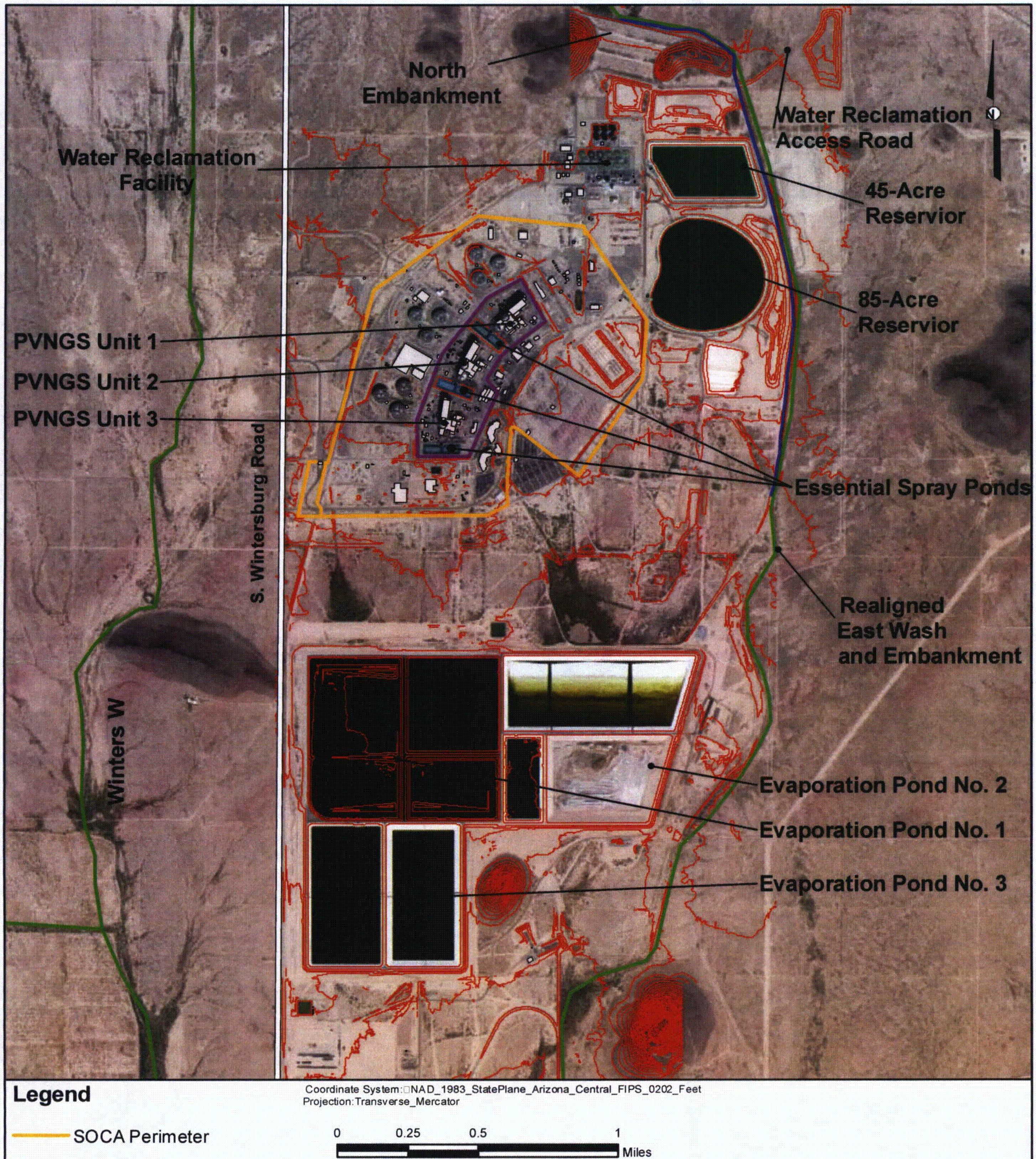
NOTE:

BACKGROUND IMAGE MODIFIED FROM: GOOGLE EARTH, 2014

**FIGURE 2-2**

**POWERBLOCK ARRANGEMENT**

**PALO VERDE NUCLEAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**




Reference:
Background Source: ESRI, 2013b, Environmental Systems Research Institute (ESRI), "World Imagery"
Website:http://goto.arcgisonline.com/maps/World_Imagery
Date Accessed: January 14, 2014

**FIGURE 2-3**

**AERIAL VIEW OF SITE LAYOUT**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

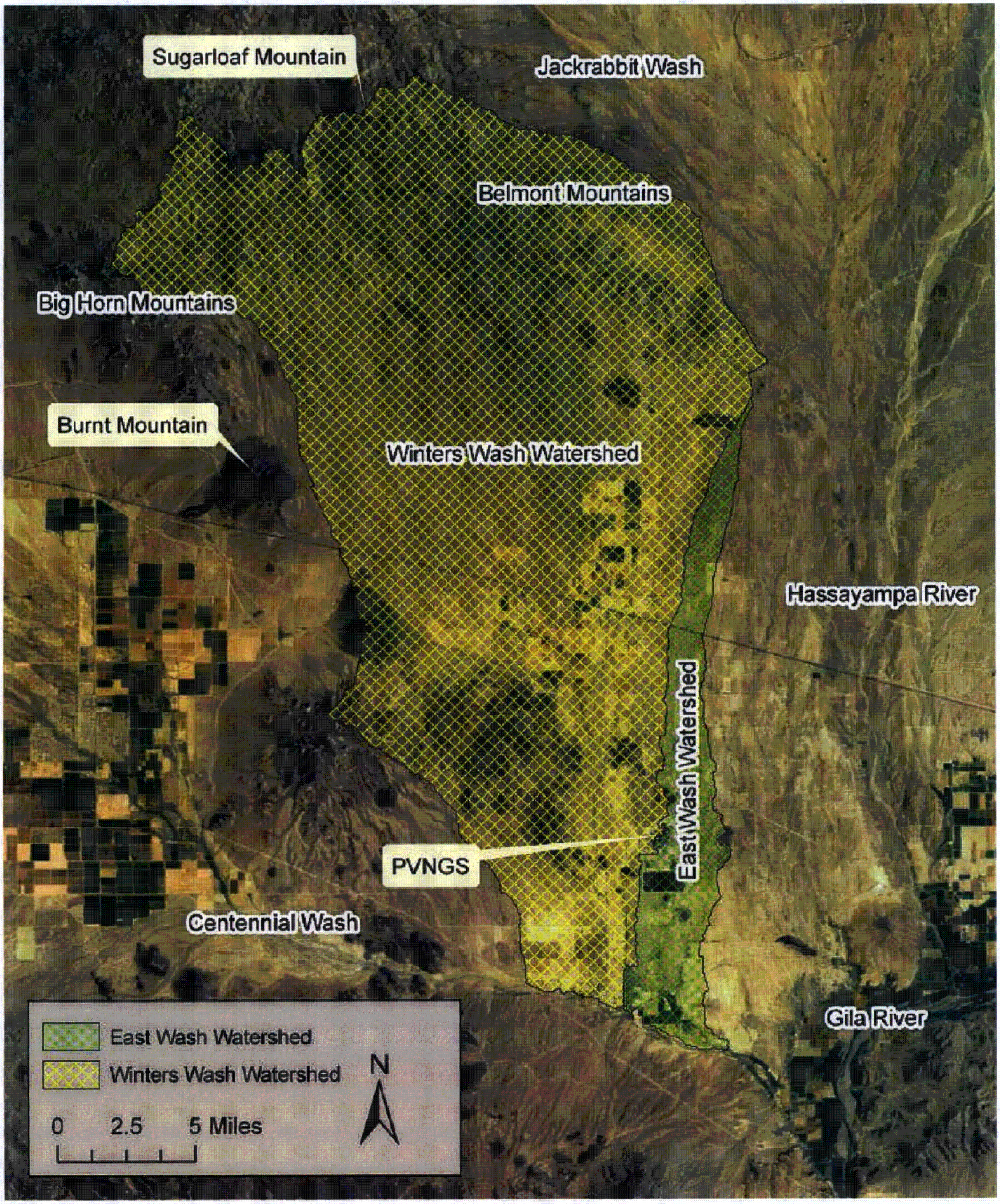
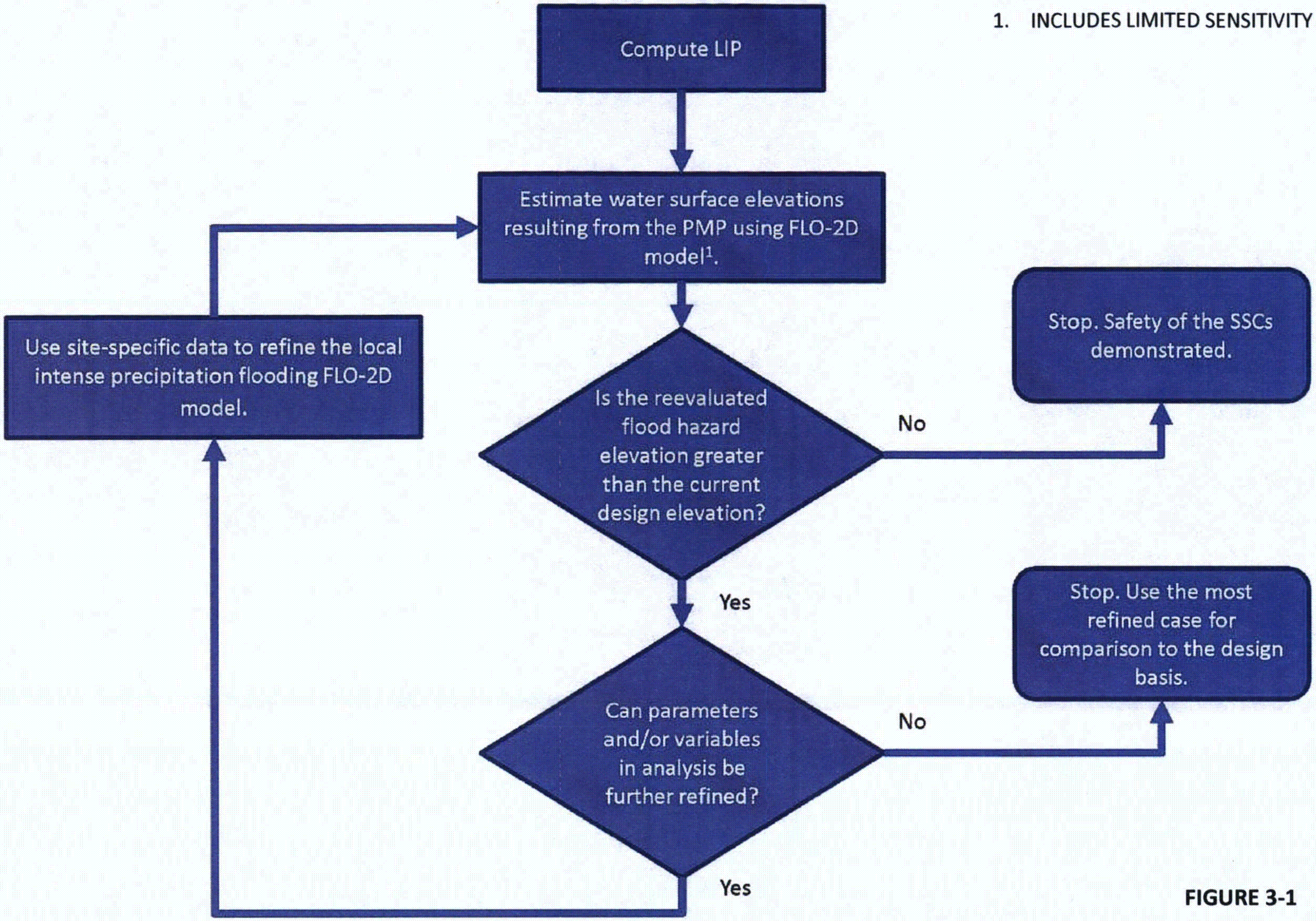



FIGURE 2-4

EAST WASH & WINTERS WASH WATERSHED

PALO VERDE NUCLEAR GENERATING STATION
FLOOD HAZARD REEVALUATION REPORT



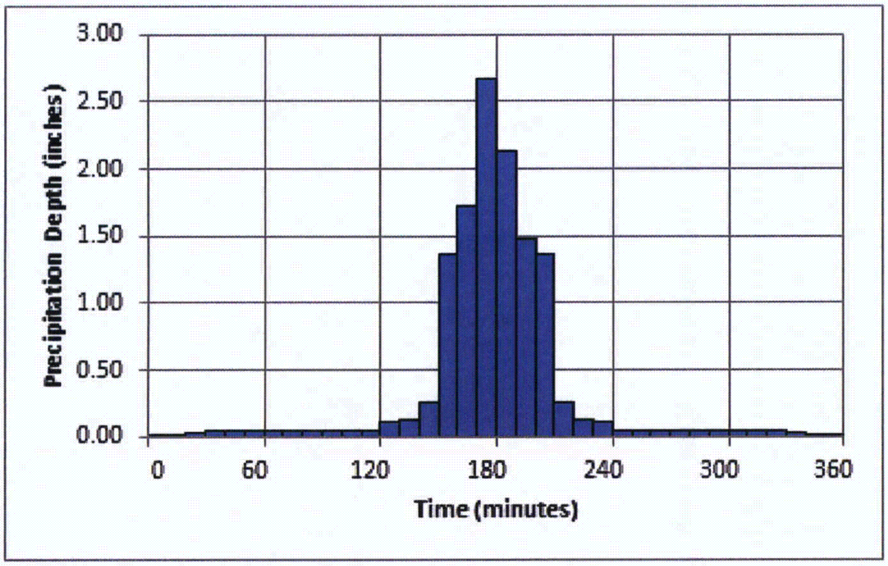

NOTE:

1. INCLUDES LIMITED SENSITIVITY ANALYSIS

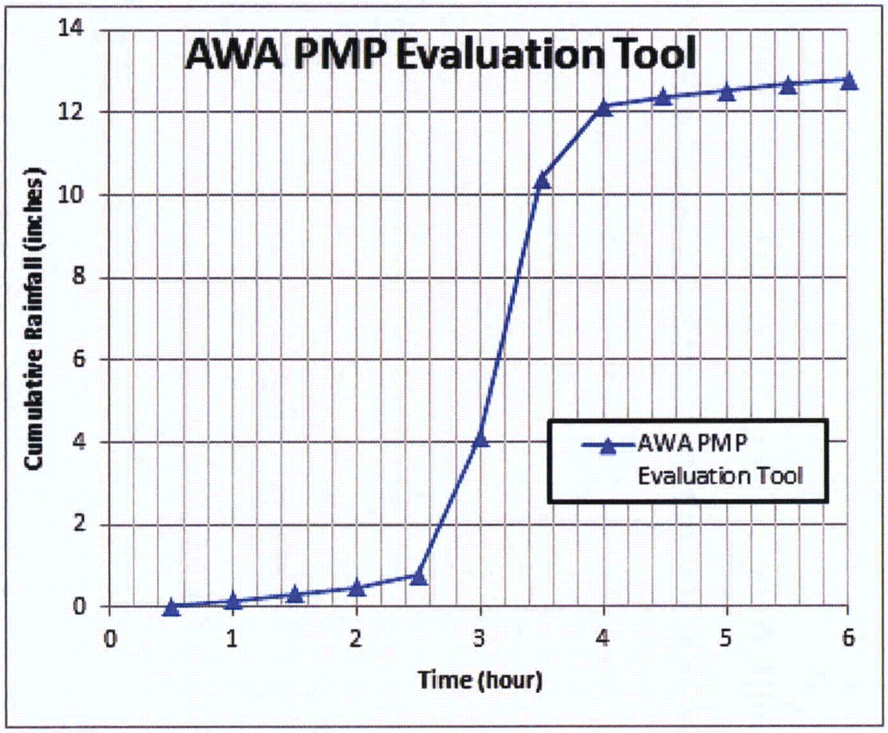
**FIGURE 3-1**

**HHA DIAGRAM FOR LOCAL INTENSE PRECIPITATION FLOODING ANALYSIS**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**




**INCREMENTAL RAINFALL DISTRIBUTION FOR THE 6-HR DURATION LIP FOR REEVALUATION ANALYSIS**




**CUMULATIVE REEVALUATION LIP HYETOGRAPHS**

**FIGURE 3-2**

**LOCAL INTENSE PRECIPITATION HYETOGRAPHS**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

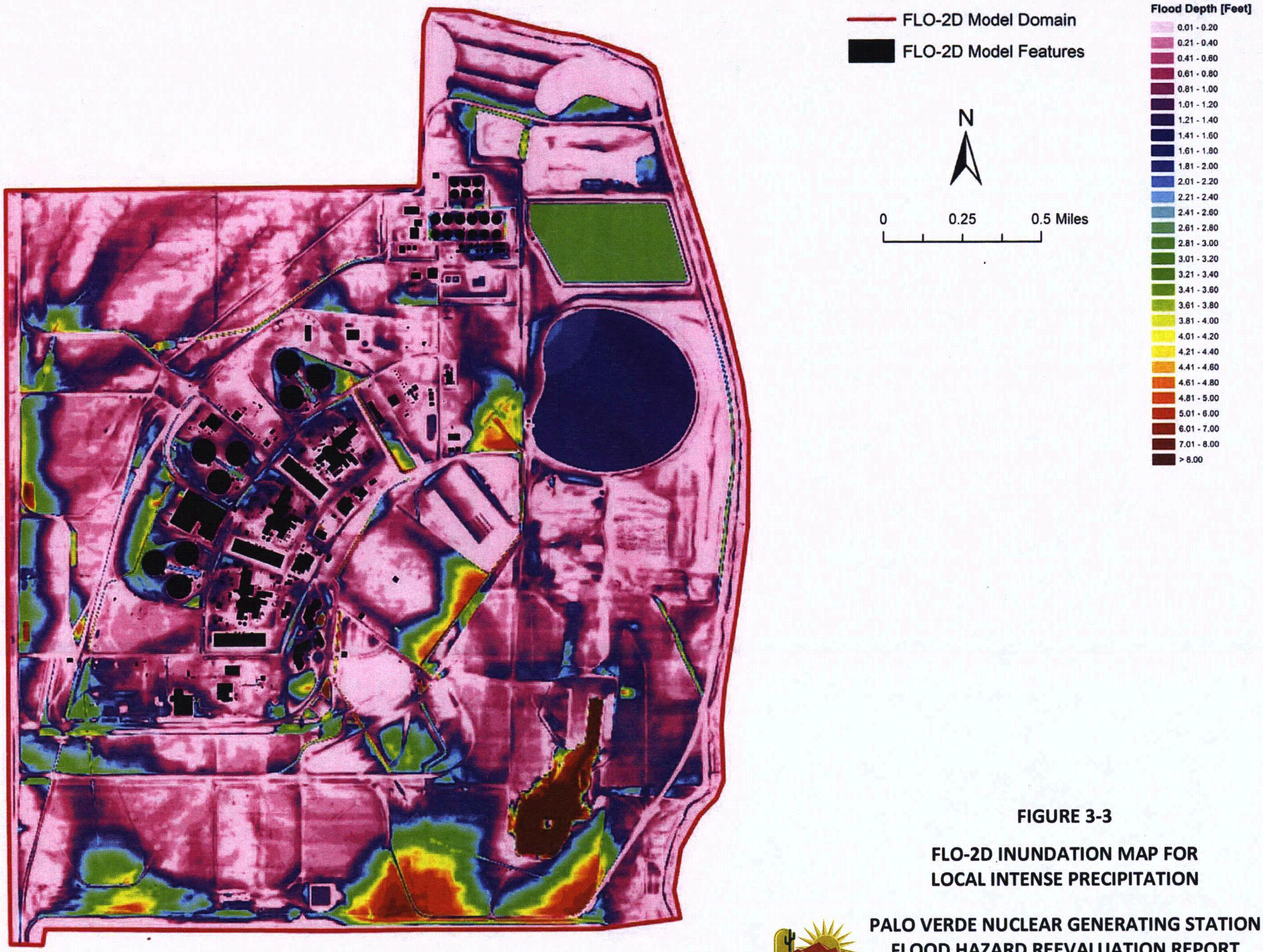



**FIGURE 3-3**

**FLO-2D INUNDATION MAP FOR LOCAL INTENSE PRECIPITATION**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

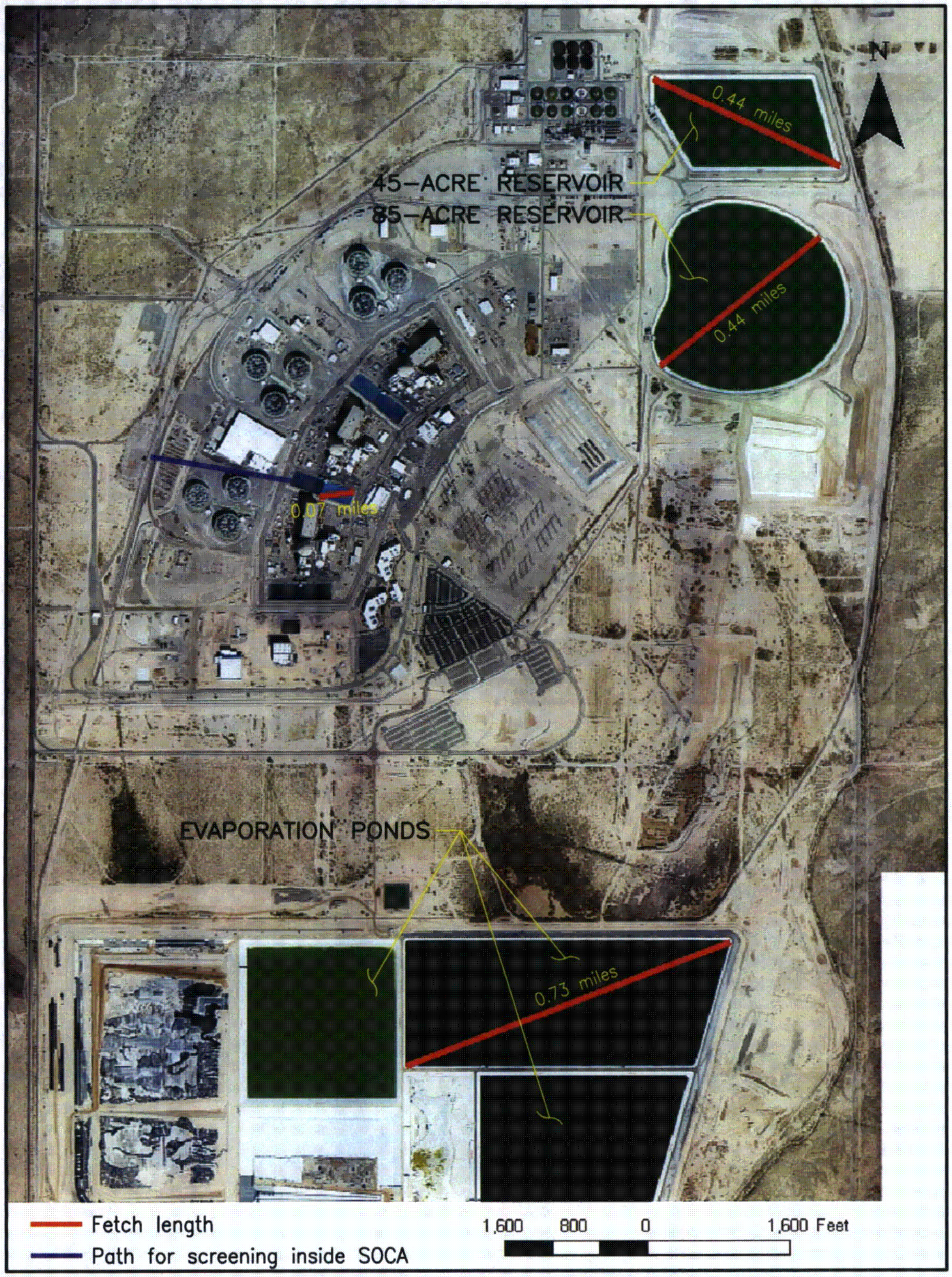


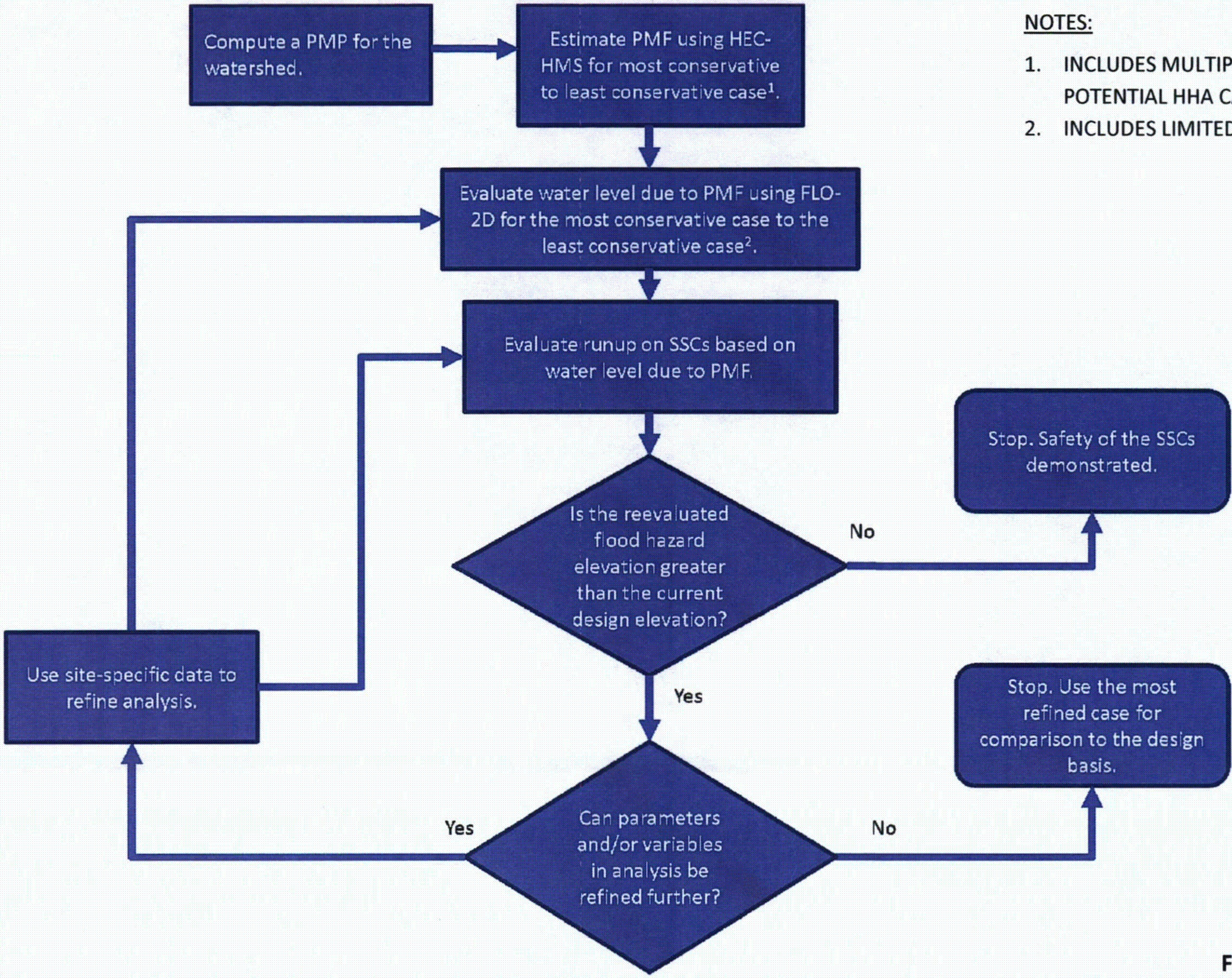

**FIGURE 3-4**

**FETCH LOCATIONS FOR WIND-WAVE ACTIVITY COINCIDENT WITH LIP FLOODING**

NOTE:

FETCH LENGTHS ARE APPROXIMATE

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**



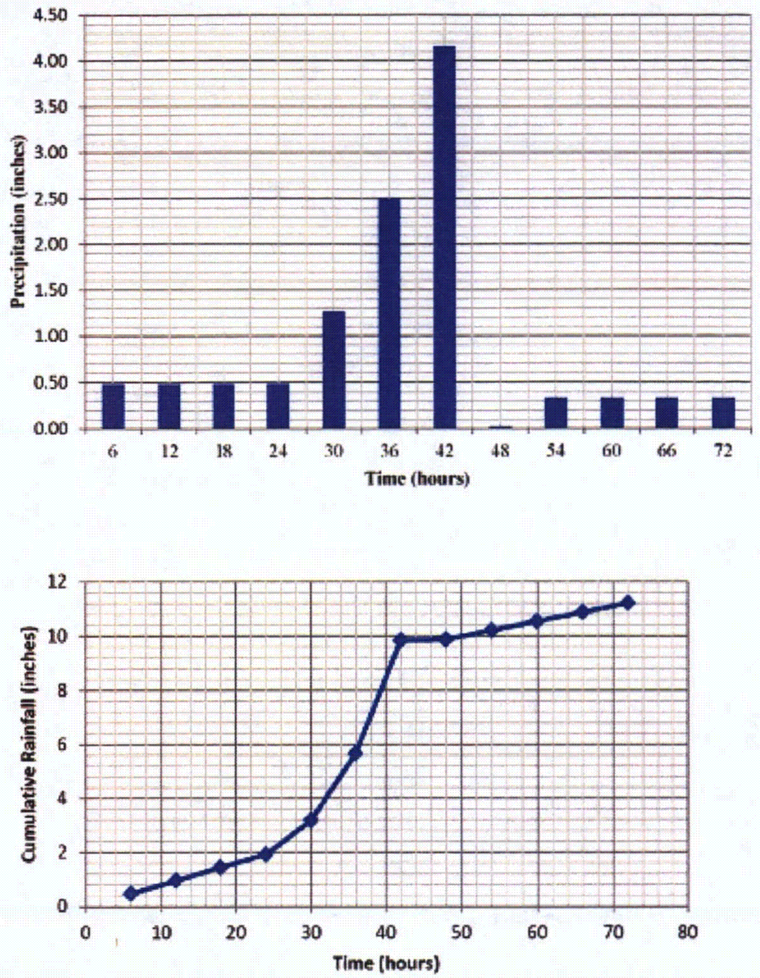

NOTES:

1. INCLUDES MULTIPLE CASES IN SUPPORT OF POTENTIAL HHA CASES TO FOLLOW.
2. INCLUDES LIMITED SENSITIVITY ANALYSIS.

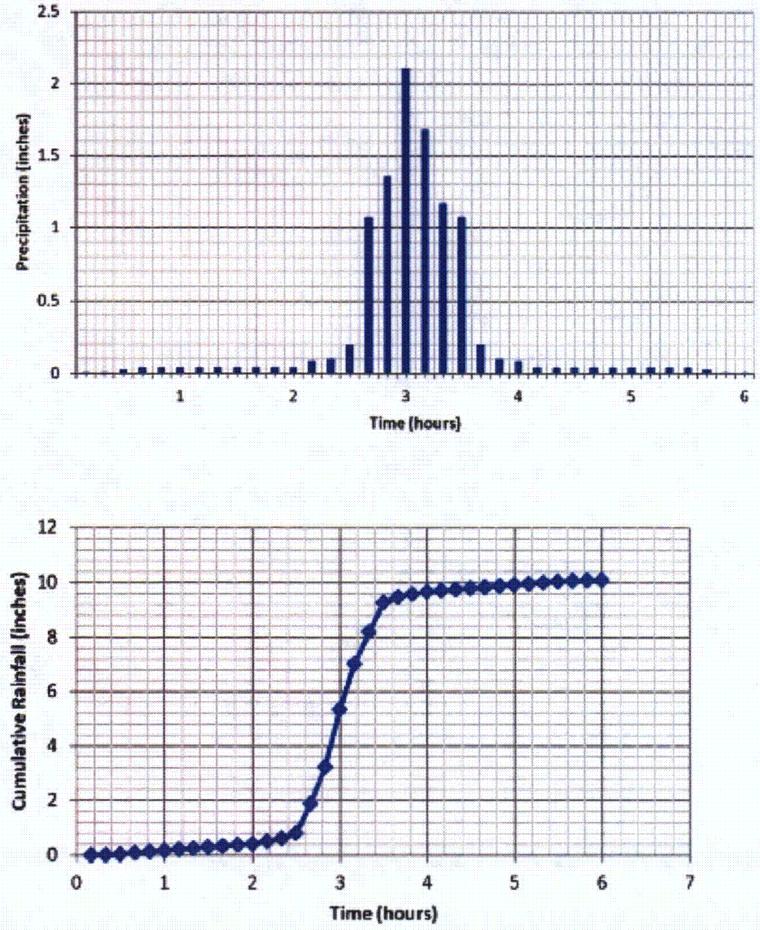
**FIGURE 3-5**

**HHA DIAGRAM FOR ANALYSIS OF FLOODING IN RIVERS AND STREAMS**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**




WINTERS WASH PMP DISCRETE AND CUMULATIVE HYETOGRAPHS

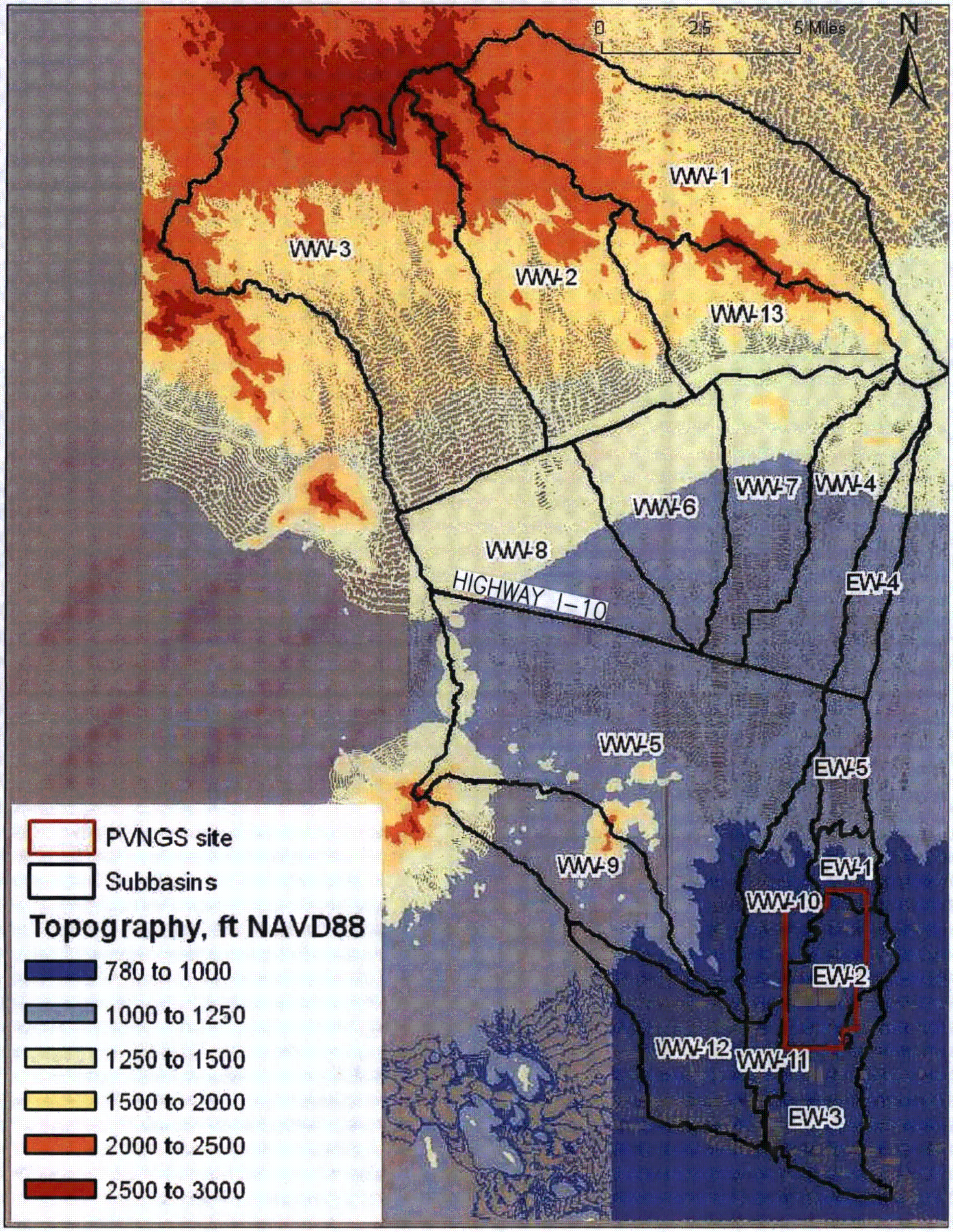



EAST WASH PMP DISCRETE AND CUMULATIVE HYETOGRAPHS

**FIGURE 3-6**

**PMP HYETOGRAPHS FOR WINTERS WASH AND EAST WASH WATERSHEDS**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**




NOTE:

WW = WINTERS WASH

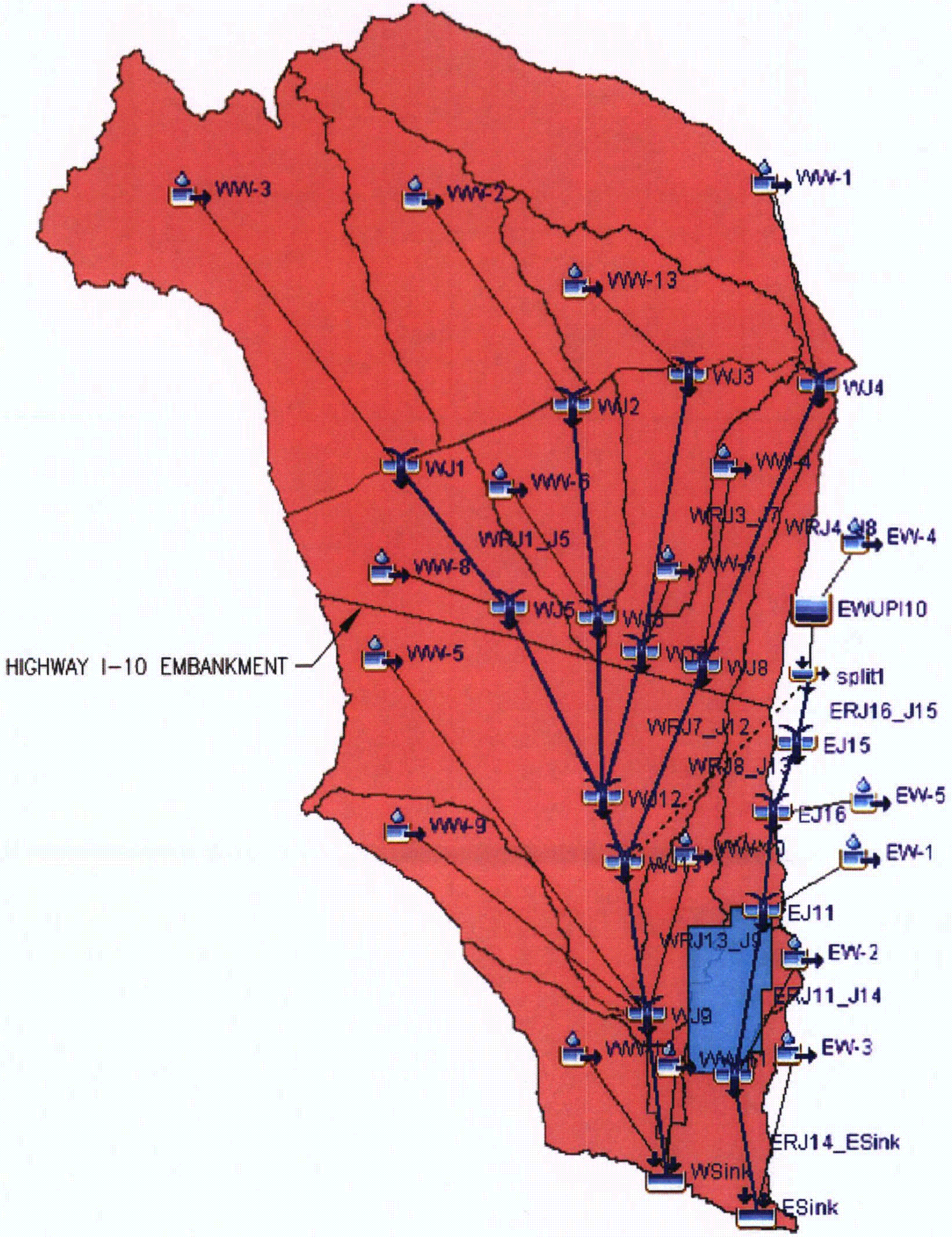
EW = EAST WASH

FIGURE 3-7

WINTERS WASH AND EAST WASH SUB-BASINS

PALO VERDE NUCLEAR GENERATING STATION

FLOOD HAZARD REEVALUATION REPORT




LEGEND:

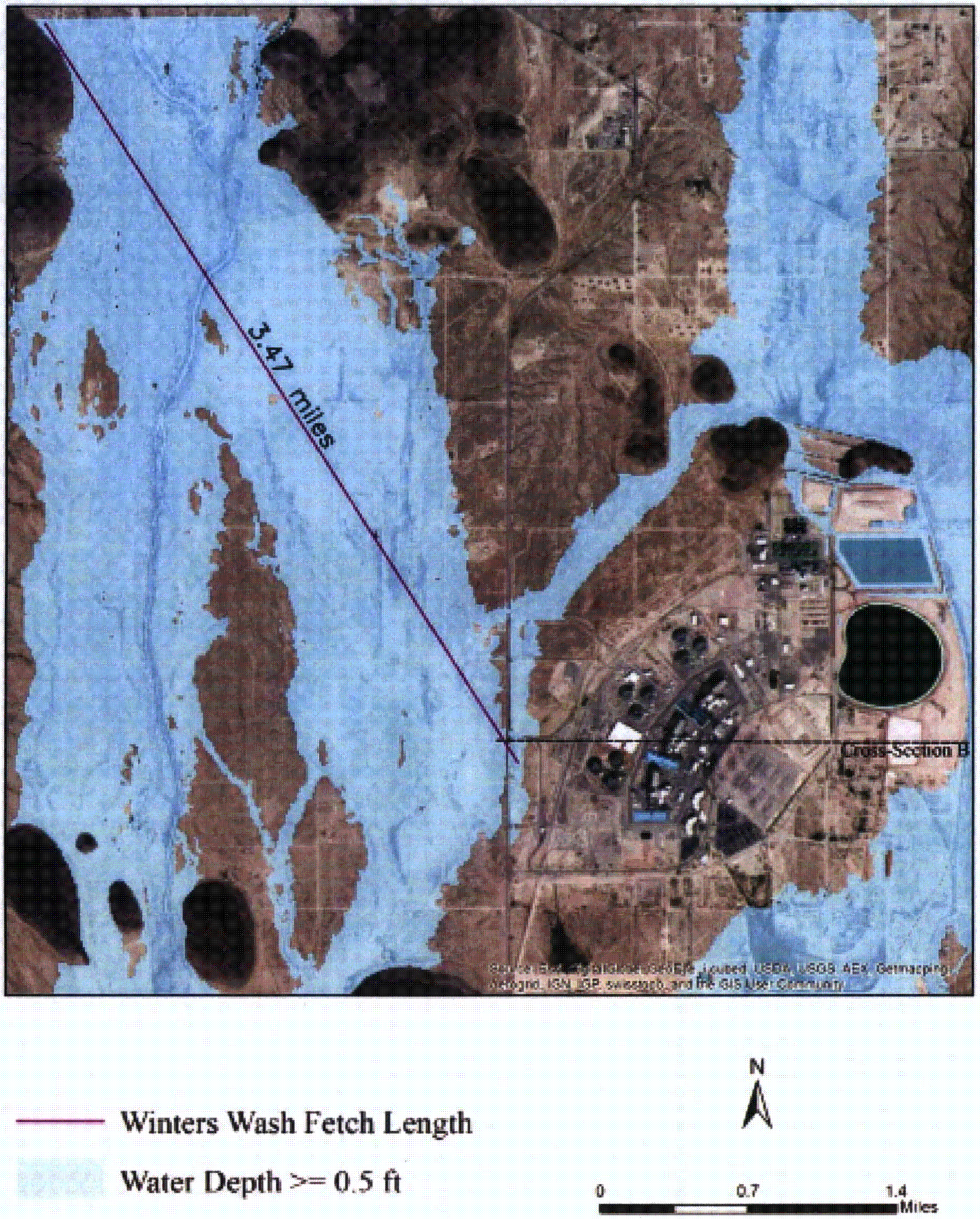
| | |
|---|---|
| WW-1 | SUBBASIN |
| WJ4 | JUNCTION |
| EWUPI10 | STORAGE |
| split1 | FLOW SPLIT |
| ESink | SINK |
| | REACH |
| | PVNGS SITE |

NOT TO SCALE

**FIGURE 3-8**

**INITIAL ANALYSIS**
**HEC – HMS MODEL FOR EAST WASH AND WINTERS WASH WATERSHEDS**

**PALO VERDE NUCLEAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**




REFERENCE:

Background Image: ESRI, 2013c, Environmental Systems Research Institute (ESRI), "ARCGIS IMAGERY,"

WEBSITE: <http://www.arcgis.com/home/item.htm

Date of publication: January 16, 2012, date accessed: June 21, 2013

**FIGURE 3-9**

**FETCH LOCATIONS FOR FLOODING IN WINTERS WASH**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

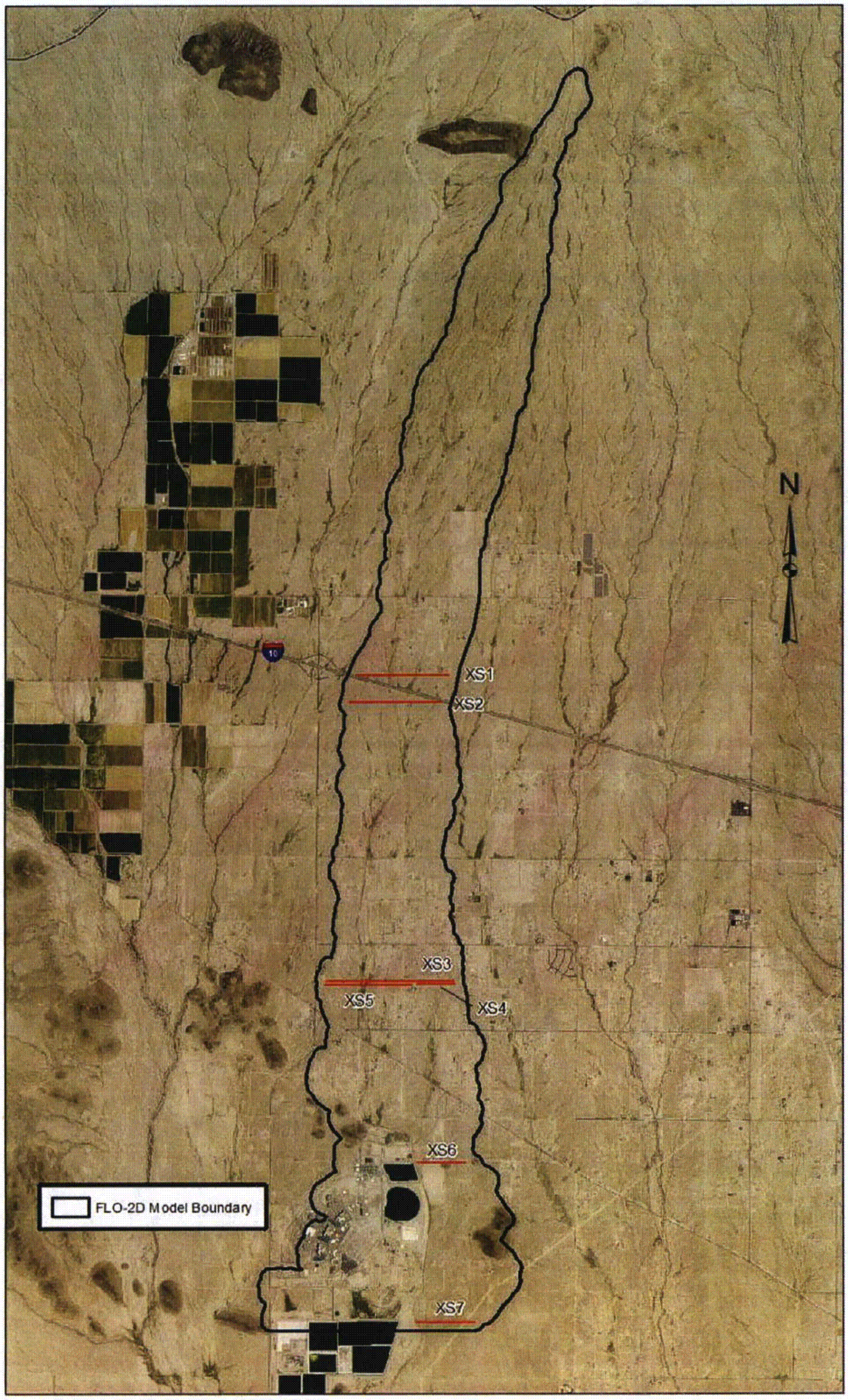



**FIGURE 3-10**

**EAST WASH LOCATION MAP AND MODEL EXTENT**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

REFERENCE:

Model Boundary is based on the East Wash Watershed delineation.
Image Source: 2013 U.S. Department of Agricultural, National Agricultural Inventory Project.

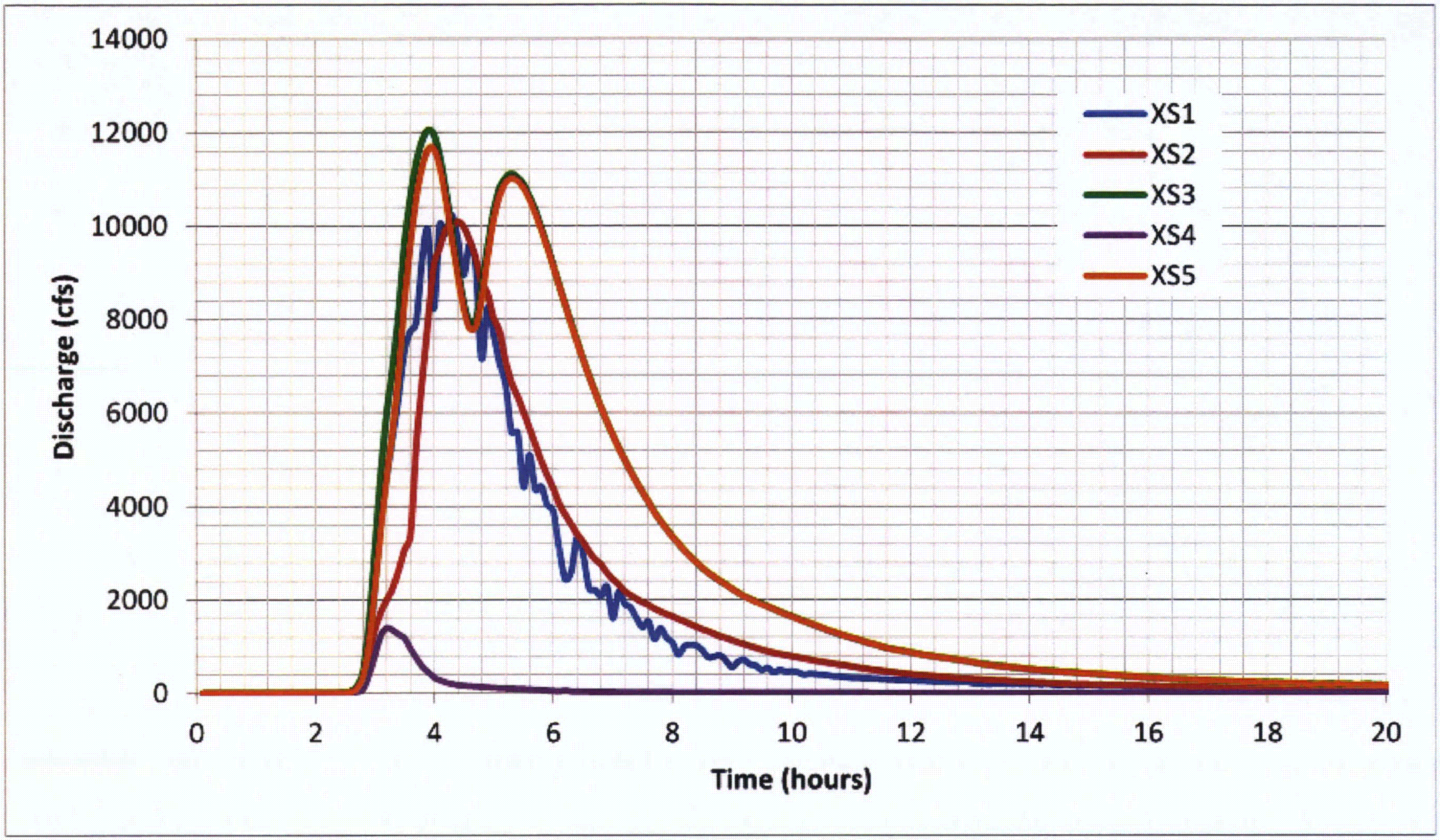



**FIGURE 3-11**

**EAST WASH FLOODPLAIN CROSS-SECTION HYDROGRAPHS – REFINED PMF (100-FT GRID ELEMENT MODEL)**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

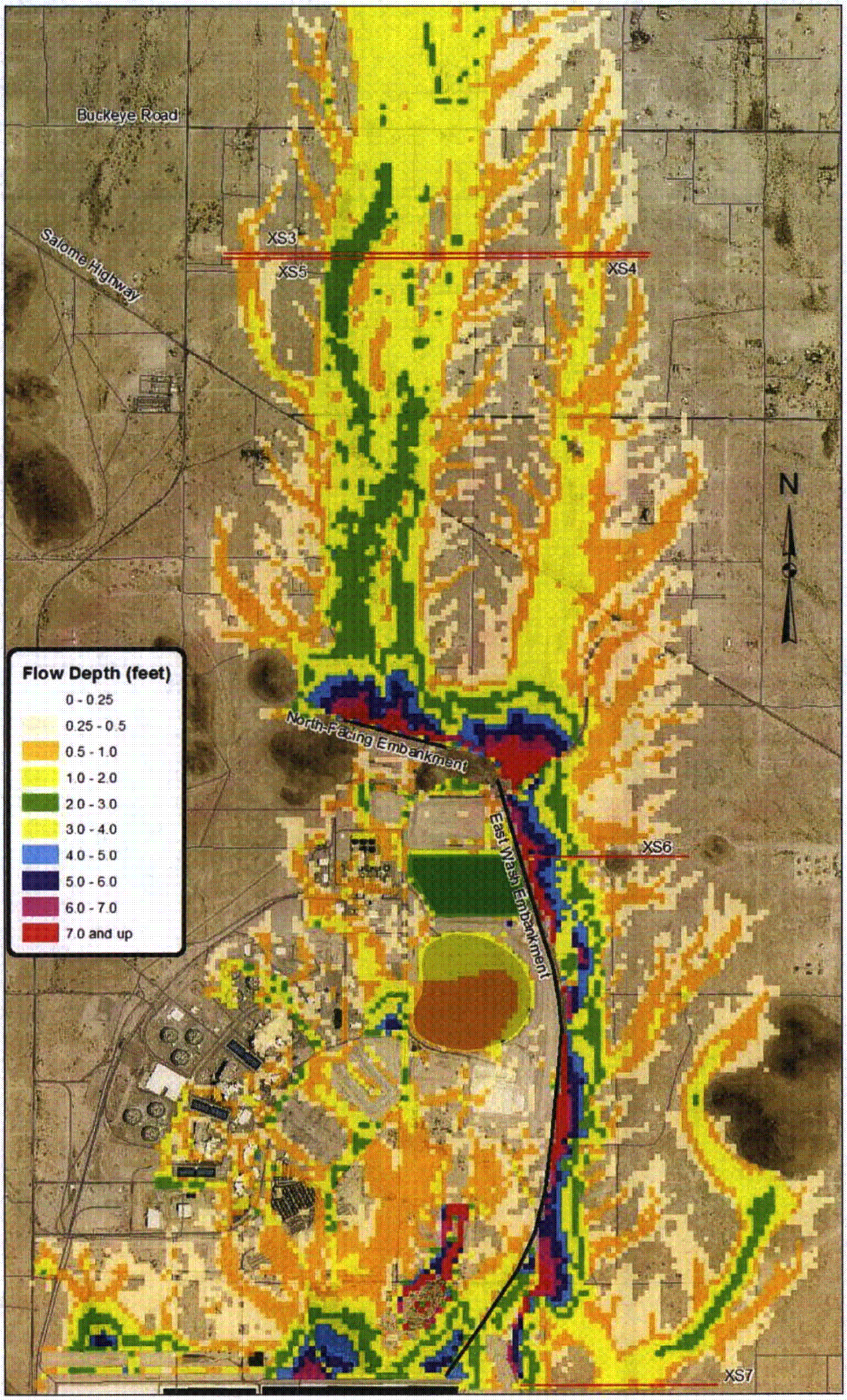



**FIGURE 3-12**

**EAST WASH FLOW DEPTHS – REFINED PMF (100-FT GRID ELEMENT MODEL)**

**REFERENCE:**

Flow depths in power block area are a result of direct rainfall only.

Image Source: 2013 U.S. Department of Agricultural, National Agricultural Inventory Project.

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

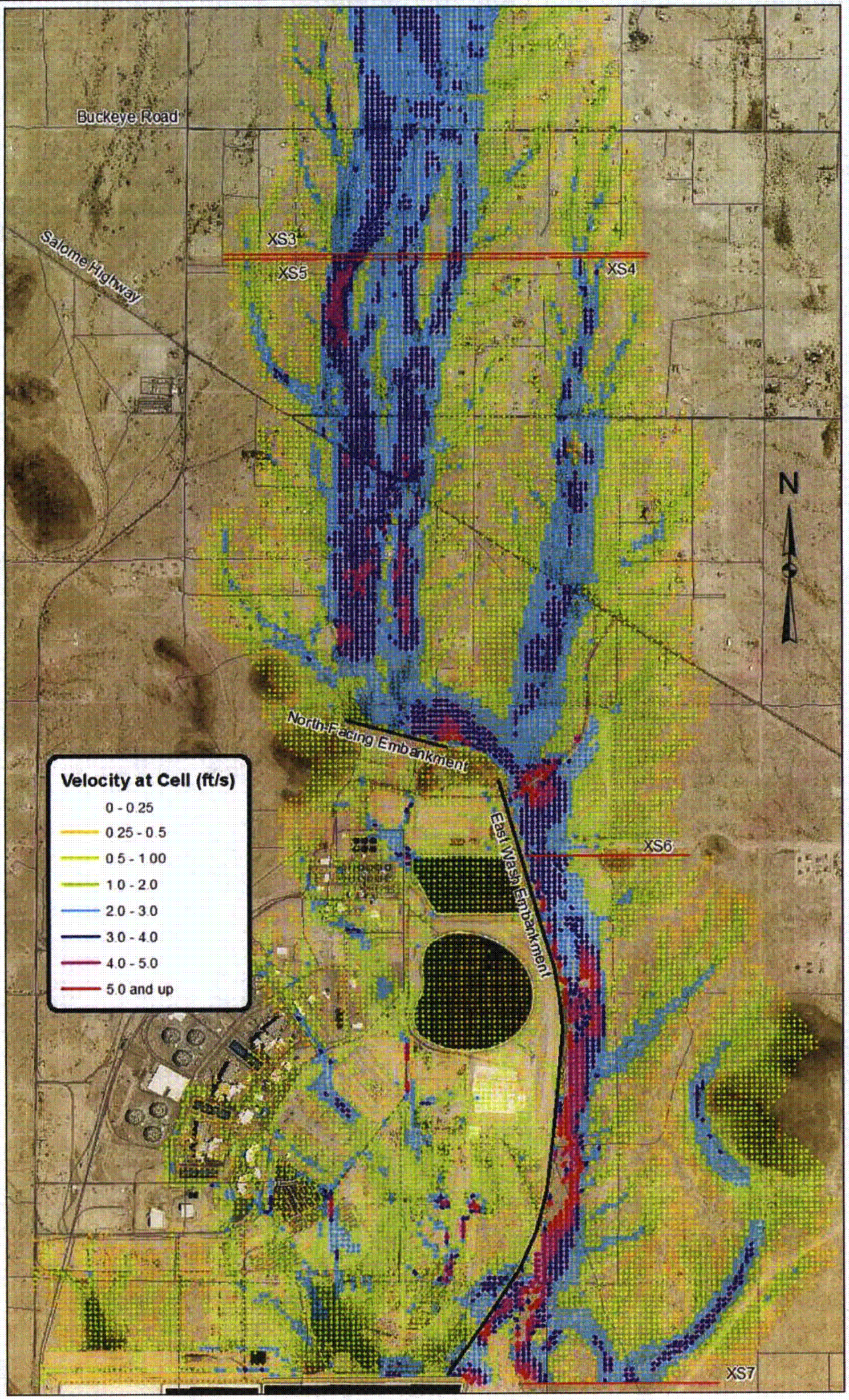



**FIGURE 3-13**

**EAST WASH FLOW VELOCITIES – REFINED PMF (100-FT GRID ELEMENT MODEL)**

REFERENCE:

Flow velocities in powerblock area are a result of direct rainfall only
Image Source: 2013 U.S. Department of Agricultural, National Agricultural Inventory Project.

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

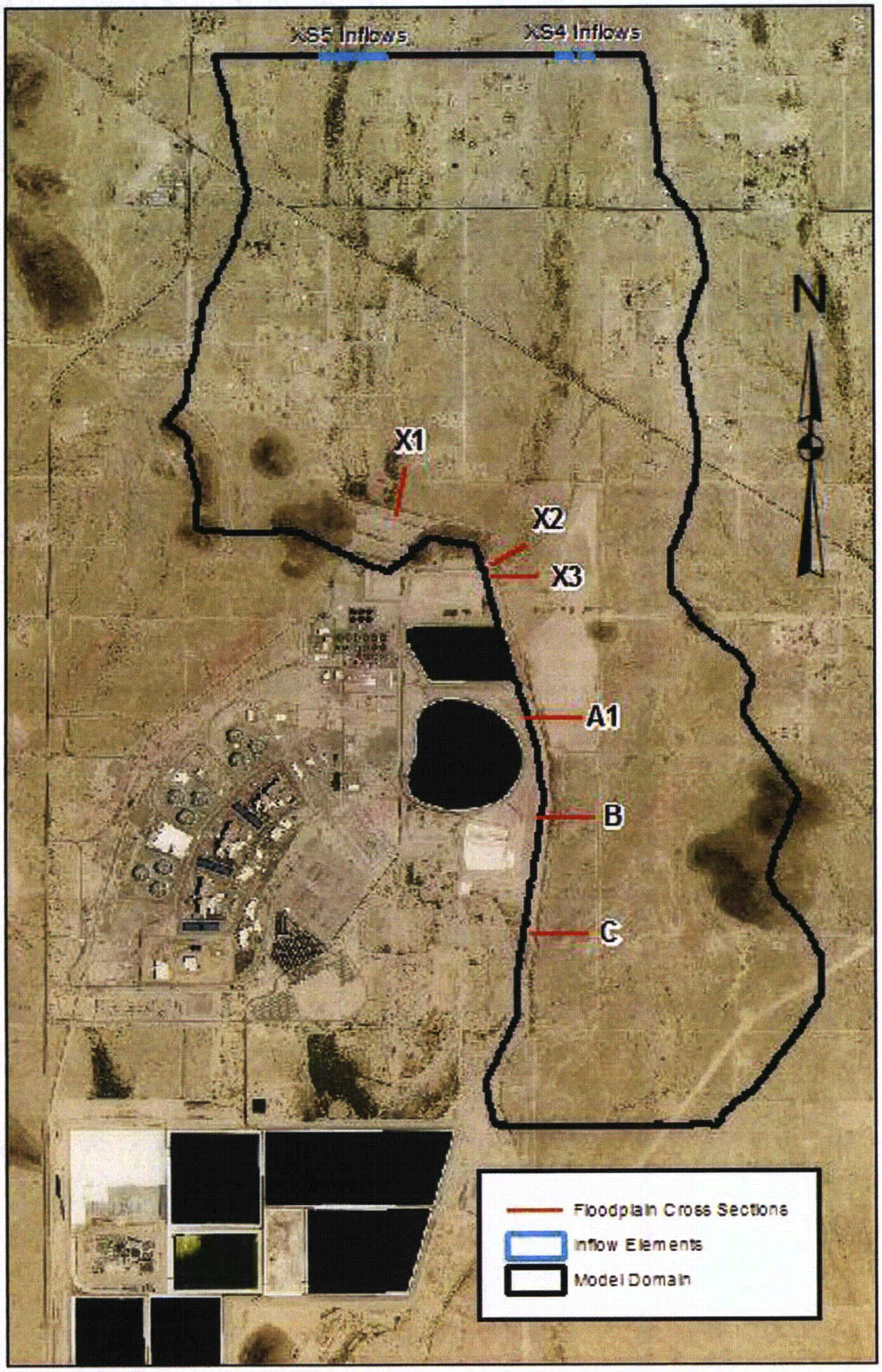



FIGURE 3-14

EAST WASH FLOODPLAIN CROSS SECTIONS

REFERENCE:

Image Source: 2013 U.S. Department of Agricultural, National Agricultural Inventory Project.

PALO VERDE

PALO VERDE NUCLEAR GENERATING STATION
FLOOD HAZARD REEVALUATION REPORT

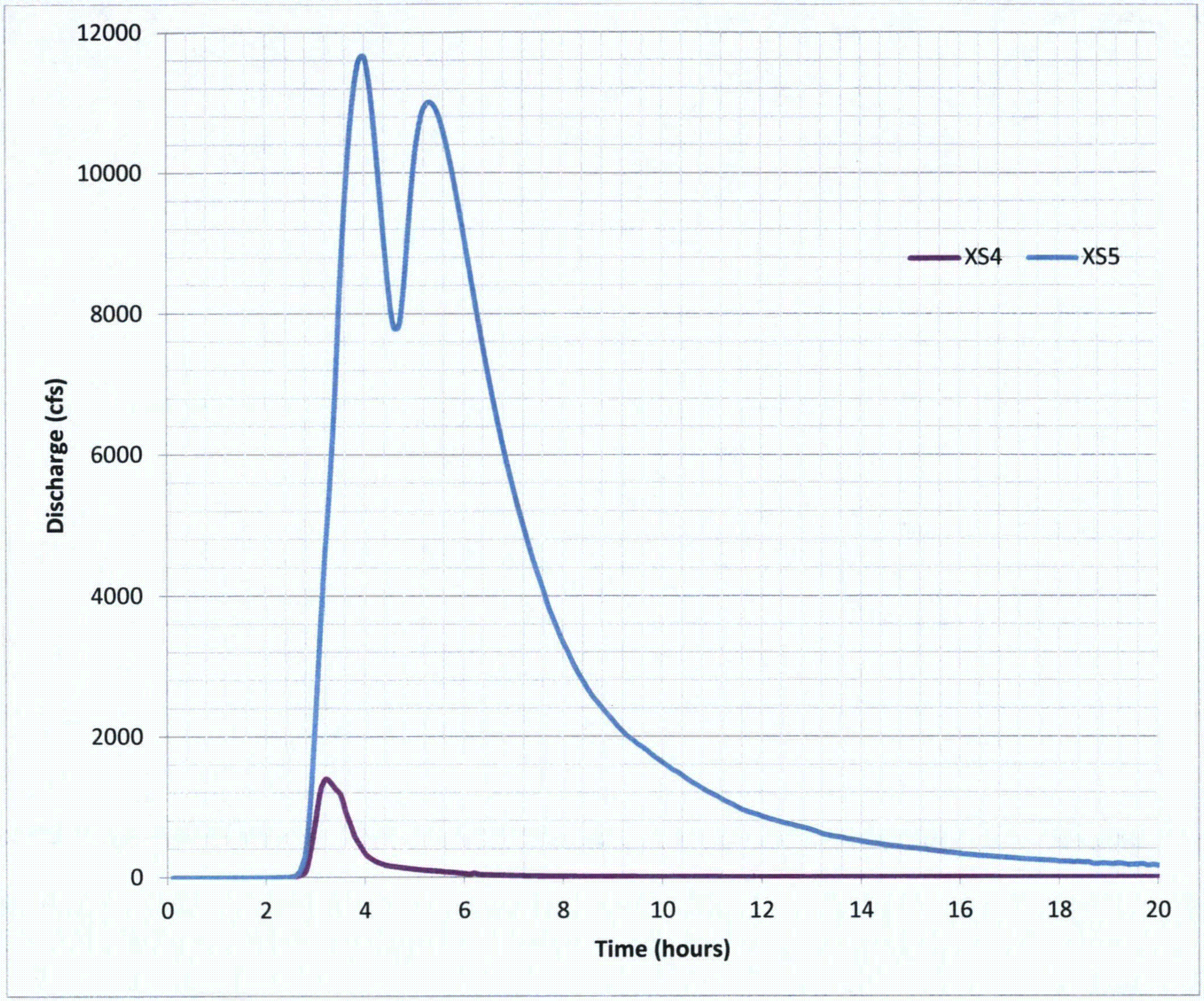



**FIGURE 3-15**

**EAST WASH WATERSHED INFLOW HYDROGRAPHS (REFINED ANALYSIS)**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

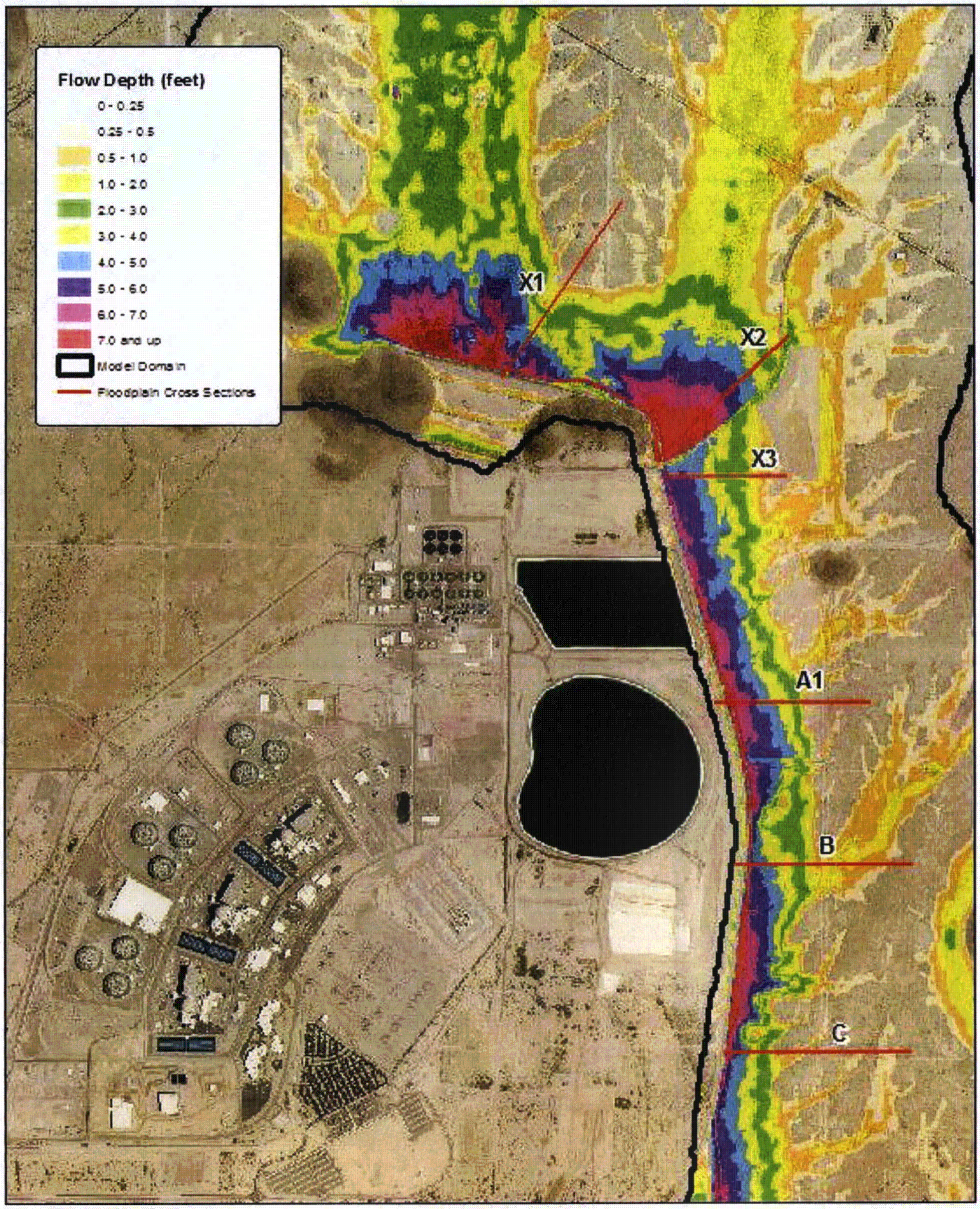



**FIGURE 3-16**

**EAST WASH WATER DEPTHS – REFINED PMF**
**(25-FT GRID ELEMENT MODEL)**

REFERENCE:

Image Source 2013 U.S. Department of Agriculture, National Agricultural Inventory Project * Maximum water surface elevation does not include effects from wind-wave action.

PALO VERDE

**PALO VERDE NUCLEAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**

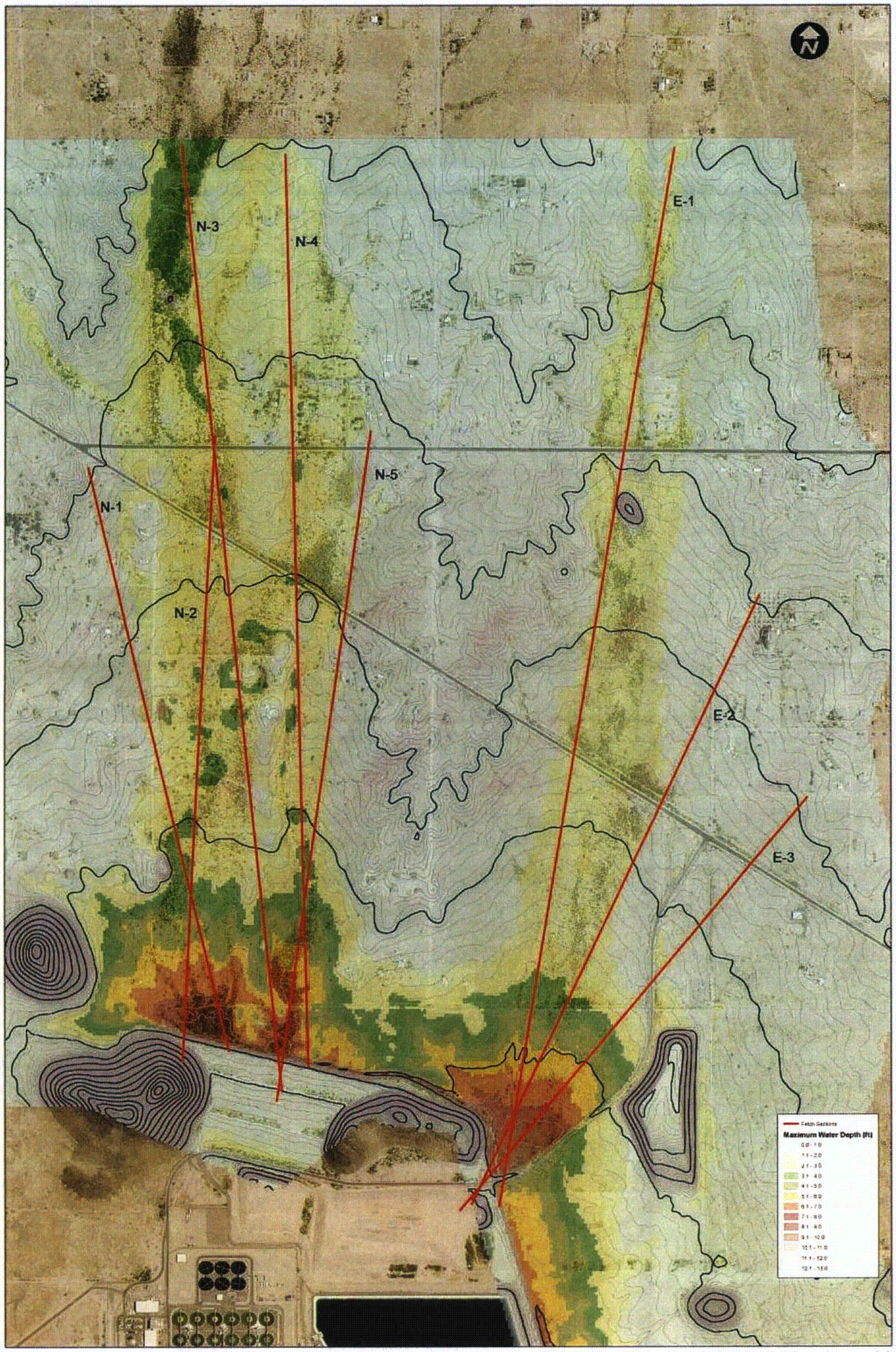



**FIGURE 3-17**

**EAST WASH POTENTIAL FETCHES**
**(REFINED ANALYSIS)**

REFERENCE:

Image Source: 2013 U.S. Department of Agricultural, National Agricultural Inventory Project.

**PALO VERDE NUCLEAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**

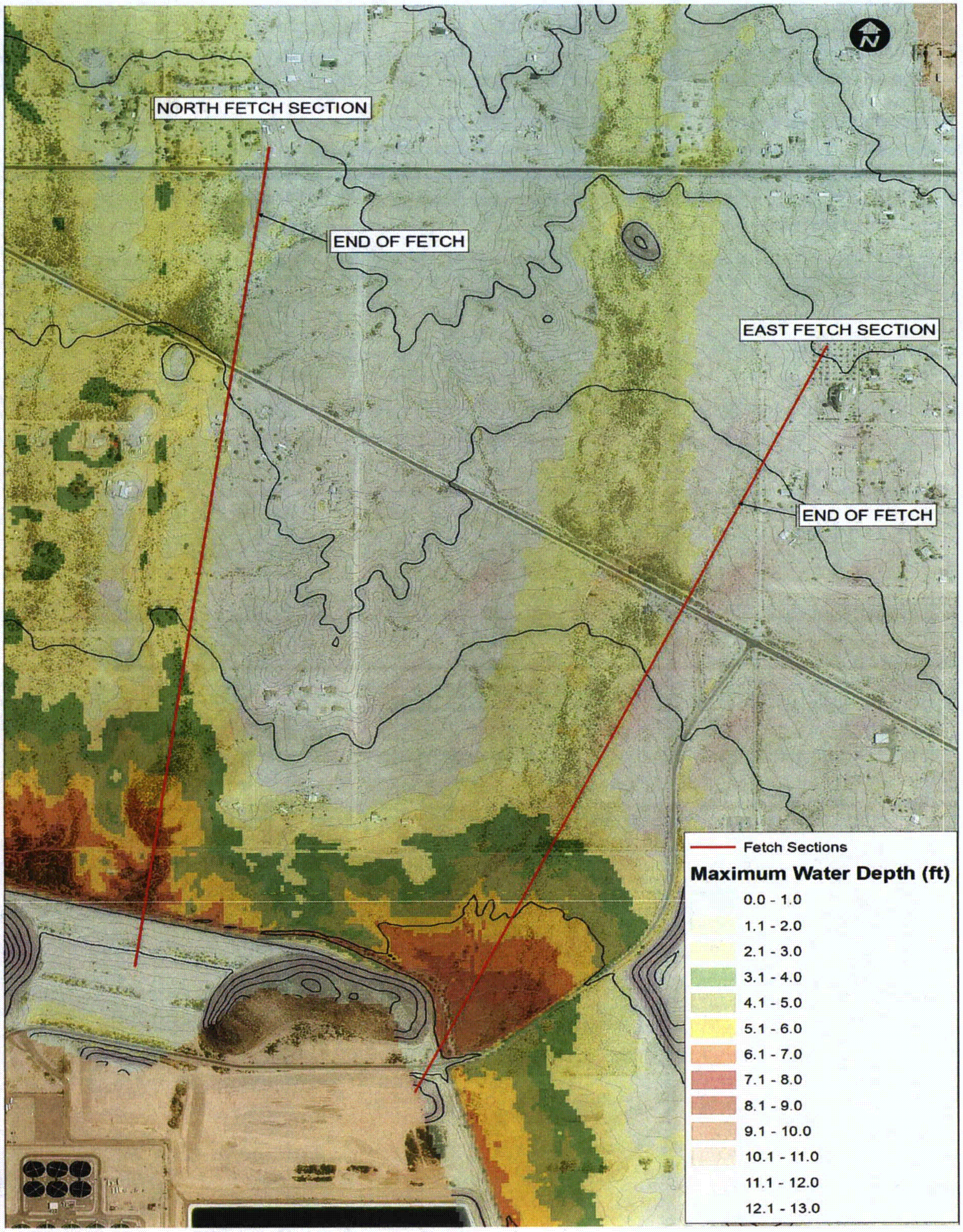



**FIGURE 3-18**

**NORTH AND EAST EMBANKMENT FETCHES**

**REFERENCE:**

Image Source: 2013 U.S. Department of Agricultural, National Agricultural Inventory Project.

**PALO VERDE NUCLEAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**

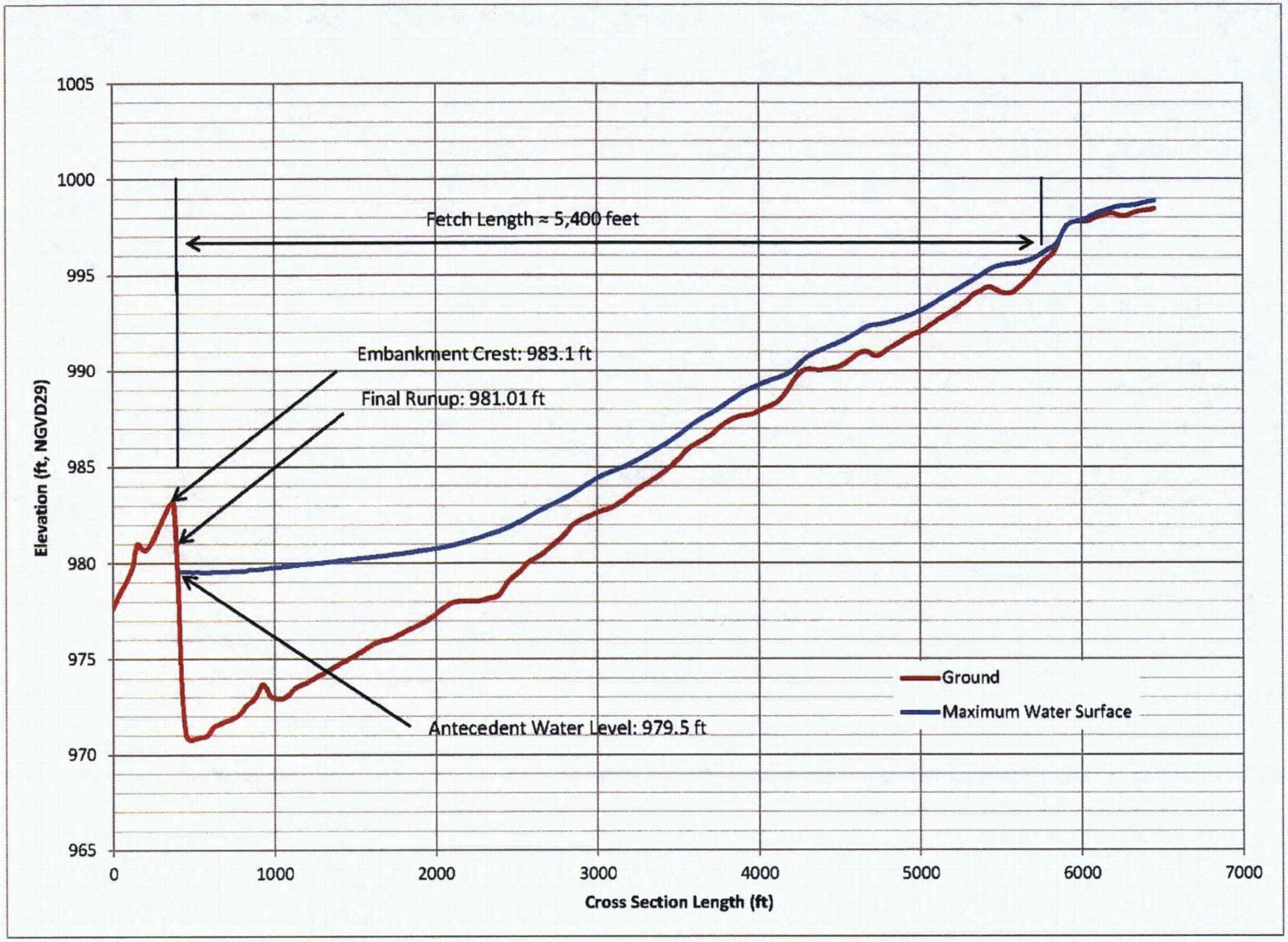



**FIGURE 3-19**

**NORTH EMBANKMENT FETCH SELECTION**
**EAST WASH (REFINED ANALYSIS)**

**PALO VERDE NUCLEAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**

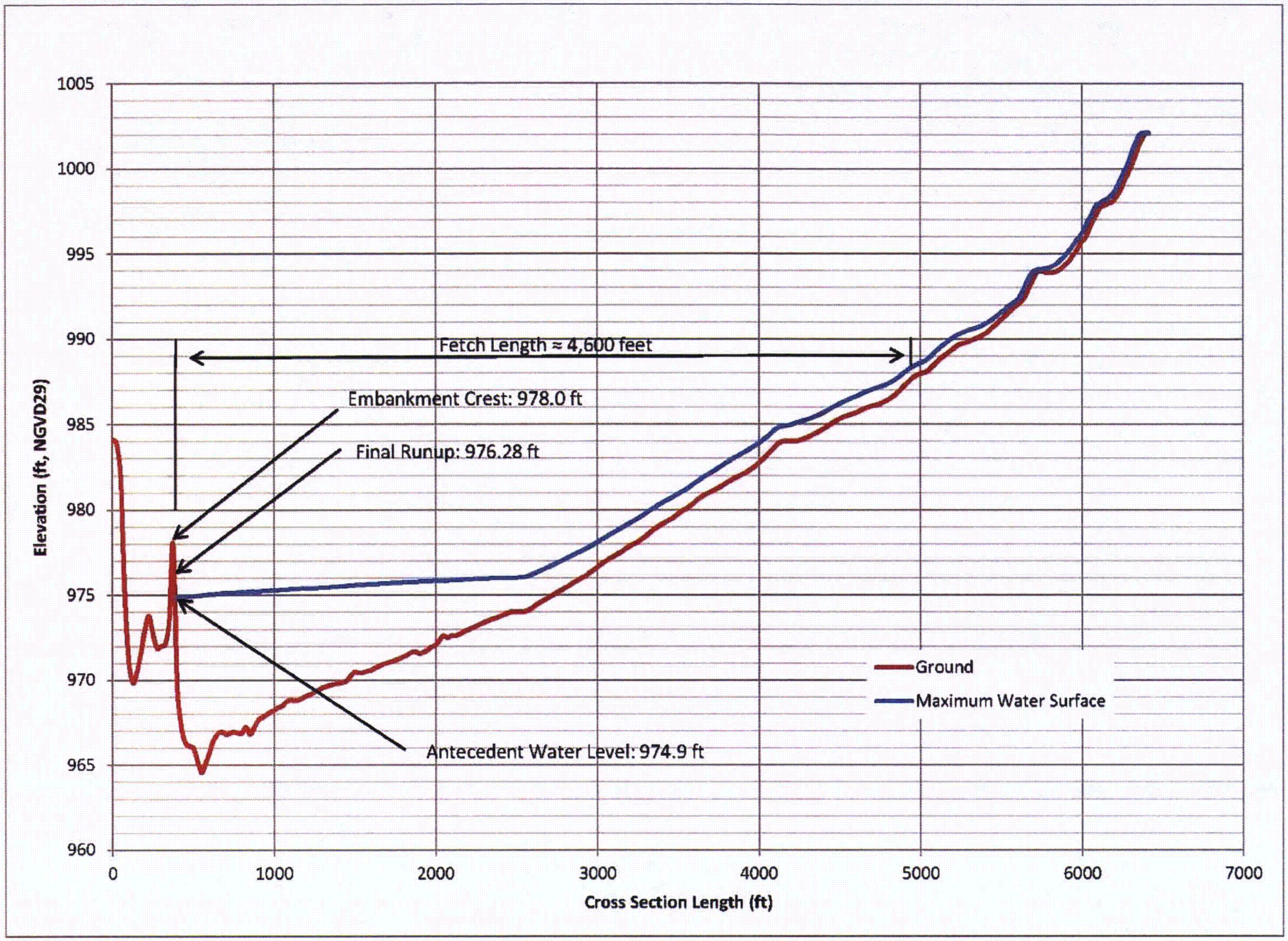
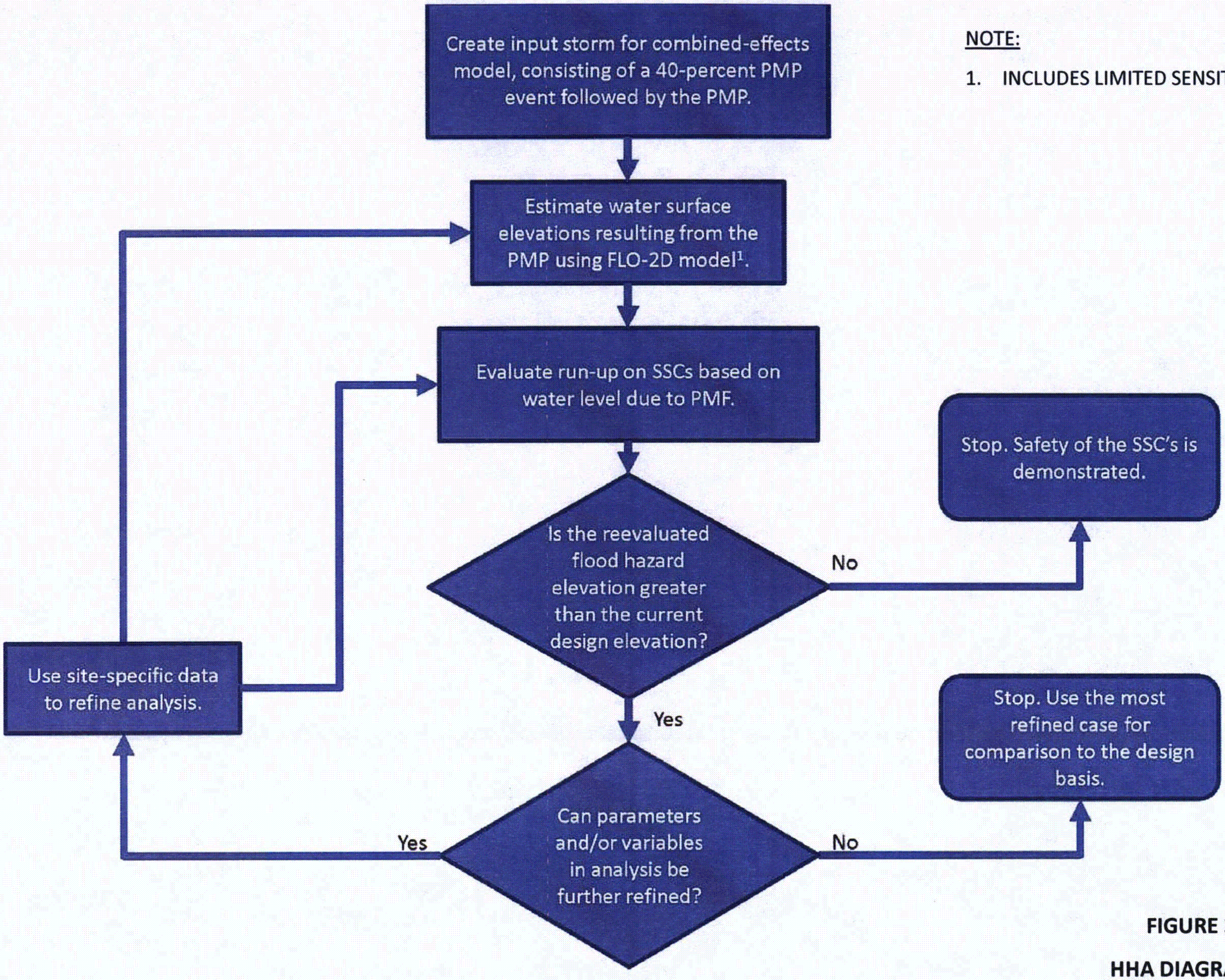



FIGURE 3-20

EAST EMBANKMENT FETCH SELECTION
EAST WASH (REFINED ANALYSIS)

PALO VERDE NUCLEAR GENERATING STATION
FLOOD HAZARD REEVALUATION REPORT




NOTE:

1. INCLUDES LIMITED SENSITIVITY ANALYSIS

**FIGURE 3-21**

**HHA DIAGRAM FOR COMBINED EFFECTS FLOODING ANALYSIS**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

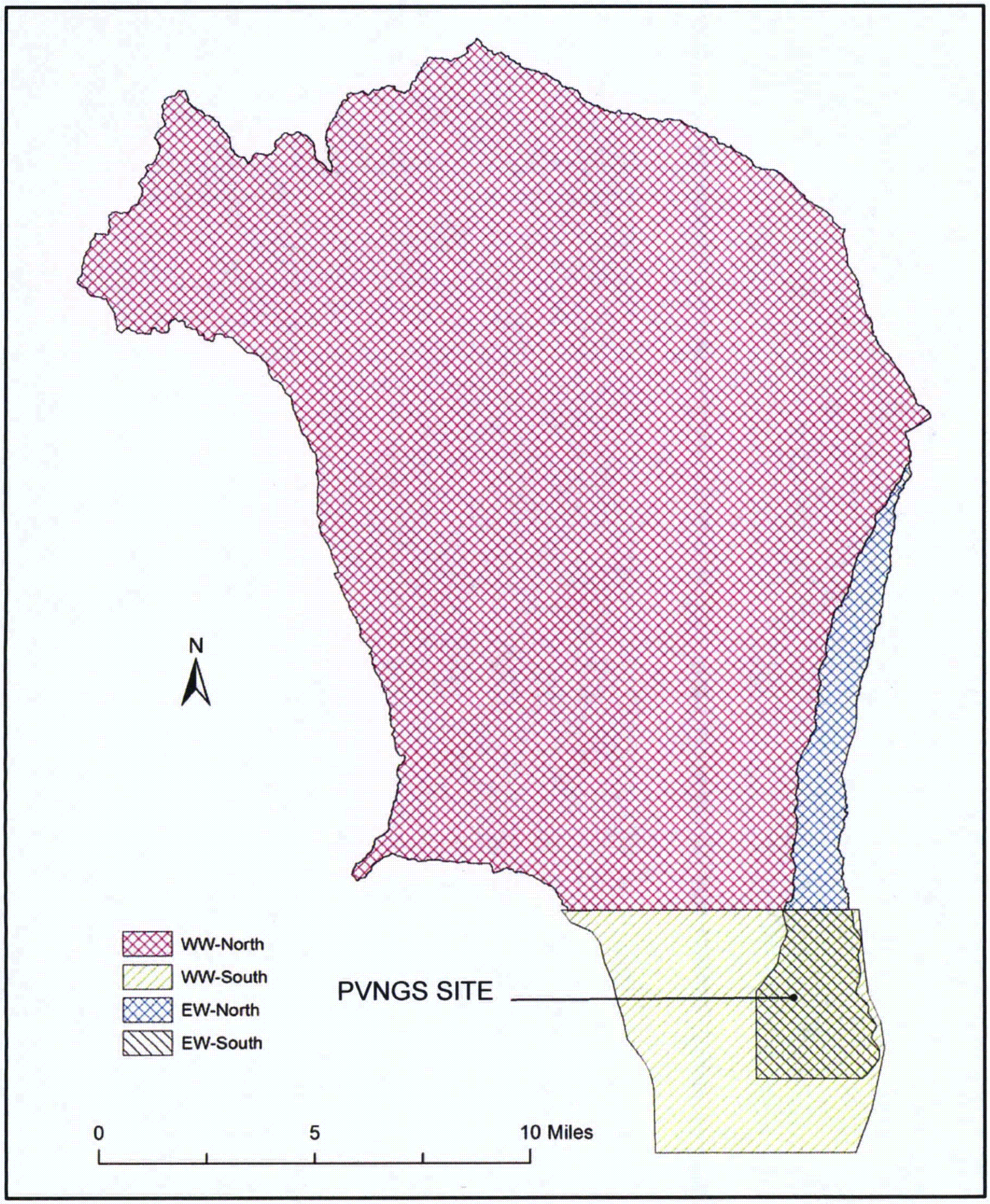
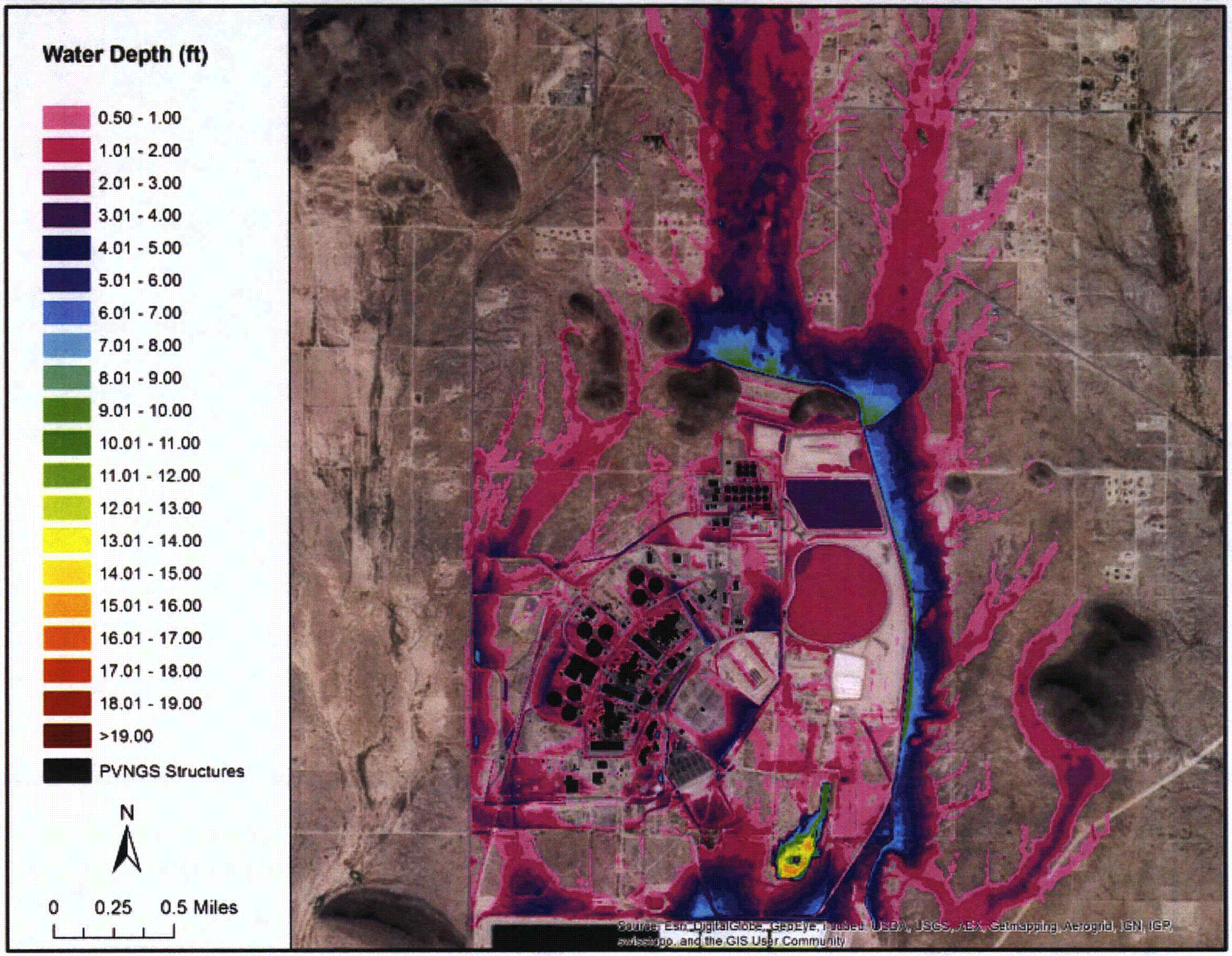



FIGURE 3-22

FLO–2D MODEL DOMAINS FOR COMBINED EFFECTS ANALYSIS

PALO VERDE NUCLEAR GENERATING STATION
FLOOD HAZARD REEVALUATION REPORT




NOTES:

THE WATER DEPTHS INDICATED IN IMPOUNDED WATER BODIES DO NOT INCLUDE THE INITIAL WATER DEPTHS. THE ILLUSTRATED FLOOD DEPTHS AT THE POWERBLOCK ARE DUE TO PMP AT THE PVNGS SITE, NOT DUE TO FLOODWATER FROM THE WASHES

**FIGURE 3-23**

**MAXIMUM COMBINED EFFECTS FLOOD DEPTH (FT) FOR CASE 3**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

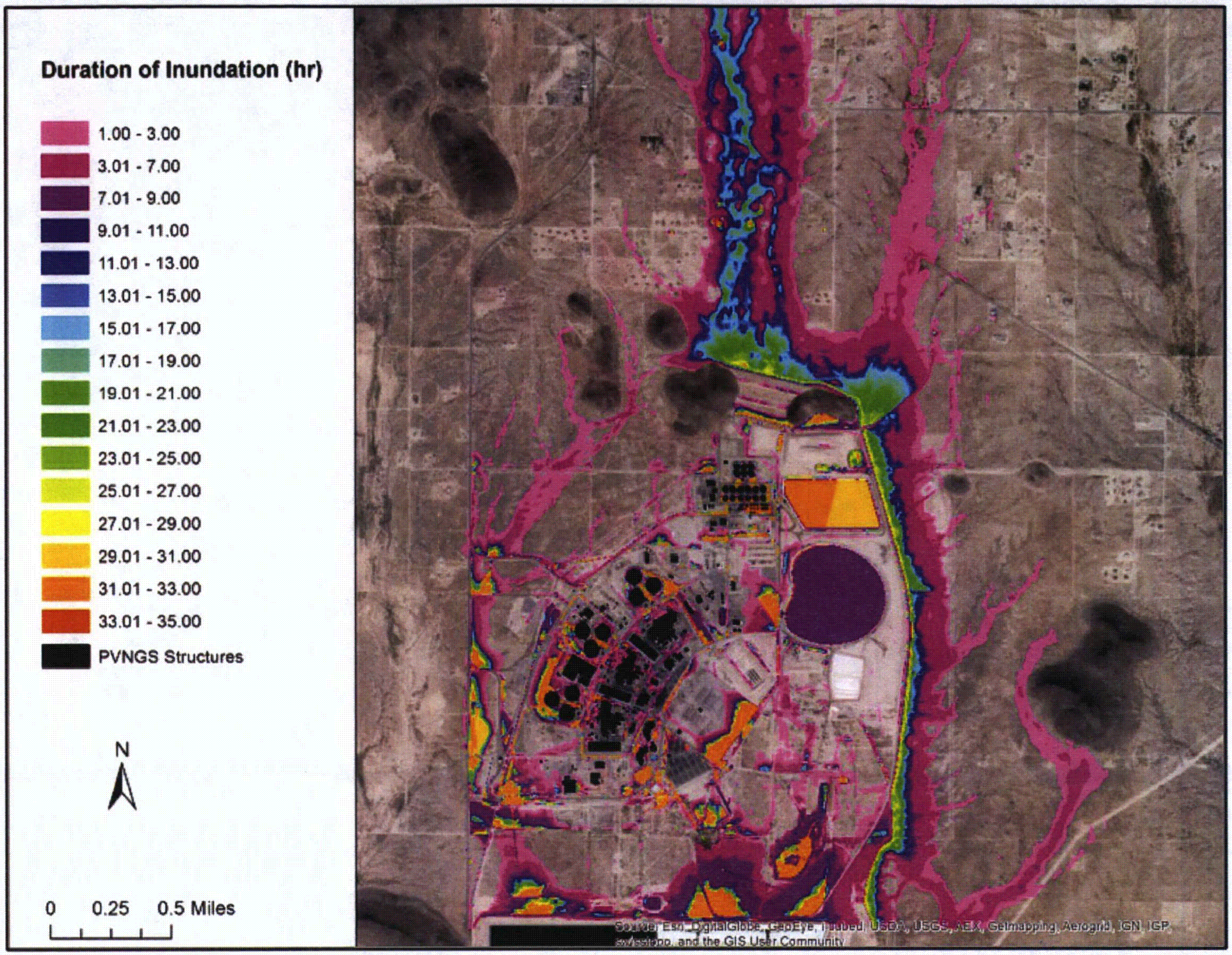
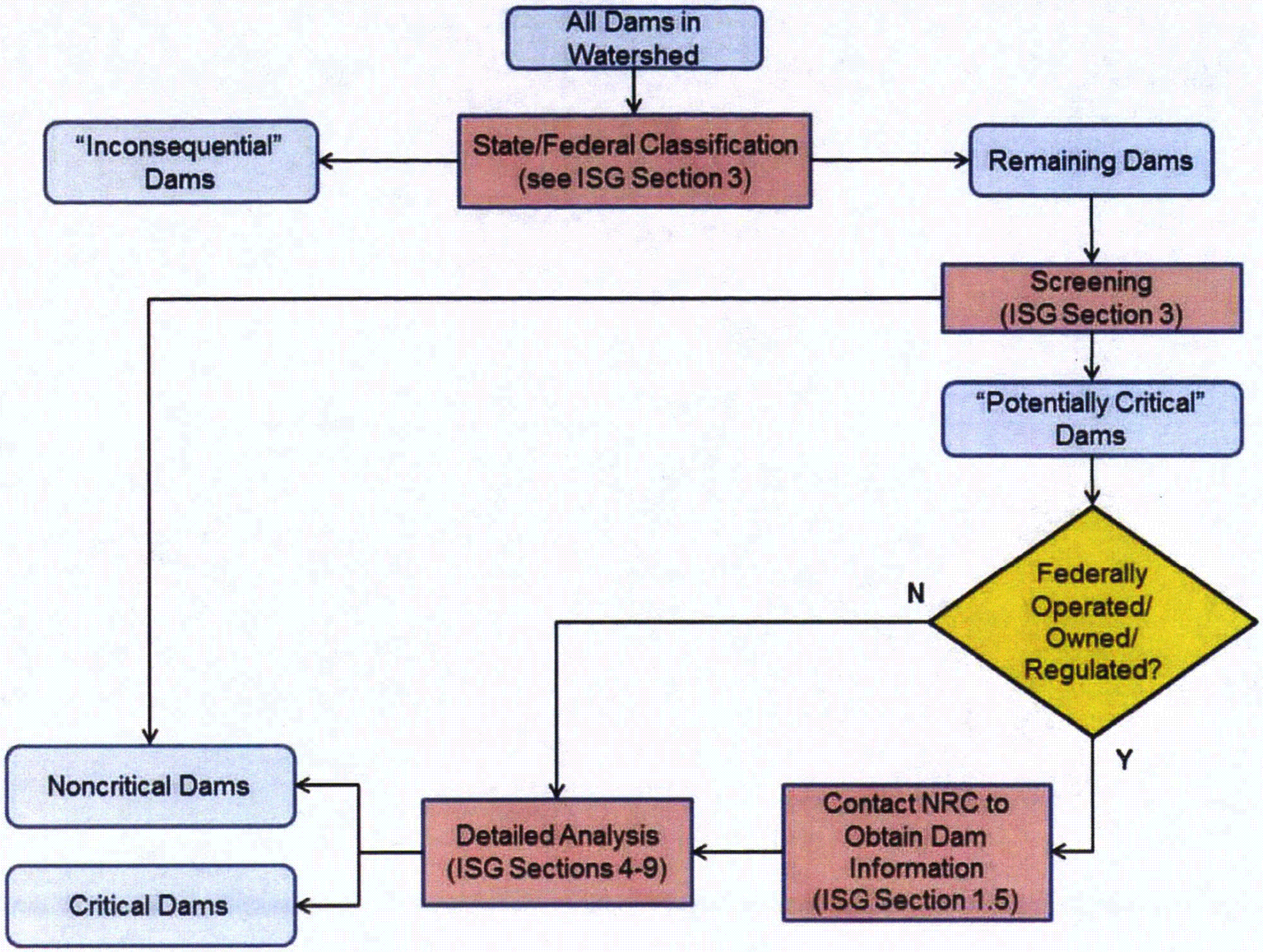



**FIGURE 3-24**

**DURATION OF COMBINED EFFECTS FLOODING (HOURS) FOR CASE 3**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**




LEGEND:

ISG – INTERIM STAFF GUIDANCE

REFERENCE:

Background Image: ESRI, 2013e, United States Nuclear Regulatory Commission (NRC), "GUIDNCE FOR ASSESSMENT OF FLOODING HAZARDS DUE TO DAM FAILURE," JLD-ISG-2013-01, NRC INTERIM STAFF GUIDANCE (ML13151A153), WASHINTON, DC, REVISION 0, JULY 29, 2013.

**FIGURE 3-25**

**ISG–2013-01 DIAGRAM FOR DETERMINING LEVELS OF ANALYSIS FOR DAM BREAK EVALUATION**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

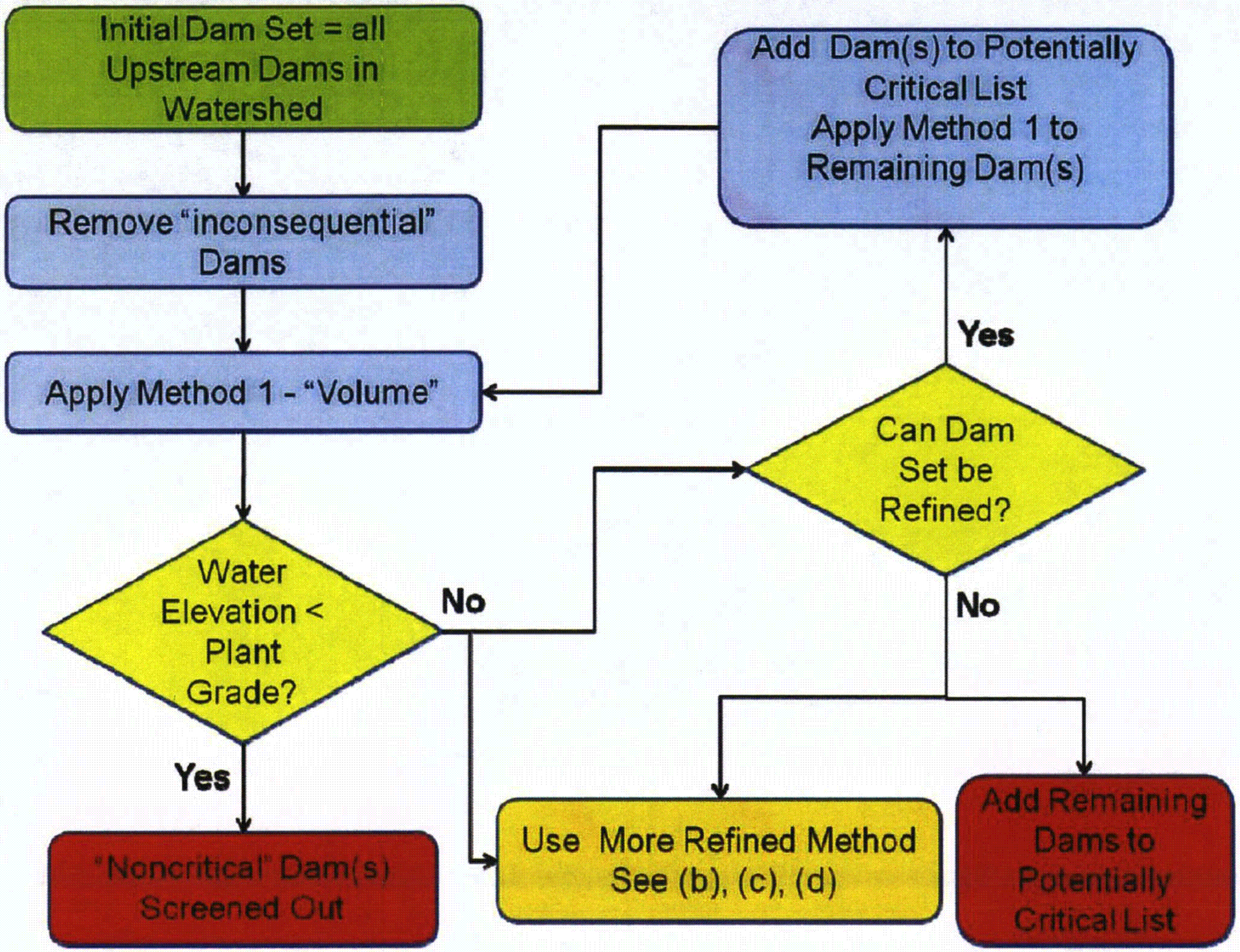


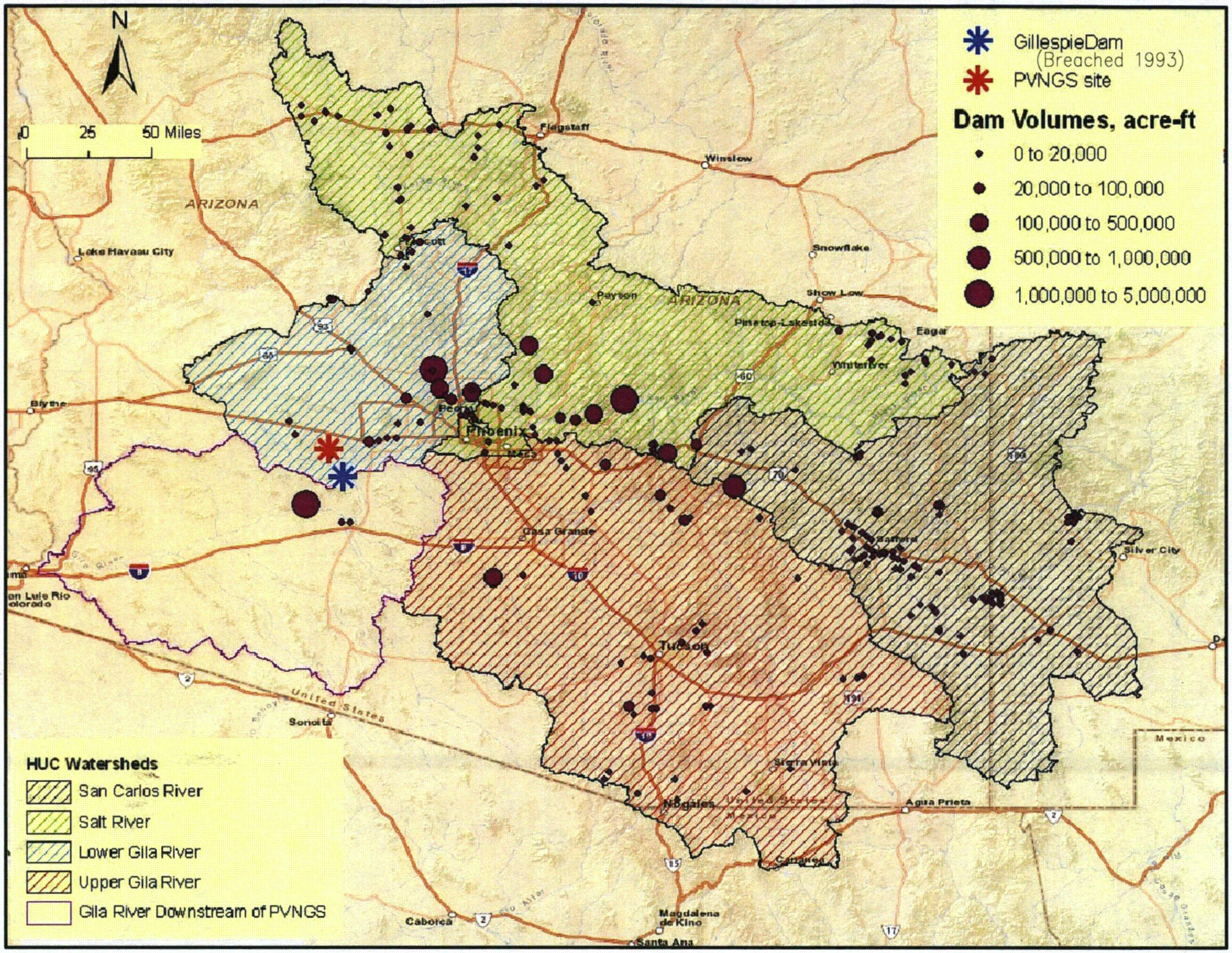

**FIGURE 3-26**

**ISG–2013-01 DIAGRAM FOR ANALYSIS OF DAM BREACHES AND FAILURES USING THE VOLUME METHOD**

**PALO VERDE NUCLEAR GENERATING STATION FLOOD HAZARD REEVALUATION REPORT**

REFERENCE:

Background Image: ESRI, 2013e, United States Nuclear Regulatory Commission (NRC), "GUIDNCE FOR ASSESSMENT OF FLOODING HAZARDS DUE TO DAM FAILURE," JLD-ISG-2013-01, NRC INTERIM STAFF GUIDANCE (ML13151A153), WASHINTON, DC, REVISION 0, JULY 29, 2013.





REFERENCES:

1. ESR, 201c, ENVIRONMENTAL SYSTEM RESEARCH INSTITUTE (ESRI), "ARCGIS IMAGERY," WEBSITE <http://arcgis.com/home/item.htm, DATE OF PUBLICATION: JANUARY 16, 2012, DATE ACCESSED: JUNE 21, 2013.
2. USGS, 2013b, UNITED STATES GEOLOGIGAL SURVEY (USGS), "USGS WATER DATA FOR THE NATION," WEBSITE: <http://waterdata.usgs.gov/nwis>, DATE ACCESSED: OCTOBER 9, 2013.
3. USGS, 2013, UNITED STATES ARMY CORPS OF ENGINEERS (USACE), "NATIONAL INVENTORY OF DAMS," WEBSITE: <http://geo.usace.army.mil/pgis/> , DATE ACCESSED: JUNE 11, 2013


**FIGURE 3-27**

**LOCATION OF DAMS NEAR THE SITE**

**PALO VERDE NUCLEAR GENERATING STATION**
**FLOOD HAZARD REEVALUATION REPORT**

# APPENDIX C

# PROBABLE MAXIMUM PRECIPITATION IN ARIZONA

## APPENDIX C

## PROBABLE MAXIMUM PRECIPITATION IN ARIZONA

The intent of this document is to justify the use of the Probable Maximum Precipitation (PMP) values used to calculate the Local Intense Precipitation at the Palo Verde Nuclear Generating Station (PVNGS) in response to the Nuclear Regulatory Commission (NRC) letter requesting information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) (NRC, 2012) and conformance with NUREG/CR-7046 (NRC, 2011). The updated Arizona 6-hour and 72-hour PMP values and distributions are also utilized in calculating the potential flood hazards from East Wash and Winters Wash, respectively. The potential flood hazard impacts of two washes are evaluated in detail because the site is located in both washes watersheds.

The methodology used in Arizona for estimating the PMP for the design of dams was updated by the state through the Arizona Department of Water Resources (ADWR). An update was necessary because the previous methodology utilized the procedures described in *Hydrological Report No. 49* (HMR 49) prepared by the National Weather Service (NWS) for the US Army Corps of Engineers (NWS, 1977). At the time, HMR 49 was prepared to estimate the PMP for locations in the Colorado River and Great Basin Drainages (part or all of Arizona, Utah, New Mexico, Colorado, Nevada, and Wyoming) as well as all of California. The methodology was used to calculate the PMP for areas up to 5,000 sq mi and durations up to 72-hours. HMR 49 is an outdated publication and many of the locations are already covered by other HMRs. Presently, the PMP calculations for California have been updated by HMR 59 in 1999 and there is a state-wide PMP update for Utah. The updates occurred as a result of additional storm information and procedures that are physically based. The updates include a narrower domain pertaining to location, similar orographic, topographic, and meteorologic characteristics.

The report *Probable Maximum Precipitation Study for Arizona* (AWA, 2013) is the successor document to HMR 49 for the State of Arizona. The most relevant reason for the update is increasing the period of the data base originally used in HMR 49 and the document is old. Only a few storms were evaluated in preparing HMR 49. Over the past forty years more storms have occurred and the state-of-the practice in evaluating extreme storm events has changed making the new analysis more appropriate and up-to-date (AWA, 2008).

Applied Weather Associates (AWA) completed the statewide PMP study for Arizona in 2013. The study was funded by the ADWR with financial support from the Flood Control District of Maricopa County, Arizona Game and Fish Department, and USDA Natural Resources Conservation Service. These agencies also provided technical input on the document. An independent Peer Review Committee (PRC) reviewed and provided feedback on the methodology at critical milestones and the final document. The PRC was comprised of Dr. Keim a climatologist and professor at Louisiana State University, Dr. Sabol a Principal and water resources engineer with Stantec in Phoenix, and Dr. Selover a climatologist and research professor at Arizona State University. The PRC published a final report in August 2013 (PRC, 2013).

The statewide study is being implemented to calculate the PMP events used in the hydrologic analysis of dams in Arizona. Maricopa County is applying the PMP tool in preparing new PMF hydrology in support of Emergency Action Plans for the following structures:

- Wickenburg Structures, Arizona
- Powerline, Vineyard, and Rittenhouse Structures, Arizona
- McMicken Dam, Arizona
- Guadalupe Flood Retarding Structure, Arizona

AWA is presently preparing a similar study for the State of Wyoming, where the USACE is on the technical review committee. They have utilized the methodology for use in evaluating dams in other states for the Federal Energy Regulatory Commission (FERC); other state dam programs, and PMP estimates at Nuclear Power Plant sites.

Individual storm events were not evaluated in HMR 49 providing a direct correlation between depth-area and area-duration (DAD envelope). Instead general ratios were used when evaluating both the area (area reduction) and duration of PMP storms based on the overall study area. The widely varying climatology and topography of the region is not conducive to using a general approach. HMR 49 uses the persisting (lowest) 12-hour dewpoint in developing the PMP. Newer studies use the average dewpoint that fits with predicting extreme rainfall events.

The procedures used in the Arizona update are required by the state for dams and recommended for other projects because the study has updated the data since 1977 to include many more rainfall events. The method evaluated 51 extreme value precipitation storm events and 91 storm centers using the Storm Precipitation and Analysis System (SPAS) in characterizing magnitude, temporal, and spatial data (DAD values, mass curves, and total storm isohyets). The use of NEXt generation RADar (NEXRAD) data (mid 1990's) has contributed significantly to the quality of the storm data being used. The procedure considers climate zone, topography and other variables. Updated dewpoint data was used to maximize the effects of moisture that is associated with the rainfall events. The study was done to develop a consistent procedure using a consistent data base for the entire state. However, the PMP tool (AWA, 2013) applies the data base to calculate site specific PMP values. The input data to the PMP tool is a shapefile of the study area. The tool then determines PMP values using a 2.5 sq mi grid to calculate the PMP for each grid in the study area. The grid values are calculated from the storm data base that are similar to the site. It then uses a weighting process to calculate the average PMP value for the specific study. The tool is used for analyzing local storms where durations analyzed vary from 1 to 6 hours.

Tropical and general storms are analyzed for durations of 6, 12, 18, 24, 48, and 72 hours.

The PMP values developed by the state of Arizona (AWA, 2013) that update HMR 49 and are consistent with the storms of record and state-of-the-practice technology. The APS contractor used the PMP tool (AWA, 2013) and values as the foundation for evaluating extreme event flood hazards at the Palo Verde Nuclear Generating Station site.

References

AWA, Evaluation of the Reliability of Generalized PMP Values Provided by HMR 49 for Arizona, Alternative Approaches that can be Used on a Statewide Basis for PMP Determination, Applied Weather Associates, LLC, September 2008.

AWA, Probable Maximum Precipitation Study in Arizona, Applied Weather Associates, LLC, July 2013b.

NOAA, Hydrometeorological Report No. 49, Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages, NOAA, National Weather Service, Septmenber 1977.

NRC, United States Nuclear Regulatory Commission (NRC), Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046. PNNL-20091, NRC Job Code N6575, Washington, D.C., November 2011.

NRC, United States Nuclear Regulatory Commission (NRC), Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Force Review of Insights from the Fukushima Dai-ichi Accident, Washington, D.C., March 12, 2012.

PRC, Peer Review Committee Final Report for the Probable Maximum Precipitation Study for Arizona by Applied Weather Associates' Peer Review Committee, August 2013.

# APPENDIX D

# SOFTWARE USED IN FLOOD HAZARD REEVALUATION

## APPENDIX D

## SOFTWARE USED IN FLOOD HAZARD REEVALUATION

The following software was used to perform the flood hazard reevaluation analyses:

- FLO-2D Pro (FLO-2D, 2012)
- FLO-2D Pro Release 14.03.07 (FLO-2D, 2014a)
- FLO-2D Pro Release 14.03.07.URS (FLO-2D, 2104d)
- ArcGIS 9.3.1 (ESRI, 2009)
- ArcGIS 10.1 (ESRI, 2012)
- ArcHydro 10.1 (ESRI, 2011)
- United States Army Corps of Engineers HEC-HMS 3.5 (USACE, 2010a)
- USACE HEC-GeoHMS (USACE, 2009)
- USACE HEC-RAS 4.1 (USACE, 2010b).

The FLO-2D Pro software is a volume conservation model that routes fluid flow in one-dimensional channel flow, two-dimensional overland flow, or an interaction between the two model components. The FLO-2D Pro software is an effective tool for delineating flood hazards or designing flood mitigation. The software is also available in a Basis configuration with fewer capabilities. The Basic model has been approved by the Federal Emergency Management Agency (FEMA) for use in Flood Insurance Studies. The Pro model, though not specifically approved by FEMA, includes all the features of the Basic model, along with some additional features (e.g., storm drain interface with surface water using the EPA SWMM program, parallel processing capabilities, and expanded capabilities for simulating sediment transport (FLO-2D, 2012).

FLO-2D Pro Release 14.03.07.URS utilized a modified version of the FLO-2D model that specifically accounted for roof detention, parapet walls, scupper inlets, scupper outlets locations, leaders, and downspouts. The flow to the ground for the LIP event was attenuated on the roof and discharged to specific locations, in lieu, of directing runoff directly to the ground. This enhancement to the FLO-2D program was developed by the FLO-2D developers specifically for this project.

HEC-GeoHMS and ArcHydro are tools that function within the ArcGIS interface. At the time of the writing of this report, hydrologic and hydraulic simulation models developed, described, and maintained by the USACE Hydrologic Engineering Center (HEC) are acceptable to the NRC (NRC, 2011).

HEC-HMS is designed to simulate the precipitation-runoff processes of drainage basin networks. It is applicable in a wide range of geographic areas, including large and small urban and natural watersheds. HEC-HMS is capable of representing many different watershed sizes and land coverage conditions (USACE, 2010a).

HEC-GeoHMS (USACE, 2009) and ArcHydro (ESRI, 2011), which are run within the ArcGIS framework (ESRI, 2012), were used in the pre-processing of watershed data to develop input data files for the HEC-HMS model.

HEC-RAS (USACE, 2010b) is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. It was used to provide rating curves for modeling the flow through culverts under Highway I-10 in the East Wash. These rating curves were then used in the HEC-HMS model developed to support the reevaluation of the river flooding hazard due to the PMP event.

All software used to perform the flood hazard reevaluation analyses has been verified and validated, and commercially dedicated in accordance with a quality assurance program that meets the requirements of 10 CFR 50 Appendix B through compliance with the Basic and Supplementary Requirements established in Part I of American Society of Mechanical Engineers (ASME) NQA-1-2008 and NQA-1a-2009 Addenda[2] (ASME, 2009), as well as the following Subparts that apply to the previously identified software:

- Subpart 2.7, "Quality Assurance Requirements for Computer Software for Nuclear Facility Applications."
- Subpart 2.14, "Quality Assurance Requirements for Commercial Grade Items and Services."

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[2] ASME NQA-1-2008 and NQA-1a-2009 Addenda documents cannot be reproduced electronically or via hardcopy without the written consent of ASME.