FLOOD HAZARD REEVALUATION REPORT

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

for the

COMANCHE PEAK NUCLEAR POWER PLANT UNITS 1 & 2

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Facility Operating License Nos. NPF-87 & NPF-89
NRC Docket Nos. 50-445 and 50-446

LUMINANT GENERATION COMPANY, LLC

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The U.S. Nuclear Regulatory Commission (NRC) issued a letter on March 12, 2012 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 (NTTF), a portion of which calls for performing flood hazard reevaluations at all Nuclear Power Plants (NPPs) in the United States (Reference 1). On behalf of Luminant Generation Company, LLC (Luminant), this Flood Hazard Reevaluation Report for the Comanche Peak Nuclear Power Plant (CPNPP) Site provides the information to address NRC Recommendation 2.1 with due consideration to the most recent guidance and regulations. The reevaluated flood hazard is based on a study provided by Paul C. Rizzo Associates, Inc. (Reference 26).

1.2 DESCRIPTION OF STUDY AREA

The CPNPP Site is located in Somervell County in north central Texas (***Figure 1-1***). Squaw Creek Reservoir (SCR), established for CPNPP cooling, extends northward into Hood County. SCR is impounded by Squaw Creek Dam which was completed in 1977 (Reference 2). The Site is situated adjacent to the SCR and Squaw Creek, a tributary of the Paluxy River, which is a tributary of the Brazos River. The Site is over 30 miles southwest of the nearest portion of Fort Worth and approximately 4.5 miles north-northwest of Glen Rose, the nearest community. The general location map of the CPNPP Site can be seen on ***Figure 1-1***.

1.3 SITE BACKGROUND AND HISTORY

The CPNPP Site, which is operated by Luminant, presently contains two Westinghouse Pressurized Water Reactors (PWRs), the construction of which began in 1974. Unit 1 became operational in 1990 and Unit 2 became operational in 1993. CPNPP Units 3&4 are proposed to be built approximately 0.49 miles west-northwest of CPNPP Units 1&2 in Somervell County (Reference 3).

2.0 FLOOD HAZARDS AT THE SITE

***Section 2.0* has been prepared in response to Requested Information Item 1.a. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1) and documents results, as well as pertinent site information and detailed analysis related to the applicable flood hazards. Relevant Systems, Structures and Components (SSCs) important to safety and the Ultimate Heat Sink (UHS) are included in the scope of this reevaluation including pertinent data concerning these SSCs.**

2.1 HISTORIC FLOODS

Historical flooding at the CPNPP Units 1&2 Site is of interest with respect to the Brazos River, the Paluxy River, Squaw Creek, and the SCR. These rivers, stream, and reservoir were identified based on an initial qualitative screening of rivers, streams, and creeks in the vicinity of CPNPP Units 1&2 Site, which was carried out taking into account the distance from the Site and location relative to the Site, as well as historical flow rates. All historical floods occurring within the drainage areas relevant to the CPNPP Units 1&2 Site have been due to precipitation runoff into streams and rivers, with no known instances of flooding occurring due to hurricanes, storm surges, tsunamis, dam failures, channel diversions, or ice jams (Reference 2). A list of the most significant historical floods with respect to maximum water levels near the CPNPP Units 1&2 Site is provided in ***Table 2-1***.

All elevations in this Report are given with respect to mean sea level (MSL), unless otherwise specified on a case-by-case basis.

The greatest known flood on the Brazos River occurred in 1876 (Reference 2). However, little quantitative information is available regarding this flood as it occurred before flow monitoring began. The United States Geological Survey (USGS) gauge 08091000 on the Brazos (***Figure 1-1***), nearest to the CPNPP Units 1&2 Site, is located just upstream of the confluence with the Paluxy River (Reference 3). The maximum recorded stage and discharge measurements for gauging station 08091000 indicate that the maximum recorded water surface elevation was 603.58 feet (ft) on April 28, 1990, which corresponds to a discharge of 79,800 cubic feet per second (cfs) (Reference 3).

USGS gauge 08091500 is located on the Paluxy River upstream of the confluence with the Squaw Creek tributary near Glen Rose, Texas (***Figure 1-1***). The maximum recorded water surface

elevation for gauging station 08091500 is 636.86 ft on April 17, 1908, which corresponds to the maximum recorded discharge of 59,000 cfs (Reference 3).

The USGS gauge (08091750) which is closest to the CPNPP Units 1&2 Site is located on Squaw Creek just below the SCR (***Figure 1-1***). The maximum recorded water surface elevation for gauging station 08091750 is 610.90 ft on April 8, 1975, which corresponded to the maximum recorded discharge of 9,030 cfs (Reference 3).

It is noted that none of the aforementioned historical floods approached the CPNPP Units 1&2 plant grade at elevation 810 ft.

2.2 DETAILED SITE INFORMATION

Section 2.2 **has been prepared in response to Requested Information Item 1.a.i. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1). Relevant site data to be considered includes detailed site information (both design and as-built), and present-day site layout, elevation of pertinent SSCs important to safety, site topography, as well as pertinent spatial and temporal data sets.**

The following Sections present detailed information regarding the CPNPP Units 1&2 Site, including information on site topography, a description of relevant safety-related Structures, Systems, and Components (SSCs), and a description of the Ultimate Heat Sink.

2.2.1 Designed Site Information

Designed site information describes characteristics considered for the original licensing basis of the CPNPP Units 1&2 Site. ***Figure 2-1*** shows the CPNPP design site layout and topography. A list of the CPNPP Units 1&2 safety-related structures is provided in ***Table 2-2***. A list of design parameters found in the license document (Reference 2) is included in ***Table 2-3***.

2.2.1.1 Site Topography

The CPNPP Units 1&2 Site is located within the Great Plains Physiographic Province, approximately four miles west of the Brazos River at a site elevation of 810.0 ft (Reference 2). The normal operating elevation of the SCR is 775.0 ft (Reference 2). The overall topography in the site vicinity ranges from slightly undulating to stair-stepped (Reference 2). The CPNPP

Units 1&2 Site is located on a relatively narrow ridge trending east-southeast and is bordered on the north and south by portions of the SCR (Reference 2). The design site layout and site-specific topography can be seen on ***Figure 2-1***. Elevations within five miles of the Site range from approximately 600 to 700 ft in valleys to 900 to 1,000 ft on ridge tops (Reference 2). Steep slopes are present in the site area, ranging from 15 to 30 degrees, or more in some places, and generally exhibiting a stair-stepped appearance (Reference 3). The elevations of USGS gauging stations 08091000, 08091500, and 08091750 are 561.79, 609.66, and 599.00 ft, respectively (Reference 3).

2.2.1.2 Description of Safety-Related Structures, Systems, and Components

The CPNPP Units 1&2 Site contains several safety-related SSCs. Each CPNPP Unit is comprised of an individual safety-related Reactor Building, a Safeguards Building, a Diesel Building, a Condensate Storage Tank, a Reactor Makeup Water Storage Tank, and a Refueling Water Storage Tank. The remaining safety-related structures, which are mutual to Units 1&2, are the Auxiliary Building, Electrical and Control Building, Fuel Building, and the Service Water Intake Structure (SWIS) (Reference 2). A list of all designed CPNPP Units 1&2 safety-related structures and their elevations can be found in ***Table 2-2***.

2.2.1.3 Description of the Ultimate Heat Sink

The single source of safety-related cooling water and the ultimate heat sink for CPNPP Units 1&2 is the Safe Shutdown Impoundment (SSI). The SSI was designed to contain a volume of water, including evaporative contingency, that is sufficient to provide cooling water for a period of over 30 days without makeup water, which would safely limit the effects of an accident in one unit, permit the safe shutdown of the other unit, and maintain both units in a safe shutdown condition (Reference 2). The design for the available volume of water in the SSI is based on postulated 100-year drought conditions, as specified by NRC Regulatory Guide 1.27 (Reference 4), and after a postulated 40 years of sedimentation (Reference 2).

The SSI is part of the SCR formed by a portion of an inlet of the SCR to the south of CPNPP Units 1&2, which is separated from the main body of the reservoir by a seismic Category I rock-fill dam (Reference 2). The surface water elevation of both the SSI and SCR is a minimum of 770.0 ft during normal operation. The normal operating elevation of the SCR is 775.0 ft (Reference 2). A channel was excavated through the ridge to the southwest of the SSI Dam in order to connect the SSI with the main body of the SCR (***Figure 2-2***). The bottom of the channel

is at 769.5 ft, 6 inches (in.) which is below the normal minimum operating level, and allows water to flow back and forth under normal conditions keeping the reservoir surfaces at the same elevation (Reference 2). In the event that the water level in the SCR drops under emergency conditions, the SSI Dam is designed to hold back between 284 acre-feet and 367 acre-feet of water to allow for continued cooling and safe shutdown of CPNPP Units 1&2 (Reference 2).

2.2.2 As-Built Site Information

As-built site information describes changes to the CPNPP Units 1&2 Site which occurred since the current licensing basis (CLB) (Reference 2), which may have the potential to influence the reevaluation of hydrological and flooding hazards. ***Figure 2-2*** shows the CPNPP as-built site layout and topography.

2.2.2.1 Site Topography

According to the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) "several changes to the site layout that had external flooding impact evaluations performed for them were identified in the flooding design basis document and other plant modification documents. These changes include:

- A floating raft system has been provided in the southern portion of the SSI equalization channel.
- The vehicle barrier system, including roadbase material placed over the SSI dam rip rap.
- The addition of concrete walkways installed on the slopes of the SSI near the east and west sides of the SWIS including the installation of a new fence for Perifeld supports, placement of concrete fence post pads, and extend drain pipes at SWIS.
- A pre-cast concrete trench ('Trenwa') containing electrical conduits was installed below grade, west of the Turbine Buildings."

All flooding analyses described in this Report, and described in ***Section 3.0,*** have been undertaken with consideration to and implementation of current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. Up-to-date topographic information and data including site surveys have been utilized in the generation of all of the applicable reevaluated flooding models.

As such, any changes to regional and site topography since the time of license issuance have been captured within the current analyses.

2.2.2.2 Description of Safety-Related Structures, Systems, and Components

As stated in the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5), "Several changes to the plant layout were identified during the walkdown which did not have documented flooding impact evaluations nor were they explicitly described in the FSAR. Based on field observations, the alterations to the topography by the modifications do not adversely affect the runoff assumed in the current licensing basis." Flood related changes to the licensing basis are discussed in ***Section 2.4***. The results of the flooding hazard reevaluation, with respect to safety-related SSCs, are presented in ***Section 3.0***.

2.2.2.3 Description of the Ultimate Heat Sink

The single source of safety-related cooling water and the ultimate heat sink for CPNPP Units 1&2 is the SSI. Texas Water Development Board (TWDB) undertook a detailed volumetric and sedimentation survey in 2007 of the entire SCR, including the SSI, utilizing both sub-bottom profiling depth sounders and sediment core samples (Reference 6). The results of the TWDB survey indicated that the SCR had a total reservoir capacity, including the SSI, of 151,273 acre-feet encompassing 3,169 acres at the conservation pool elevation of 775.0 ft (Reference 6). The as-built normal operating elevation of 775.0 ft is the same as the original designed elevation. The TWDB study results further indicated that the SSI had, at the time, a total capacity of 641 acre-feet and encompassed 45 acres at the elevation of 775.0 ft (Reference 6).

The overall results of the TWDB 2007 sedimentation survey indicate that SCR has accumulated 3,735 acre-feet of sediment since it was initially impounded in 1977, of which 40 acre-feet accumulated within the SSI (Reference 6). The majority of sediment accumulation was found to have occurred within the main body of the SCR with the thickest deposits being observed within the submerged Squaw Creek channel (Reference 6). Assuming a constant sedimentation rate and based on this measured sediment volume, SCR loses approximately 125 acre-feet of capacity per year with almost one acre-foot lost within the SSI per year (Reference 6).

The FSAR for CPNPP Units 1&2 gives an estimated sediment production from the watershed above the SSI during the 40 year projected service life of the plant. The anticipated reduction in storage capacity of the SSI during that period due to sediment accumulation was found to be 91 acre-feet, of which 85 acre-feet would be below elevation of 770.0 ft and the remaining 6 acre-feet between elevations of 770.0 ft and 775.0 ft (Reference 2).

2.3 CURRENT DESIGN BASIS

***Section 2.3* has been prepared in response to Requested Information Item 1.a.ii of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1). Relevant site data to be considered includes the current design basis flood elevations for all flood causing mechanisms.**

The current design basis, as presented in the Final Safety Analysis Report (FSAR) for CPNPP Units 1&2, indicates that all safety-related structures, except the SWIS and the Electrical and Control Building, do not require flood protection. The remaining safety-related structures are not subject to flooding, wave action, or wave run-up and do not require flood or wave protection (Reference 2). The SWIS is protected from wind wave run-up on the SCR by the SSI Dam, and the operating deck and safety-related equipment are located above the current design basis Probable Maximum Flood (PMF) level (Reference 2). The Electrical and Control Building is protected from flooding through the use of incorporated barriers in the circulating water system (Reference 2). The CPNPP Units 1&2 grade is at elevation 810 ft while the peak SCR water level is at 789.7 ft for the PMF (Reference 2). The maximum calculated water level within the SSI during the PMF is 790.5 ft, which leaves a freeboard within the SSI of 5.5 ft with respect to the SWIS operating deck and SSI Top-of-Dam crest elevations of 796.0 ft (Reference 2). The maximum SCR wave run-up and setup elevation at the CPNPP Units 1&2 Site due to coincident wind wave activity is 794.7 ft (Reference 2). The maximum wave run-up and setup elevations at the Squaw Creek Dam and SSI Dam are 793.7 and 791.3 ft respectively (Reference 2). A list of CPNPP Units 1&2 licensed water levels and corresponding flooding mechanisms can be found in ***Table 2-4***.

2.4 FLOOD-RELATED CHANGES TO THE LICENSING BASIS

***Section 2.4* has been prepared in response to Requested Information Item 1.a.iii of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1). Relevant site data to be considered includes flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance.**

2.4.1 Description of Hydrological Changes and Flood Elevations

The design basis flood elevations for CPNPP Units 1&2 applicable flood causing mechanisms are summarized in ***Table 2-4***. A description of flood-related and flood protection changes since license issuance is provided in ***Section 2.4.2***. Reevaluated flood elevations are presented in ***Section 3.0*** and are summarized in ***Table 3-3***.

2.4.2 Description of Flood-Related / Flood Protection Changes (Including Mitigation)

The flood protection system and flood mitigation measures described in the CPNPP Units 1&2 FSAR (Reference 2) and as observed and documented in the Post Fukushima Flooding Walkdown Report (Reference 5) are specifically relevant to the flooding hazard reevaluation analyses. With respect to the condition of the site drainage system, observations made during the CPNPP Units 1&2 Post Fukushima Flooding Walkdown inspection identified that several components of the overall site drainage system, such as catch basins, drainage basins, and concrete swales were observed to be partially clogged or obstructed with gravel and/or vegetation (Reference 5). These observations are utilized in evaluating the effects of the Local Intense Precipitation (LIP) rainfall event described in ***Section 3.2.1.***

Additionally, several changes to the plant layout were identified during the walkdown which did not have documented flooding impact evaluations nor were they explicitly described in the FSAR. Based on field observations, the alterations to the topography created by these changes/ modifications did not adversely affect the runoff assumed in the current licensing basis. Furthermore, there were no observations which required the implementation of newly installed flood protection or mitigation measures or other planned flood protection measures as currently incorporated into the CPNPP current licensing basis and supporting programs and processes.

2.5 CHANGES TO THE WATERSHED AND LOCAL AREA

***Section 2.5* has been prepared in response to Requested Information Item 1.a.iv of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1). Relevant site data to be considered includes changes to the watershed and local area since license issuance.**

Any changes to the contributing watershed within which the CPNPP Units 1&2 Site is situated since the time of license issuance have the potential to influence potential flooding hazards. A description of the watershed at the time of license issuance and pertinent changes to the watershed since license issuance are presented in the following Sections.

2.5.1 Description of Watershed and Local Area at the Time of License Issuance

The CPNPP Units 1&2 Site is situated on a peninsula in the SCR at a site grade elevation of 810.0 ft within the Squaw Creek watershed. Squaw Creek is a small tributary of the Paluxy River, which is in turn a tributary of the Brazos River (***Figure 1-1***). The calculated PMF design basis flood level is 789.7 ft on the SCR and 790.5 ft on the SSI (Reference 2). The SCR, established for station cooling, borders the site on the north, east, and south sides. Flooding at the CPNPP Units 1&2 Site is of interest with respect to Squaw Creek, as well as the Brazos River and the Paluxy River.

The Brazos River and its tributaries extend from New Mexico through Texas and into the Gulf of Mexico, constituting one of the principal river systems in the south-central United States (Reference 2). Numerous river control structures and reservoirs have been built on the Brazos River. The FSAR for CPNPP Units 1&2 principally considers that the most potential for significant effect on plant operation will occur as a result of flooding on the Brazos River between Lake Granbury and Lake Whitney, which are upstream and downstream of the Site, respectively (Reference 2). Within this reach, the Brazos River channel is located in incised meanders, which are flanked by rock slopes that confine the river within a relatively narrow channel (Reference 2). The geometry of the river banks is typically characterized as steep on the outside of a meander bend and generally more gently sloping on the inside of a bend
(Reference 2).

Squaw Creek drains parts of Hood and Somervell counties and empties into the Paluxy River, upstream of the confluence of the Brazos River and Paluxy River (Reference 2). Squaw Creek is a small and intermittent stream, which often has no flow during dry periods (Reference 2). The SCR, which has an approximately 64-square mile (mi^2) catchment, is impounded by Squaw Creek Dam approximately 4.3 stream miles north of Squaw Creek's confluence with the Paluxy River (Reference 2). The topography within the Squaw Creek watershed is gently to steeply rolling, being influenced by the underlying geology with generally steeper slopes in limestone areas than in shale and sandstone areas (Reference 2). Topographic maps used to characterize the watershed at the time of license issuance included a number of small man-made ponds in the catchment, the total volume of which was estimated to be approximately 1,150 acre-feet (Reference 2). Apart from these small ponds, no water control structures, weirs, or canals were considered in the watershed characterization (Reference 2). At the time of license issuance, the town of Tolar, Texas was the only community in the catchment with the remainder of the catchment being characterized largely as ranchland with some cultivated areas (Reference 2).

At the time of license issuance, the Paluxy River was used as the basis for developing hydrologic parameters for Squaw Creek and its sub-catchments, as it is hydrologically similar to Squaw Creek, which was ungauged at that time (Reference 2). No water control structures on the Paluxy River were taken into consideration at the time of license issuance (Reference 2).

2.5.2 Description of Any Changes to the Watershed and Local Area Since License Issuance

Changes to the predominantly rural agricultural watershed and local area since the time of license issuance, mostly in the form of changes in land use and land development are not significant. All flooding analyses described in this Report and described in ***Section 3.0*** have been undertaken with consideration to and implementation of current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. For each delineated watershed, detailed information was considered regarding the current land use and soil types within each sub-basin. Additionally, for both the Squaw Creek and Paluxy River watersheds, consideration was given to a potential 50-year projected scenario with an assumed 5 percent increase in impervious area for each sub-basin. The selection of the 5 percent increase in impervious area is based on conservative engineering judgment. As such, changes to the watershed current land use and local area since the time of license issuance, as represented by the existing conditions, have been considered within the current analyses.

2.6 CURRENT LICENSING BASIS FLOOD PROTECTION

***Section 2.6* has been prepared in response to Requested Information Item 1.a.v. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1). Relevant site data to be considered includes current licensing basis flood protection and pertinent flood mitigation features at the site.**

With the exception of the SWIS and the Electrical and Control Building, safety-related structures at the CPNPP Units 1&2 Site do not require flood protection under the current licensing basis (Reference 2). The CPNPP Units 1&2 grade is at elevation 810 ft and the current license basis peak SCR water level is at elevation 789.7 ft for the PMF (Reference 2). The maximum calculated water level within the SSI during the PMF is 790.5 ft (Reference 2). The maximum SCR wave run-up and setup elevation at the CPNPP Units 1&2 Site due to coincident wind wave activity is 794.7 ft (Reference 2). The maximum wave run-up and setup elevations at the Squaw Creek Dam and SSI Dam are 793.7 ft and 791.3 ft, respectively (Reference 2). Safety-related plant structures are conservatively designed for hydrostatic loads with the design basis ground water level at the plant grade elevation of 810 ft, except for the Service Water Intake Structure which is designed for a design basis groundwater level at elevation 793 ft (Reference 2).

Given the above noted bounding flooding events, several pertinent flood protection and mitigation features that are considered in the CPNPP current licensing basis to protect against the ingress of water to safety related SSCs were identified in the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5).

<u>SWIS and SSI Dam</u>

The Service Water Intake Structure and SSI Dam are the only safety-related structures subject to wave action. The SWIS is currently protected from the effects of wind wave activity occurring on the SCR by the SSI Dam having a top of dam crest elevation of 796 ft. The SWIS operating deck and safety-related equipment are located above the current design basis PMF level at 796 ft (Reference 2). The limiting current design basis assumes an overland 40 miles per hour (mph) wind occurring coincidentally with the SSI PMF (Reference 2). Because the wind wave activity within the SSI at the SWIS is negligible, no significant dynamic forces are considered (Reference 2). The maximum wave run-up elevation caused by the 40-mph wind coincident with the PMF is included in the design hydrostatic load (Reference 2).

Exterior Walls and Floors (Including Penetrations)

From the Walkdown Report (Reference 2), any exterior wall (above or below grade) protecting space credited as dry in the CLB from groundwater or surface water flooding was included in the walkdown scope, even if the exterior walls were not explicitly mentioned in the CLB. The inspection applied to portions of the walls below design basis flood and/or groundwater levels. No conditions were observed during the walkdown that indicated rooms in Seismic Category I structures below the design groundwater levels could be inundated by groundwater. A few rooms showed signs of past or active in-leakage from groundwater through penetrations. The observed locations of in-leakage were minute in nature and were not of significant accumulated volume that exceeded the bounds of internal flooding analyses, nor did those locations and volumes of in-leakage affect the operability of safe shutdown systems and equipment important to safety. The condition of the seal or sleeve for several of the penetrations were found to be slightly impaired or inconsistent with the original design and entered into the site's Corrective Action Program (CAP). The overall observed integrity and credited function of the flood protection feature was determined to be acceptable to mitigate the potential effects of the applicable flood causing mechanism, in this case groundwater intrusion.

Seismic Category I Roofs

SWIS Structure:

The roof of the SWIS is designed with a slope of 1 foot over the 44 foot width of the roof to accomplish drainage of rainfall (Reference 2). There are multiple equipment hatches in the roof. The sealing lip for each is located above the top of the roof.

Non-SWIS Structures:

Each building is equipped with a roof drainage system designed to effectively collect, pass and discharge the water volume resulting from a six-inch rainfall in one hour with a maximum intensity two inches in five minutes. Scuppers are provided in parapet wall openings where the scupper invert elevation will not be more than three inches above the roof at the outside wall or not more than five inches above the low point of the roof.

The design load on all nuclear safety-related buildings considers an eight-inch maximum uniform depth of water in addition to the regular live loads. Roof parapet wall relief openings ensure that the eight-inch water level is not exceeded during the PMP. The openings are specifically located at all roof low points and extend from the roof low-point elevation to the top of the parapet.

They are designed to preclude roof ponding and the possibility of ice and snow build-up blocking the openings.

Class 1E Cable Vaults/Manholes

There are 14 Class 1E cable vaults (4-Unit 1 & 10-Unit 2) some of which were identified in the walkdown effort as the only plant site feature to have small Available Physical Margin (APM). Design drawings depict a distance of 14 inches from the bottom of the lowest cable tray in each vault to the floor elevation of each vault. Design drawings show the cover plates of the vaults as being sealed with a gasket and with the top of the cover plate being 4 inches above grade when not on a roadway and being flush with grade when being constructed in a roadway. The drawing also provides for inter-connecting conduit ductbanks sloped toward the cable vaults/manholes.

The APM issue was entered into the site's CAP. It was determined that the submergence of instrumentation cable and low-voltage cables operating at 120V in the applicable vaults did not pose a challenge to the operability of safe shutdown equipment. Potential leakage pathways down the attached conduits were inspected from the interior of the attached buildings as part of the penetrations walkdown.

Manhole Covers

There are nine manhole covers associated with Seismic Category I SSCs that were included in the scope of protection features in the flooding walkdowns. One manhole provides access to the Fuel Building Service Water Pipe Tunnel (X-FBSWT). The access point is covered by multiple missile barriers that have been incorporated through design changes. Although the manhole cover itself does not contain a gasket, the cover is effectively sealed from in-leakage due to precipitation.

The remaining eight manhole covers are associated with the four buried Diesel Generator Fuel Oil Storage Tanks (DGFOST). The East manhole for each of the four tanks provides a watertight enclosure by design and contains a level element transducer and a diesel fuel oil fill standpipe that is capped and sealed when not in use (Reference 2). The West manholes provide a similar cover plate design and contain fuel oil supply and return overflow piping to the Diesel Generator (DG) building's fuel oil Day Tank. Each manhole enclosure has a caulked and gasketed cover plate at the surface. The walkdown inspection of these eight manholes and associated cover plates revealed observations of past water in-leakage around cover plate and sample access cover gaskets. The walkdowns revealed that the caulking applied to the manhole covers at the plant grade surface did not entirely prevent water from entering below the cover, but the main gasket joint of the cover to the support plate prevented this water from entering the manhole in most

locations on the gasket. In-leakage quantities were minimal and insufficient to cause loss of function of any DG equipment within the manhole. Observations of the physical configuration of these manholes were entered into the CAP.

Severe Weather Abnormal Procedure Simulation

Flood protection measures have been taken into consideration with respect to the Electrical and Control Building (Reference 2). The Electrical and Control Building is protected from flooding through the use of incorporated barriers in the circulating water system (Reference 2). The circulating water system is a closed system during plant operation and flood protection is only required when the system is open for maintenance. Existing Severe Weather Abnormal Procedures provide response guidance when the SCR water level increases and the system is open for maintenance.

Reference 5 described external flooding walkdowns and simulation of applicable CPNPP flood response procedures, including applicable technical requirements manual sections and technical specifications. Potential clarifications and enhancements for administrative implementation of flood protection actions and the features that protect Seismic Category I structures from the effects of the current licensing basis PMP and PMF were identified. These observations were entered into the Corrective Action Program (CAP). The actions relating to the reasonable simulation of flood protection procedures were determined to be feasible and effective.

Onsite Natural Drainage

The CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) states that "the onsite drainage system is designed to remove the water resulting from a rainfall of six inches in one hour and 7.5 inches in two hours, in such a manner that the runoff is accomplished without ponds forming on the ground. Further, the drainage system is designed to adequately drain a rainfall of 15 inches in one hour and 22 inches in two hours in such a way that there are no ponds which can back up into the structures and affect safety-related systems." In addition, the Units 1&2 FSAR (Reference 2) states that a "possible clogging of any ditch will not affect the system's water removal capacity." This accounts for debris accumulation in those structures. Additionally, a partial clogging factor was used in the design of the catch basin inlets in the design basis of the yard drainage, which accounts for debris accumulation of those structures.

Drainage system catch basins, drainage basins, swales, and trenches were inspected during the flooding walkdowns. Observations were made which identified that some of these drainage features were partially clogged or inconsistent with the configuration as presented within the current licensing basis. However, it was concluded that the drainage system will still perform its design function and will not result in sufficient ponding that would cause water ingress into the safety related structures. The observed conditions were entered into the CAP to reconcile the as-found configuration with the current licensing basis description.

2.7 ADDITIONAL SITE DETAILS

***Section 2.7* has been prepared in response to Requested Information Item 1.a.vi of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1). Relevant site data to be considered includes additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc).**

2.7.1 Bathymetry

The design site layout elevation contours beneath the surface of the SCR and the SSI in the vicinity of CPNPP Units 1&2 can be seen on ***Figure 2-1***. For purposes of the computation of wave run-up at the Site, an average bottom elevation of the relevant portion of the SCR/SSI was utilized along the direction of the longest fetch (***Section 3.0***). Cross sections along the relevant fetch length were used to determine wave run-up, slope angle, and the average water depth.

2.7.2 Recommendation 2.3 Walkdown Results

The CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) gives results of the walkdown, including applicable key findings and identified degraded, non-conforming, or unanalyzed conditions, and includes detailed descriptions of the actions taken or planned to address these conditions. The CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) concluded there were no newly installed flood protection or mitigation measures and no observations which required the implementation of other planned flood protection measures as currently incorporated into the CPNPP current licensing basis and supporting programs and processes.

Given the reevaluated flood hazard results established in Section 3.0, the effects of water levels that are not bounded by the current licensing basis to the pertinent flood protection and mitigation features described in Section 2.6 and as identified in the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) will be provided in Section 4.3.2.

2.7.3 Site-Specific Visit Results

A site visit was conducted from September 11, 2012 to September 14, 2012. The SWIS, Safe Shutdown Impoundment Dam, and Squaw Creek Dam were inspected during that time. Additionally, photographs were taken of the Site and Site area which were later used to aid in developing the appropriate inputs to the hydrologic models, such as Manning's n roughness coefficients.

3.0 FLOOD HAZARD REEVALUATION ANALYSIS

***Section 3.0* has been prepared in response to Requested Information Item 1.b. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1): Evaluation of the flood hazard for each flood causing mechanism should be based on present-day methodologies and regulatory guidance. Provide an analysis of each flood causing mechanism that may impact the site including local intense precipitation and site drainage, flooding in streams and rivers, dam breaches and failures, storm surge and seiche, tsunami, channel migration or diversion, and combined effects. Mechanisms that are not applicable at the site may be screened-out; however, a justification should be provided. Provide a basis for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data.**

3.1 SUMMARY OF RECOMMENDATION 2.1

To respond to NRC Recommendation 2.1 (Reference 1), the NRC requested that each licensee provide a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation on the Site, PMF on streams and rivers, storm surges, seiches, tsunamis, and dam failures. A hazard evaluation should be performed for each reactor licensed under 10 CFR Part 50, including the spent fuel pool and the various modes of reactor operation. The reevaluation should apply present-day regulatory guidance and methodologies being used for Early Site Permit (ESP) and Combined Operating License (COL) reviews. The reevaluation should employ current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard.

3.2 FLOOD CAUSING MECHANISMS

The NRC NUREG/CR-7046 (Reference 7) recommends using a Hierarchical Hazard Assessment (HHA) method for evaluating the safety of SSCs. The HHA method is a progressively refined, stepwise estimation of site-specific hazards that evaluates the safety of the Site with the most conservative plausible assumptions consistent with available data. The HHA process proceeds as follows (Reference 1):

a) Select one flood causing mechanism to be reanalyzed.

b) Develop a conservative estimate of the site-related parameters using simplifying assumptions for a flood causing mechanism and perform the reevaluation.

c) Determine if the reevaluated flood hazard elevation (from step b) is higher than the original design flood elevation for the selected flood causing mechanism. If not, use this flood elevation for this causal mechanism for comparison of reevaluation against the current design basis.

d) Determine if the site-related parameters can be further refined. If yes, perform reevaluation (repeat step c). If no, use this flood elevation for this causal mechanism for comparison of reevaluation against the current design basis.

e) Determine if all flood causing mechanisms have been addressed. If yes, continue to the following. If no, select another flood causing mechanism (step a).

For each flood causing mechanism, compare the final flood elevations from the hazard reevaluation against the current design basis flood elevations. Using this comparison, determine whether the design basis flood bounds each reevaluated hazard.

Each potential flood causing mechanism has been evaluated based on present-day methodologies and regulatory guidance. Details regarding the considerations and outcome of the analyses regarding each flood causing mechanism are presented herein.

3.2.1 Local Intense Precipitation

Section 3.2.1 addresses the effects of Probable Maximum Precipitation (PMP) on the local area of the CPNPP Units 1&2 Site. The HHA diagram for local intense precipitation flooding analysis is presented in ***Figure 3-1***.

3.2.1.1 Local Intense Precipitation Rainfall

The local intense PMP was evaluated to determine the Probable Maximum Storm (PMS) event capable of generating the maximum amount of direct runoff (or peak discharge) at the CPNPP Units 1&2 Site. The local intense PMP for the site drainage basin was calculated using the Hydrometeorological Report No. 52 (HMR 52) (Reference 8). The CPNPP Units 1&2 Site drainage basin boundary utilized for the calculation of the local intense PMP encloses an area of 6.4 acres or 0.01 mi^2. The peak one-hour storm event acting on an area of 1 mi^2 (1-hour, 1-mi^2) was determined for utilization as input to the reevaluation of the CPNPP local site drainage HEC-HMS model. The PMP depth generated by the 1-hour 1-mi^2 storm event acting on the CPNPP Units 1&2 Site was determined using the isohyet chart presented on Figure 24 of the HMR 52 report (Reference 8).

The FSAR for CPNPP Units 1&2 does not present the PMP generated by the 1-hour 1-mi^2 storm determined for local intense precipitation, values are instead based on HMR No. 33 methods and are not analogous to reevaluated results (Reference 2). However, each safety-related building at the CPNPP Units 1&2 Site is equipped with a roof drainage system designed to effectively collect, pass, and discharge the water volume resulting from a six-inch rainfall in one hour with a maximum intensity of two inches in five minutes (Reference 2). Furthermore, the roofs of all safety-related buildings are designed to support an eight inch maximum uniform depth of water in addition to the regular live loads considered (Reference 2). Additionally, the onsite drainage system is designed to remove the water resulting from a rainfall of 6 in. in one hour and 7.5 in. in two hours, in such a manner that runoff does not form ponds on the ground (Reference 2). Further, the drainage system is designed to adequately drain a rainfall of 15 in. in one hour and 22 in. in two hours in such a way that there are no ponds which can back up into the SSCs (Reference 2). A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-4 and 3-5***, respectively.

The calculated results of the flooding reevaluation pertaining to CPNPP Units 1&2 utilizing HMR 52 Figure 24 give a 1-hour 1-mi^2 PMP depth of 19.1 in. The PMP results generated by the HMR 52 program give a cumulative 72 hour precipitation of 49.08 in. Local intense precipitation hyetographs are provided on ***Figure 3-2***.

3.2.1.2 Effects of Local Intense Precipitation

The local PMF at the CPNPP Units 1&2 Site was calculated per the HHA method per NRC guidance by first using the Rational Runoff Transformation Method (see description below) and second using more site-specific data in the HEC-HMS software (Reference 7). The site drainage basin model used to calculate the local PMF divides the total power plant site area into 31 sub-basins. The area of each of the 31 sub-basins is listed in ***Table 3-1***.

The Rational Runoff Transformation Method is appropriate for small basins, especially in areas that are mostly paved. The Rational Runoff Transformation Method assumes that the maximum rate of runoff from a drainage basin occurs when all parts of the watershed contribute and that the rainfall is uniformly distributed over the catchment area. No runoff losses were assumed and the runoff coefficient was assumed to be 1.0. The 1-hour 1-mi^2 reevaluated PMP described in ***Section 3.2.1.1*** was converted to rainfall intensity with consideration to the rainfall depth and duration. The intensity duration curve for durations up to 30 minutes was used as the intensity input for the Rational Runoff Transformation Method.

The peak runoff for each sub-basin was calculated based on the rainfall intensity and sub-basin area as well as the runoff coefficient. The calculated rainfall intensity and peak runoff for each sub-basin is listed in ***Table 3-1***.

In accordance with the approach of the HHA method (***Figure 3-1***), a more refined and more detailed evaluation of runoff at the CPNPP Units 1&2 Site was performed using HEC-HMS 3.5 (Reference 9). ***Figure 3-3*** shows the HEC-HMS model watershed routing layout for the 31 sub-basins. The sub-basins for CPNPP Units 1&2 Site area were modeled accounting for runoff flowing into the SCR or SSI.

For the most refined scenario, the US Department of Agriculture (USDA) – Natural Resources Conservation Service (NRCS), curve number (CN) method, which is commonly used to model runoff from urbanized watersheds, was used to model hydrologic loss characteristics (Reference 10). The plant footprint area was assumed to be impervious. A curve number of 98 was used, which is commonly used for impervious surfaces such as parking lots, roofs, and driveways (Reference 11). A Unit Hydrograph (UH) Transform Method was specified for each sub-basin which was based on the NRCS synthetic graph applicable to drainage areas of 2,000 acres or smaller (Reference 12). Time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of a sub-basin was limited to a minimum value of 6 minutes (Reference 13). The HEC-HMS model utilizes the reevaluated PMP discussed above.

A summary of site area drainage details, including peak runoff, calculated for both the Rational Runoff Transformation Method and using the HEC-HMS model are shown in ***Table 3-1***. Utilization of the HHA method (Reference 7) for determination of the PMF yields less conservative results in the case of the HEC-HMS model which incorporates more site-specific data than the Rational Runoff Transformation Method. The determination of water levels due to local intense precipitation at the CPNPP Units 1&2 Site is discussed in ***Section 3.2.1.3***.

3.2.1.3 Water Level Due to Local Intense Precipitation

Following the standards of the HHA approach, four scenarios were considered with each less conservative than the next. In Scenario 1, peak runoff rates based on the Rational Runoff Transformation Method were used with all at-grade hydraulic drainage structures blocked, including the openings along a 5 foot tall security Vehicle Barrier System (VBS) (***Table 3-1***) which surrounds the perimeter of the site protected area containing the safety related SSCs. Scenario 2 is similar to Scenario 1 except that the peak runoff rates are based on the Soil

Conservation Service (SCS) Runoff Transformation Method. Scenario 3 is similar to Scenario 2 except that the existing 5 ft openings along the VBS were represented and assumed not to be blocked. Scenario 4 is like Scenario 3 except that the time-varying discharge calculated in HEC-HMS, including the SCS Curve Number Method rainfall losses and unit hydrograph runoff transformation were used instead of the peak discharge rate. For all Scenarios, the underground drainage system including associated catch basins and drainage basins were assumed blocked and not credited. This is to account for debris that could potentially accumulate and block the structures in accordance with NUREG-7046 (Reference 7). ***Figure 3-3*** shows the HEC-HMS model and the sub-basins which were used to represent the conditions of Scenario 2 through Scenario 4. ***Table 3-1*** lists the peak discharge rates for each sub-basin calculated with Scenario 1 using the Rational Runoff Transformation Method and in the HEC-HMS simulations associated with Scenario 4.

The grade elevation at CPNPP Units 1&2 is 810 ft, and the SSI Dam and the operating deck of the SWIS are at an elevation of 796 ft (Reference 2). An approach is considered to be in need of further refinements if the simulated peak water levels for that approach exceed the elevation of 810 ft at any of the SSCs, or 796 ft for the SWIS or SSI Dam, or if ponding occurs in excess of that which is described in the CPNPP Units 1&2 FSAR (Reference 2).

The HEC-HMS simulations provide an estimate of water level increases within the SSI and the SCR. However, these changes were not considered in terms of the HHA approach, and the potential for flooding at the SSI is addressed separately (see ***Section 3.2.2.2.3***).

The simulated peak water levels due to ponding for Scenarios 1 through 4 exceeded the plant grade elevation of 810 ft at safety-related SSCs. The peak simulated water surface elevation for Scenario 4 (i.e., the most refined scenario) is 810.34 ft which corresponds to a maximum ponding depth of 0.34 ft near several safety related building structures. It should be noted that all exterior entrances which could provide a propagation pathway into the safety-related structures are above grade at elevation 810.5 ft or greater.

An equipment ramp entrance on the west face of the Unit 2 non-safety related Turbine Building (TB) is less than elevation 810.5 ft and has the potential to communicate local intense precipitation runoff interior to lower TB elevations which are located adjacent to the safety related Electrical Control Building at elevation 778 ft. The total volume of runoff that can accumulate in the lower TB elevations (i.e., elevations lower than 778 ft) for the duration of the Local Intense Precipitation event was determined to be less than the available combined capacity of the TB

sump and condenser pit areas, as discussed in Section 4.3.2. Thus, runoff propagation to the lower safety related Electrical Control Building at elevation 778 ft does not occur for this flood causing mechanism. ***Table 3-2*** provides simulated water levels and ponding depths for Scenario 4. ***Figure 3-4*** illustrates the simulated peak water levels and inundation area for each sub-basin modeled using Scenario 4.

A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-4 and 3-5***, respectively.

3.2.1.4 Wind Waves and Run-up Coincident with Local Intense Precipitation

The water levels in the SCR and the SSI due to local intense precipitation are lower than the water levels in the SCR and SSI due to the regional river flooding on the Squaw Creek watershed. Therefore, the wind-wave activity for water levels coincident to the regional PMF is considered bounding for the determination of water levels on the safety-related SSCs. Wind waves and run-up and associated water levels due to the regional PMF are discussed in ***Section 3.2.2.2.4***.

3.2.2 River Flooding

Flooding at the CPNPP Units 1&2 Site is of interest with respect to Squaw Creek, the Paluxy River, and the Brazos River. The HHA diagram for river flooding analysis is provided on ***Figure 3-5***. For purposes of analyzing hazards due to dam failures, the PMF of the Squaw Creek and Paluxy River watersheds is combined with Brazos River dam failure flood flow to determine potential backwater effects at the CPNPP Units 1&2 Site. Dam failure flooding is discussed in detail in ***Section 3.2.3***.

3.2.2.1 Probable Maximum Precipitation for River Flooding

The PMP was calculated for both the Squaw Creek and the Paluxy River basins in accordance with the criteria specified in HMR 52 (Reference 8). Basin-average precipitation for the PMS was determined in accordance with the temporal and spatial storm patterns associated with the PMP estimates provided in Hydrometeorological Report No. 51 (HMR 51) for storms up to 72 hours in duration (Reference 14). The current design basis determined the PMF for Squaw Creek by imposing the 48-hour PMP as obtained from Hydrometeorological Report No. 33 (HMR 33) upon the catchment (Reference 2).

The boundaries of the Squaw Creek and Paluxy River watersheds were defined within the HMR 52 computer program based on the reevaluated watershed delineation. Watersheds for Squaw Creek and the Paluxy River can be seen on ***Figure 3-6***. For each delineated watershed, detailed information was considered regarding the current land use and soil types within each sub-basin.

The locations of the Squaw Creek and Paluxy River watersheds were utilized, along with the HMR 51 PMP isohyet charts to determine generalized estimates of the all-season PMP for storm areas from 10 to 20,000 mi^2 with durations of 6, 12, 24, 48, and 72 hours. In both cases, the storm center coordinates were conservatively placed at the basin centroid resulting in higher runoff. A time interval of five minutes was selected in estimating the rainfall distribution.

The critical storm area for Squaw Creek was found by HMR 52 software to be 100 mi^2 with a critical storm orientation of 304 degrees and a critical 72-hour storm PMP rainfall of 43.03 in. The critical storm area for the Paluxy River was found by HMR 52 software to be 450 mi^2 with a critical storm orientation of 300 degrees and a critical 72-hour storm PMP rainfall of 36.24 in. An estimation of a 72-hour PMP total is not included in the current licensing basis as presented in the FSAR for CPNPP Units 1&2 (Reference 2).

The FSAR for CPNPP Units 1&2 gives a PMP of 25.5 in. in 6 hours on an area of 64 mi^2 in the CPNPP vicinity and a PMP of 31.3 in. in 12 hours, 34.7 in. in 24 hours, and 39.1 in. in 48 hours (Reference 2). A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-4 and 3-5***, respectively.

The PMS rainfall hyetographs developed for Squaw Creek and the Paluxy River were used as input when developing the Squaw Creek and the Paluxy River watershed HEC-HMS models to determine the maximum runoff. Watershed hyetographs are provided on ***Figure 3-7***.

3.2.2.2 Probable Maximum Flood for Squaw Creek and the Paluxy River

The PMF has been determined for both the Squaw Creek and Paluxy River watersheds. A runoff model for each watershed was created using HEC-HMS 3.5 software (Reference 9). The HEC-HMS models take into consideration the sub-basin areas, loss, transform, and routing methods, and estimate a flow rate for each of the defined junctions. ***Figure 3-8*** shows the HEC-HMS model watershed routing layouts for both the Squaw Creek and Paluxy River watersheds and sub-basins.

3.2.2.2.1 Probable Maximum Flood for Squaw Creek

The SCS Curve Number, and Snyder and Muskingum-Cunge methods (Reference 15) were used to calculate the basin loss, transform, and routing for Squaw Creek. The Snyder method was chosen for transform calculation as the Squaw Creek basin was modeled as an ungauged watershed, and empirical equations were used for the calibration. The Snyder method also accounts for the physical characteristics of the basin. The developed land use within the Squaw Creek watershed was used as a basis for the percentage of impervious area. Because flow rates for Squaw Creek have been controlled, flow rates could not be used for calibration purposes.

As per the HHA procedure for river flooding (***Figure 3-5***), several scenarios were simulated in HEC-HMS with decreasing levels of conservatism. Reaches and junctions for the Squaw Creek watershed can be seen on ***Figure 3-8***. The different scenarios considered in the HEC-HMS model for Squaw Creek are as follows:

- **Scenario 1**: No loss, no transformation, and instantaneous routing.
- **Scenario 2**: No loss, Snyder Unit Hydrograph transform method, and instantaneous routing.
- **Scenario 3**: No loss, Snyder Unit Hydrograph transform method, and Muskingum Cunge routing method.
- **Scenario 4**: SCS Curve number loss method, Snyder Unit Hydrograph transform method, and Muskingum Cunge routing method (representing calculated impervious percent for existing conditions).
- **Scenario 5**: Same as Scenario 4, but for 50 year land use projection (assumed 5 percent increase in percent impervious area)
- **Scenario 6**: Same as Scenario 4, but impervious percent increased to 100 percent.
- **Scenario 7**: Reservoir existing conditions case – The reservoir is modeled with existing land use conditions.
- **Scenario 8**: Reservoir projected case – The reservoir is modeled with projected land use conditions.
- **Scenario 9**: Same as Scenario 4 with basin 391 split into 2 basins as 391, and 392, and the SCR is represented only with the Service Spillway.
- **Scenario 10**: Same as Scenario 1 with basin 391 split into 2 basins as 391, and 392, and the SCR is added. Both the Service Spillway and Emergency Spillway are modeled.

- **Scenario 11**: Same as Scenario 2 with basin 391 split into 2 basins as 391 and 392 and the SCR is added. Both the Service Spillway and Emergency Spillway are modeled.
- **Scenario 12**: Same as Scenario 3 with basin 391 split into 2 basins as 391 and 392 and the SCR is added. Both the Service Spillway and Emergency Spillway are modeled.
- **Scenario 13**: Same as Scenario 4 with basin 391 split into 2 basins as 391 and 392 and the SCR is added. Both the Service Spillway and Emergency Spillway are modeled.
- **Scenario 14**: Same as Scenario 5 with basin 391 split into 2 basins as 391 and 392 and the SCR is added. Both the Service Spillway and Emergency Spillway are modeled.

The HEC-HMS model was calibrated using a 100-year return period rainfall against calculated peak flow for a 100-year return period. Since no usable data are available from the USGS gauge for Squaw Creek, empirical regression equations were used for calibration.

The reevaluated calculated flow value out of the SCR (from Scenario 13) is 169,463.8 cfs. The calculated flow rate into the SCR is 257,306 cfs. Combined effects are discussed separately in ***Section 3.2.8***. A flood level time history of the SCR for the river flooding analysis is provided on ***Figure 3-9***.

The FSAR for CPNPP Units 1&2 reports a PMF discharge value of 131,150 cfs for the outlet of the SCR and a peak inflow of 149,000 cfs (Reference 2). These values were computed using bounding isohyet PMF values and do not include any routing. Thus, the values reported for Units 1&2 are not directly comparable to the reevaluated PMF values. A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-4 and 3-5***, respectively.

The outflows from several of the scenarios are utilized for the determination of the water level due to the PMF as described in ***Section 3.2.2.2.3***.

3.2.2.2.2 Probable Maximum Flood for the Paluxy River

The SCS Curve Number, and Snyder and Muskingum-Cunge methods were used to calculate the basin loss, transform, and routing for the Paluxy River. The Snyder method was chosen for transform calculation. The Manning's n roughness coefficient for the main stream was taken as 0.045 and was further differentiated based on land use within the Paluxy River watershed for the overbanks. Transmission losses through the reaches were assumed to be zero for all scenarios, which is conservative because all runoff that reaches the channel will be routed through the channel instead of being absorbed. The developed land use within the Paluxy River watershed was used as a basis for the percentage of impervious area. The HEC-HMS model for the Paluxy River was calibrated to USGS gauging station 08091500.

As per the HHA procedure for river flooding (***Figure 3-5***), several scenarios were simulated in HEC-HMS with decreasing levels of conservatism. Reaches and junctions for the Paluxy River watershed can be seen on ***Figure 3-8***. The five different scenarios incorporated into the HEC-HMS model for the Paluxy River are as follows:

- **Scenario 1**: No loss, no rainfall-to-runoff transformation, no routing through the channels – Most conservative case.
- **Scenario 2**: No loss, Snyder Unit Hydrograph transformation, no routing through the channels.
- **Scenario 3**: No loss, Snyder Unit Hydrograph transformation, include routing through the channels.
- **Scenario 4**: Same as Scenario 3, but with losses and existing conditions – Existing conditions, based on model calibration.
- **Scenario 5**: Same as Scenario 4, but with assumed 5 percent increase in percent impervious for each sub-basin, 50 years from the present – Projected conditions.

The peak outflows from the Paluxy River watershed for Scenarios 1 through 5 are 2,560,000; 472,000; 419,000; 405,000; and 406,000 cfs, respectively. The FSAR for CPNPP Units 1&2 does not provide analogous peak outflows for the Paluxy River watershed (Reference 2).

A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-4 and 3-5***, respectively.

3.2.2.2.3 Water Level Due to Probable Maximum Flood

The potential for river flooding at the CPNPP Units 1&2 Site due to the PMF was evaluated using a HEC-RAS (Reference 16) model to determine water surface flood elevations during the PMF on the Squaw Creek and the Paluxy River watersheds. The HEC-RAS model for Squaw Creek and the Paluxy River is shown on ***Figure 3-10***.

With regards to the construction of the HEC-RAS model, contraction coefficients of 0.1 and expansion coefficients of 0.3 were used for normal flowing cross sections. When an obstruction was encountered, such as a bridge, a contraction coefficient of 0.3 was used with an expansion coefficient of 0.5 for cross sections immediately upstream and downstream of the bridge. A Manning's n roughness coefficient of 0.04 was used for the Paluxy River stream channel after model calibration (used for natural streams, clean straight, full stage, no rifts, or deep pools, with stones and weeds). A Manning's n roughness coefficient of 0.095 was used for the Squaw Creek stream channel through both reaches after model calibration, based on a coefficient of 0.10 (used for very weedy reaches, deep pools, or floodways with heavy stands of timber and brush). A Manning's n roughness coefficient of 0.15 was used for the overbanks of the Paluxy River and for the overbanks of both reaches of Squaw Creek, which is consistent with the approach of the FSAR for CPNPP Units 1&2 (Reference 2). A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-4 and 3-5***, respectively.

Flow was assumed to be steady and one-dimensional for this analysis and, hence, a steady state analysis was conducted, which is a conservative assumption. Since both supercritical and subcritical flows were anticipated, a mixed flow regime was assumed. The upstream boundary condition was set at normal depth, whereas, the downstream boundary condition was set at critical depth for both streams. The storage area was set at an elevation of 775 ft, representing the operating water surface elevation on the SCR (Reference 2), and the runoff rates resulting from the PMP events described in ***Section 3.2.2.1*** were used. The SSI is connected to the SCR via an equalization channel; the bottom of the channel is at 769.5 ft, 6 in. below the normal minimum operating level. During a PMF event, the water level in the SSI is the same as the water level in the SCR in a steady state model due to the presence of the equalization channel.

HEC-RAS can model an inline structure as only an ogee crest or a broad crested weir. However, the Squaw Creek Dam's two spillways represent both. Therefore, the service spillway and the

emergency spillway are modeled two different ways in two geometry files to account for the differences in water surface elevations obtained for each type.

The HEC-RAS model was calibrated to USGS gauge 08091500 and USGS gauge 08091750. The model calibration was performed by inputting the maximum recorded discharge at the two gauges in the Paluxy River and Squaw Creek and comparing the model predicted water surface elevation with the observed gauge heights. The model is considered to be adequately calibrated with respect to these gauges as the model predicted heights are within 10 percent of the observed gauge heights for the respective river water levels.

The scenarios considered in the HEC-RAS model are as follows:

- **Scenario 1**: No loss, no rainfall-to-runoff transformation, no routing through the channels – Most conservative case.
- **Scenario 2**: No loss, include transformation, no routing through the channels.
- **Scenario 3**: No loss, include transformation, include routing through the channels.
- **Scenario 4**: Same as Scenario 3, but with losses and existing conditions – Existing conditions.
- **Scenario 5**: Same as Scenario 4, but with assumed 5 percent increase in percent impervious for each sub-basin, 50 years from the present – Projected conditions.

Of particular interest is the predicted maximum water surface elevation at the Squaw Creek Dam, which provides information to determine if river flooding is expected to impact the CPNPP Units 1&2 Site. Water surface elevations at the Squaw Creek Dam for Scenarios 1 through 5 with the broad crested weir plan are 794.80, 792.06, 792.06, 791.80, and 791.81 ft, respectively. Water surface elevations at the Squaw Creek Dam (cross section 24626) for Scenarios 1 through 5 with the ogee crested weir plan are 792.82, 790.61, 790.61, 790.41, and 790.42 ft, respectively. Thus, it can be concluded that the maximum predicted water surface elevations for all scenarios as represented by water surfaces given at the Squaw Creek Dam will not exceed the site grade of 810 ft, the SWIS operating deck at 796 ft, or the SSI Dam with an elevation of 796 ft (Reference 2). The reevaluated PMF flood elevations are presented along with the current design basis flooding elevations in ***Table 3-3***. HEC-RAS water surface profiles for Squaw Creek are shown on ***Figure 3-10***. The water surface profiles include results for the scenarios based on existing conditions and projected conditions (Scenario 4 and Scenario 5, respectively).

The still water level reported for the river flooding hazard is based on a steady state HEC-RAS model. The dam failure flooding hazard (Section 3.2.3) also included the evaluation of the PMP on the SCR using an unsteady (transient model). This resulted in a peak simulated water level of 789.88 ft in the SCR and 791.39 ft in the SSI for the most conservative case of the dam failure hazard and considering no obstruction to the equalization channel.

Under the current licensing basis, if water levels in the SCR exceed 778 ft, flooding of the Electrical and Control Building from the Circulating Water System located within the Turbine building can potentially occur when the system is open for maintenance (i.e., one of the condenser discharge butterfly valves located between the condenser discharge and the discharge tunnel to SCR is removed from the system (Reference 2; Reference 5). The Circulating Water System is a closed system during plant operation and flood protection is only required when the system is open for maintenance. The reevaluated water levels will result in the same potential flood causing mechanism for the Electrical and Control Building with the Circulating Water System open. The rate that SCR will rise from the 777 ft to the 778 ft water level under the current licensing basis is approximately 5 hrs (Reference 2). Existing Severe Weather Abnormal Procedures provide response guidance when the SCR water level increases and the system is open for maintenance. An assessment of the impact the reevaluated flood levels have on the response procedures is provided in Section 4.3.2.

3.2.2.2.4 Wind Waves and Run-up Coincident with Probable Maximum Flood

The maximum water level due to wind-wave activity coincident with the PMF on the SCR and SSI was determined in order to assess the potential impact to SSCs at the CPNPP Units 1&2 Site. Statistical presentations of the wind speed and wind direction at the CPNPP Units 1&2 Site are included on ***Figure 3-11***. The wave setup and run-up generated by a two-year return period wind speed were added to the PMF still water elevation to determine the maximum water level at CPNPP Units 1&2 Site. The use of a two-year return period wind speed is consistent with the approaches presented in NUREG/CR-7046 (Reference 7). Methodologies used to determine the wind wave activity are based on the USACE Coastal Engineering Manual (CEM) (Reference 17).

As recommended in the CEM, the longest possible straight line fetches over the SCR and SSI were measured across topographic contours which would be underwater during the PMF (Reference 17). Wave run-up was determined for the scenarios (Scenarios 1 through 5) utilized and described for the determination of the still water level due to the PMF (***Section 3.2.2.2.3***).

The maximum predicted water levels due to the PMF after run-up were evaluated for Scenarios 1 through 5 on the Circulating Water Intake Structure (CWIS) side and the southern SWIS embankment side of CPNPP Units 1&2, as well as on the SSI Dam from both the eastern SCR side and the western SSI side. Maximum predicted water levels were also evaluated for the vertical face of the SWIS. Wave run-up was not evaluated on the Squaw Creek Dam because it is not a safety-related structure for CPNPP Units 1&2. In each case, the wave direction has been assumed to occur from the direction of longest fetch, which was measured over the relevant distance over the surface of the SCR/SSI. The longest fetch on the CWIS side of CPNPP Units 1&2 is approximately 15,113 ft and approximately 1,857 ft on the SSI towards the SWIS. The longest fetches on the SCR and SSI side of the SSI Dam are approximately 6,473 ft and 4,164 ft, respectively. ***Figure 3-12*** shows the fetch lengths on the SCR and SSI. Computed water surface elevations after run-up are reported in ***Table 3-3***.

The water level after wave run-up from the CWIS side represents the peak water level for the plant site and does not reach the Site grade of 810 ft for Scenarios 1 through 5. The power block, hence, will not be flooded by wind wave activity coincident with regional PMF at the Site. The water surface will not reach the SWIS operating deck of 796 ft for a 2 percent run-up for Scenarios 2 through 5. Also, water levels do not reach an elevation of 796 ft for the vertical face of the SWIS for Scenarios 2 through 5.

The water level after wave run-up will not overtop the SSI Dam, with a crest at an elevation of 796 ft, for Scenarios 2 through 5. The water level under Scenario 1 overtops the SSI Dam for run-up from the SCR and SSI sides. Scenarios 1 through 3 are conservative; therefore, it is concluded that CPNPP Units 1&2 and its safety-related structures will not be affected by flooding due to coincident wind wave activity at the Site based on model results of Scenarios 4 and 5.

A comparison of relevant assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-4 and 3-5***, respectively.

The difference in the reevaluated water elevations (***Table 3-3***) due to coincident wind wave activity to those in the FSAR for CPNPP Units 1&2 (Reference 2) is mainly due to the lower fetch lengths used to determine wind setup. Longer fetches and recent contours are used in the reevaluation, providing higher wave setup and runup results, and hence more conservative results.

3.2.3 Dam Failure Flooding

The potential flooding effects on CPNPP Units 1&2 Site due to dam breach occurrences on the Brazos River were evaluated. The SSI dam is located downstream of safety related structures and is a Seismic Category I structure. Therefore, the analysis of dam failure did not include the structural failure of the SSI dam. Failure of the dam was limited to overtopping, which could create a longer fetch distance in the SSI. There are no dams located upstream of SCR on Squaw Creek. Although there are a number of small dams located upstream of the confluence in the Paluxy River watershed, failure effects at the confluence to the Brazos River from any combination of these structures would not exceed more critical dam failure permutations as discussed below on the Brazos River (Reference 3). A combination of steady state and unsteady state HEC-RAS models were developed to evaluate the flood elevations at the Squaw Creek Reservoir during the postulated dam breach occurrences on the Brazos River.

Of the numerous dams on the Brazos River, the controlling dam failure scenario includes the overtopping domino-type failures of Fort Phantom Hill, Cedar Ridge Reservoir, Morris Sheppard, and De Cordova Bend dams. Overtopping of the Lake Stamford Dam is included in the Cedar Reservoir Dam failure (Reference 3). ***Figure 3-13*** shows the locations of these dams. Because all the dams are located on the Brazos River, the flow from the postulated dam failure will only affect the site as backwater to the downstream side of the SCR dam from the confluence point of the Brazos and Paluxy Rivers. The coincident breach of the dams along with the PMF results in an estimated flow of 6,730,000 cfs at the confluence of the Paluxy River with the Brazos River. This flow rate is conservative not taking attenuation into account.

The unsteady HEC-RAS model for Squaw Creek is illustrated on ***Figure 3-14***. The downstream boundary condition for the unsteady model is set at a constant elevation based on the result from a steady-state simulation (using HEC-RAS) of the Brazos River with a flow rate of 6,730,000 cfs (Reference 3). For the unsteady HEC-RAS model, the Squaw Creek Dam is represented as an inline structure located below the SCR. For this analysis, the HEC-RAS flow cases are run by modeling the emergency and service spillways on the Squaw Creek Dam conservatively represented as a broad crested weir with a weir coefficient of 2.63 (Reference 18).

A Manning's n roughness coefficient of 0.1 was used for within bank flow along the Brazos River and 0.095 for Squaw Creek, and Manning's n roughness coefficients of 0.15 are used for the overbanks of Squaw Creek and the Brazos River. These values are conservative and are consistent with the FSAR for CPNPP Units 1&2 (Reference 2). A comparison of relevant

assumptions and analytical inputs used in the reevaluation to those of the current design basis are provided in ***Tables 3-4 and 3-5***, respectively. A contraction coefficient of 0.1 and an expansion coefficient of 0.3 are used for normal flowing cross sections (Reference 18). Bridges present on the Brazos River and ~~in~~ Squaw Creek were not modeled, as they do not obstruct the large flows of the PMF. The runoff rates resulting from the PMP events discussed in ***Section 3.2.2.2*** are used as input for the dam failure flooding.

As per the HHA procedure for dam failure analysis (***Figure 3-15***), two scenarios were simulated in HEC-RAS with decreasing levels of conservatism. The scenarios incorporated into the HEC-RAS model are as follows:

- **Scenario 1**: No loss, no rainfall-to-runoff transformation. The boundary conditions for the downstream side of the SCR Dam in this simulation are set to a constant stage of 760.45 ft (Reference 3).
- **Scenario 2**: Like Scenario 1, except that runoff transformation is included in the analysis.

The maximum water surface elevations at the Site for the most conservative case (Scenario 1) are 789.88 ft in the SCR and 791.39 ft in the SSI, which will not flood the CPNPP Units 1&2 Site at 810 ft. The SWIS is at 796 ft and the crest of both the Squaw Creek Dam and SSI Dam is at 796 ft (Reference 2). The SWIS is not flooded and the Squaw Creek and SSI dams are not overtopped. It should be noted that the maximum water levels reported herein are as a result of the PMF in conjunction with the downstream boundary conditions on the Brazos River under the postulated dam failure. The bounding computed water surface elevations are reported in ***Table 3-3***. A flood level time history of the SCR for the dam failure analysis is provided on ***Figure 3-16***.

The FSAR for CPNPP Units 1&2 only reports the water surface elevation downstream of the Squaw Creek Dam. The reported water surface elevation is 700 ft (Reference 2). The current analysis provides the SCR and SSI water surface elevations and is more representative of the water surface elevation at the Site. Therefore, the reevaluated water levels are not comparable to those reported in the FSAR for CPNPP Units 1&2. The water surface elevation on the backwater downstream side of the SCR dam due to coincident dam failures on the Brazos River is less than the maximum SCR and SSI water surface elevations.

3.2.3.1 Wind Waves and Run-up Coincident with Dam Failure Flooding

The wave setup and run-up generated by a two-year return period wind speed were added to the SCR and SSI PMF still water level developed in the dam failure analysis, as recommended by ANSI/ANS-2.8-1992 (Reference 19). The use of a two-year return period wind speed is consistent with the approaches presented in NUREG/CR-7046 (Reference 7). The maximum predicted water levels after run-up were evaluated for the scenario discussed in ***Section 3.2.3*** on the northern CWIS side and the southern SWIS side of CPNPP Units 1&2, as well as on the SSI Dam from both the eastern SCR side and the western SSI side. Maximum predicted water levels were also evaluated for the vertical face of the SWIS and the SWIS embankment. The wave run-up on the Squaw Creek Dam was not evaluated because it is not a safety-related structure. In each case, the wave direction has been assumed to occur from the direction of longest fetch, which was measured over the relevant distance over the surface of the SCR/SSI. The bounding computed water surface elevations after run-up are reported in ***Table 3-3***.

The water level after wave run-up from the CWIS side does not reach the Site grade of 810 ft for the worst case scenario in the HHA approach. The power block, hence, will not be flooded by wind wave activity coincident with dam failure PMF at the Site. The water surface will not reach the SWIS operating deck of 796 ft for a 2 percent run-up. Also, water levels do not reach an elevation of 796 ft for the vertical face of the SWIS pump room.

The water level after wave run-up will not overtop the SSI Dam, with a crest at an elevation of 796 ft from both the SCR and SSI sides. Hence, it is concluded that SSCs at the CPNPP Units 1&2 will not be affected by flooding due to wind wave activity coincident with the dam failure PMF. It is noted that reevaluated SCR and SSI water levels coincident with dam failure flooding including run-up are bounded by the water levels due to river flooding discussed in ***Section 3.2.2.2.4***.

The FSAR for CPNPP Units 1&2 did not evaluate wave run-up levels due to dam failure PMF at the Site (Reference 2). Thus, a comparison of the reevaluated results to the results of the FSAR for CPNPP Units 1&2 is not appropriate.

3.2.4 Storm Surge and Seiche Flooding

The CPNPP Units 1&2 Site is not near any large bodies of water for which surge or seiche flooding would apply; therefore, the risk to the Plant from a storm surge and/or a meteorological seiche event that could cause flooding at the Site is not expected to be a potential flooding hazard. This is substantiated by the seiche evaluation performed in Reference 3. The evaluation determined that meteorological seiches are not likely to occur and that a seismic seiche would generate a wave no more than 5 ft high in the Squaw Creek Reservoir, which when added to the normal operating level of the SCR, is well below the PMF for the Site. Additionally, the potential for landslide induced seiches and waves were evaluated in Reference 3 and determined to not be plausible based on a slope stability analysis of the Squaw Creek Reservoir.

With respect to storm surge due to the probable maximum hurricane, according to ANSI/ANS-2.8-1992 guidance, the region of occurrence of a hurricane shall be considered for U.S. coastline areas and areas within 100 to 200 miles bordering the Gulf of Mexico (Reference 19). Because CPNPP Units 1&2 are located greater than 250 miles inland from the Gulf of Mexico, and at a site grade of 810 ft, hurricanes are not expected to be a potential flooding hazard. Surge and wave action due to the PMF on rivers and for the combined events hazards, which are the limiting hazards, are discussed in Subsections 3.2.2.2.4 and 3.2.8.1, respectively.

3.2.5 Tsunami Flooding

The CPNPP Units 1&2 Site is not near any large bodies of water for which tsunami flooding would apply. As the CPNPP Units 1&2 Site is over 250 miles from the Gulf of Mexico, and the plant grade is over 800 ft above sea level, tsunami flooding is not a risk to the Site. Furthermore, NRC guidance indicates that if regional screening identifies that the site region is not subject to tsunamis, no further analysis for tsunami hazard is required (Reference 20).

3.2.6 Ice Flooding

CPNPP Units 1&2 are located in a temperate zone in which monthly low temperatures do not typically remain below freezing long enough to allow for significant ice thickness to form on lakes, streams, and rivers. However, the potential for ice formation and associated river flooding due to ice jam(s) was evaluated.

Recorded minimum water temperature data were examined for two gauging stations, 11856 on the Brazos River and 11555 on Squaw Creek, to determine if the formation of ice could cause a potential flooding hazard to the Site. The lowest recorded water temperature was 39.0 degrees Fahrenheit at the Brazos River at Station 11856 in January 1970 (Reference 21). The lowest recorded water temperature at the Squaw Creek station (11555) was 41.9 degrees Fahrenheit in February 1982 (Reference 21). Hence, the SCR is expected to remain well above freezing point throughout the year. Additionally, the potential for frazil ice, which forms in supercooled turbulent waters in rivers and lakes (Reference 22), is remote, as evidenced by the lowest recorded water temperatures discussed above.

The lowest historical air temperatures in the region of CPNPP Units 1&2 were also considered. Historical air temperatures recorded at the Fort Worth Meacham station for the recorded periods between 1946 and 2012 (Reference 23) were examined. The maximum potential ice thickness was estimated using the historic air temperature data from the Fort Worth Meacham station (Reference 23). According to USACE methods, theoretical ice thickness was determined by analysis of Freezing Degree Days (FDD), defined as the difference between 32 degrees Fahrenheit and all recorded daily air temperatures below freezing (Reference 24).

The maximum theoretical ice thickness calculated using the above approach is of an insufficient thickness to cause an obstruction on the SCR, which has an average depth of 46 ft. The maximum calculated ice thickness is a conservative estimate, since the temperatures taken into account during the calculations are daily mean temperatures at the station. For the reasons discussed above, ice effects are not expected to be a potential threat to safety-related SSCs at CPNPP Units 1&2.

3.2.7 Channel Diversion Flooding

The potential for upstream diversion or rerouting of the source of cooling water for CPNPP Units 1&2 due to channel diversions was evaluated. Local topography, bathymetry, and geology with the potential to influence morphological changes were considered. The assessment included a review of historical information regarding streams and rivers in the vicinity of the Site.

A qualitative assessment of the streams and rivers in the vicinity of the CPNPP Units 1&2 Site was undertaken in conjunction with a review of information presented in the CPNPP Units 1&2 FSAR. The Squaw Creek watershed has developed streams with distinct valleys and has sustained numerous farm ponds and goes on to state that diversion of water from the watershed appears impossible (Reference 2). There is no evidence suggesting there have been significant historical diversions or realignments of Squaw Creek or the Brazos River and that the topography does not suggest potential diversions (Reference 3). The streams and rivers in the region are characterized by valleys with no steep, unstable side slopes that could contribute to landslide cutoffs or diversions and there is no evidence of ice-induced channel diversion (Reference 3).

The SCR is situated on the Glen Rose formation, which is a predominantly limestone sequence and has been indicated to be relatively impermeable and free of sinkholes and solutioning (Reference 2; Reference 5). There is no evidence of active karst conditions and related subsidence within the site or in the surrounding area (Reference 3). Thus, significant loss of water through the base of the reservoir is improbable (Reference 2). Lake Granbury, which is on the Brazos River, is a source of supplemental makeup cooling water for CPNPP Units 1&2 to further enable the SCR to be held at the required operating level (Reference 2). The loss of makeup water from Lake Granbury due to the diversion of the Brazos River is highly improbable because the Brazos River channel above Lake Granbury is cut into bedrock largely precluding the possibility of the river changing its channel significantly within the life of the CPNPP (Reference 2; Reference 5).

Channel diversion due to subsidence is not expected, as withdrawal of groundwater at current rates from aquifers beneath the site does not present a risk of subsidence (Reference 3). Channel diversion due to geothermal activity is not expected and the site area is relatively tectonically stable and has experienced no volcanic activity recent enough to produce geothermal heat (Reference 3).

3.2.8 Combined Events

The water level due to combined event precipitation flooding was determined, as recommended by Section 9.2.1.1 of ANSI/ANS-2.8-1992 (Reference 19) and NUREG/CR-7046 guidance (Reference 7). The combined events analysis was based on precipitation flooding due to the inland location of the CPNPP Units 1&2 Site within a watershed with no dams present. The aforementioned guidance recommends three different alternatives consisting of a combination of events. Because the CPNPP Units 1&2 Site is located in a region for which probable maximum or 100-year snowpack is not relevant, the following combined events were evaluated step by step:

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

The method used for the FSAR for CPNPP Units 1&2 (Reference 2) did not require a combined events analysis. The mean monthly base flow for the Paluxy River and Squaw Creek were obtained from USGS gauge 08091500 and USGS gauge 08091750, respectively. The highest of the average monthly flows were used as the base flow for conservatism. The base flow measurements from the USGS gauges were adjusted to each sub-basin of Squaw Creek and the Paluxy River based on the gauge area and sub-basin area. The adjusted base flows were then utilized in the HEC-HMS models for the Paluxy River and Squaw Creek, which are discussed in ***Section 3.2.2.2***.

The 72-hour, 5-minute interval PMP hyetographs for the Paluxy River and Squaw Creek calculated using HMR 52, and discussed in ***Section 3.2.2.1***, were utilized. Antecedent rain equal to 40 percent of the PMP event for 72-hours followed by a PMP event for 72-hours (6-day, 5-minute incremental PMP) was used as input into the HEC-HMS models for the Paluxy River and Squaw Creek along with the adjusted base flows. Because the 500-year rainfall is lesser than the 40 percent PMP, the use of the 40 percent PMP yields more conservative results. The HEC-HMS models were run for the following scenarios as per the HHA procedure for combined events flooding (***Figure 3-17***).

- **Scenario 1**: No loss, no rainfall-to-runoff transformation, no routing through the channels – Most conservative case. Coincides with Scenario 1 presented in ***Section 3.2.2.2.2*** and Scenario 10 presented in ***Section 3.2.2.2.1***.
- **Scenario 2**: Include transformation, no routing through the channels, and no losses – Only transformation. Coincides with Scenario 2 presented in ***Section 3.2.2.2.2*** and Scenario 11 presented in ***Section 3.2.2.2.1***.
- **Scenario 3**: Include transformation; include routing through the channels, but no losses – Only transformation and routing. Coincides with Scenario 3 presented in ***Section 3.2.2.2.2*** and Scenario 12 presented in ***Section 3.2.2.2.1***.
- **Scenario 4**: Include transformation, include routing through the channels, and include losses for existing land use – Existing conditions. Coincides with Scenario 4 presented in ***Section 3.2.2.2.2*** and Scenario 13 presented in ***Section 3.2.2.2.1***.
- **Scenario 5**: Include transformation, include routing through the channels, and include losses for projected land use – Projected conditions with an assumed 5 percent increase in percent impervious for each sub-basin. Coincides with Scenario 5 presented in ***Section 3.2.2.2.2*** and Scenario 14 presented in ***Section 3.2.2.2.1***.

The PMF values determined using the approach described above were input into the HEC-RAS models for the Paluxy River and Squaw Creek, with the other inputs and parameters (discussed in ***Section 3.2.2.2.3***) remaining the same. The two-year return period wind speed was used to compute the wave run-up. The use of a two-year return period wind speed is consistent with the approaches presented in NUREG/CR-7046 (Reference 7). The water levels obtained using the HEC-RAS models were used as input to find the depth to determine the wave run-up.

The maximum predicted water surface elevations due to combined events flooding for Scenarios 1 through 5, which are represented by water surfaces given at the Squaw Creek Dam cross section, are 795.02, 792.25, 792.25, 792.23, and 792.23 ft, respectively. There is no variation in the water surface between the existing and projected conditions because the difference in flows between the two scenarios is minimal and does not cause a significant difference in water surface elevation at the SCR. A flood level time history of the SCR for the combined events analysis is provided on ***Figure 3-18***.

The effect of the Squaw Creek watershed combined event precipitation depth duration curve on water levels within the SSI was also evaluated. Three simulations were considered as part of the HHA approach to study the impact on flood levels in the SSI due to debris blockage of the equalization channel. In the first simulation, the equalization channel was represented as fully

blocked to an elevation of 796 ft. In the second simulation the equalization channel was represented as partially blocked to an elevation of 790 ft. In the third simulation, the equalization channel was considered fully functional and un-blocked. The unblocked condition of the equalization channel under the third simulation is consistent with the assumptions of the FSAR for CPNPP Units 1&2 (Reference 2). With the equalization channel unblocked, the simulated peak water level of the SSI and SCR was approximately 792.2 ft with peak flow through the equalization channel of 19,589 cfs. The simulations conducted indicated that with normal operation of the equalization channel, water levels in the SSI would not reach an elevation of 796 ft (i.e., the elevation of the SWIS) during the PMP event.

3.2.8.1 Wind Waves and Run-up Coincident with Combined Events Flooding

The maximum predicted water levels due to combined events flooding after run-up were evaluated for Scenarios 1 through 5 on the northern CWIS side and the southern SWIS side of CPNPP Units 1&2, as well as on the SSI Dam from both the eastern SCR side and the western SSI side. Maximum predicted water levels were also evaluated for the vertical face of the SWIS pump room and the SWIS embankment. The wave run-up on the Squaw Creek Dam was not evaluated because it is not a safety-related structure. In each case, the wave direction has been assumed to occur from the direction of longest fetch, which was measured over the relevant distance over the surface of the SCR/SSI. Computed water surface elevations after run-up are reported in ***Table 3-3***.

The water level after wave run-up from the CWIS side does not reach the Site grade of 810 ft for Scenarios 1 through 5. The power block, hence, will not be flooded by wind wave activity coincident with regional PMF at the Site. The water surface will not reach the SWIS operating deck of 796 ft for a 2 percent run-up for Scenarios 2 through 5. Also, water levels do not reach an elevation of 796 ft for the vertical face of the SWIS for Scenarios 2 through 5.

The water level after wave run-up will not overtop the SSI Dam, with a crest at an elevation of 796 ft, for Scenarios 2 through 5. Scenarios 1 through 3 are conservative, and it is concluded that CPNPP Units 1&2 and its safety-related structures will not be affected by flooding due to coincident wind wave activity at the Site.

A comparison to the FSAR for CPNPP Units 1&2 (Reference 2) is not appropriate because the water level due to coincident wind wave activity reported in the FSAR is not based on a combined event PMF.

4.0 COMPARISON OF CURRENT AND REEVALUATED PREDICTED FLOOD LEVELS

***Section 4.0* has been prepared in response to Requested Information Item 1.c. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (Reference 1). Provide a comparison of current and reevaluated flood causing mechanisms at the site. Provide an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism. Include how the findings from Enclosure 4 of this letter (Le., Recommendation 2.3 flooding walkdowns) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, include 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 are river flooding, including wave run-up at the Site and dam failure flooding, storm surge and seiche flooding, tsunami flooding, ice flooding, and channel diversion flooding. Flood causing mechanisms considered under the flooding reevaluation include local intense precipitation, river flooding including wave run-up, dam failure flooding, storm surge and seiche flooding, tsunami flooding, ice flooding, channel diversion flooding, and combined events flooding.

The primary differences between the current and reevaluated flood causing mechanisms are that water levels were not determined for a 1-hour 1-mi^2 local intense precipitation storm event and combined events under the current licensing basis. The current design basis considers a 48-hour PMP, while the reevaluation considers a72-hour PMP. Additionally, dam failure flooding under the current licensing basis reports the maximum water level downstream of the Squaw Creek Dam and not at the Site. Fetch lengths considered in the wave run-up analyses differ under the reevaluation and current design basis. The reevaluated fetch lengths are across the longest possible fetch whereas the current design basis considers the longest fetch along the critical wind direction only. The longer fetches used in the reevaluation provide more conservative results. The potential for storm surge and seiche flooding, tsunami flooding, ice flooding, and channel diversion flooding is qualitatively dismissed under both the current design basis and reevaluated analyses.

4.2 ASSESSMENT OF THE CURRENT DESIGN BASIS FLOOD ELEVATIONS TO THE REEVALUATED FLOOD ELEVATIONS

The current design basis, as presented in the FSAR for CPNPP Units 1&2, indicates that all safety-related structures, except the SWIS and the Electrical and Control Building, are not subject to flooding, wave action, or wave run-up and do not require flood or wave protection (Reference 2). The CPNPP Units 1&2 grade is at elevation 810 ft while the peak SCR water level is at 789.7 ft for the PMF (Reference 2). The maximum calculated water level within the SSI during the PMF is 790.5 ft, which leaves a freeboard within the SSI of 5.5 ft with respect to the SWIS operating deck elevation of 796.0 ft (Reference 2). The maximum SCR wave run-up and setup elevation at the CPNPP Units 1&2 Site due to coincident wind wave activity is 794.7 ft (Reference 2). The maximum wave run-up and setup elevations at the Squaw Creek Dam and SSI Dam are 793.7 and 791.3 ft, respectively (Reference 2).

In regard to the effects of the local intense precipitation event, the FSAR for CPNPP Units 1 & 2 does not present the PMP generated by a 1-hour 1-mi^2 storm. Water level effects are instead based on HMR No. 33 methods and conclude that the onsite drainage system is designed to remove the water resulting from the limiting PMP event in such a manner that there are no ponds which can back up into the safety related SSCs (Reference 2). Exterior entrances which could provide a propagation pathway into the safety-related structures are above grade at elevation 810.5 ft or greater. One entrance on the west face of the Unit 2 non-safety related Turbine Building (TB) is less than elevation 810.5 ft and has the potential to communicate local intense precipitation runoff interior to a lower TB elevation which is located adjacent to the safety related Electrical Control Building at the 778 ft elevation. This event is discussed further in Section 4.3.2.

A list of design parameters found in the license document (Reference 2) is included in ***Table 2-3***. A list of CPNPP Units 1&2 licensed water levels and corresponding flooding mechanisms can be found in ***Table 2-4***.

All reevaluated flood levels for each individual flood causing mechanism are discussed in ***Section 3.0***. A comparison of all of the current design basis flooding elevations and each of the reevaluated flood elevations for each flood causing mechanism presented in Attachment 1 to NRC Recommendation 2.1 (Reference 1) is provided in ***Table 3-3***.

Although the reevaluated flood levels for river flooding do not exceed the plant grade or that of the SSCs, they do however exceed the current design basis flood levels (***Table 3-3***). Similarly,

the reevaluated flood levels, including wave run-up within both the SCR and the SSI, exceed the current design basis flood levels (***Table 3-3***).

The reevaluated effects of the Local Intense Precipitation event show that the peak ponding water elevation adjacent to safety related buildings slightly exceeds the plant grade but does not exceed entrance elevations of any safety related building in such a manner that water can back up into the safety related SSCs. ***Table 3-2*** provides simulated water levels and ponding depths due to the effects of a Local Intense Precipitation event.

4.3 SUPPORTING DOCUMENTATION

Calculation briefs in support of the flooding hazard reevaluation at CPNPP Units 1&2 have been prepared, on which the reevaluated flood levels are based. Additionally, the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) provides further information regarding the design basis flood hazard levels, as well as flooding protection and mitigation features.

4.3.1 Technical Justification of the Flood Hazard Analysis

All reevaluation flooding analyses described in this Report have been undertaken with consideration to and implementation of current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The technical basis for the various scenarios modeled under the HHA approach and the key assumptions utilized in the determination of the reevaluated flooding levels for each flood causing mechanism are discussed individually in ***Section 3.0***.

4.3.2 Technical Justification by the Walkdown Results

With respect to the implementation and conclusions of the flooding hazard reevaluation, results from the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) have been taken into consideration.

Firstly, "Several changes to the plant layout were identified during the walkdown which did not have documented flooding impact evaluations nor were they explicitly described in the FSAR. Based on field observations, the alterations to the topography by the modifications do not adversely affect the runoff assumed in the current licensing basis."

The CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) describes, "planned or newly installed flood protection systems or flood mitigation measures, including flood barriers that further enhance the flood protection." The flood protection system and flood mitigation measures described in Section 2.6 and the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report are specifically relevant to the flooding hazard reevaluation analyses.

Based on the reevaluated flood hazard results, the effects of flood levels that are not bounded by the current licensing basis on the pertinent flood protection and mitigation features described in Section 2.6 and as identified in the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) are provided below.

SSI Dam:

The SSI is designed to serve as the Ultimate Heat Sink of the CPNPP Units 1 & 2 and acts to dissipate heat rejected by the Station Service Water System during post accident shutdown and normal cooldown conditions. The Seismic Category I designed Safe Shutdown Impoundment Dam is a rockfill dam with an impervious earth core. The entire foundation was excavated to unweathered limestone to provide suitable support for the rockfill shells and to provide an impervious base for the core. The outer surfaces or shells of the dam consist entirely of selectively quarried limestone rock and processed to remove all claystone particles which could subject the surface to erosion and is designed with a minimum thickness of 10 feet. This minimum thickness occurs only at the crest of the dam and increases rapidly below the crest to a maximum of more than 100 ft (each shell surface) at the base of the embankment. The width of the crest at elevation 796 ft is 40 ft with an outer surface slope of 2.5:1 which is flatter than most dams of higher elevation in more seismically active areas. The water level of the SSI is maintained by the equalization channel between the SSI and the SCR. (Reference 2).

The limiting reevaluated PMF + Wave runup value of 795.91 ft on the SCR side of the SSI dam and 795.11 ft on the SSI side of the dam (Refer to Combined Event results in Table 3-3) will have minimal impact to the integrity and the UHS function of the SSI dam. Neither water level will cause the dam to be overtopped across the 40 ft wide crest. The relatively flat rockfilled outer slope up to the 796 ft crest elevation will dissipate the wave action dynamic forces and when combined with the lack of fine particles within the rockfill, erosion potential is negligible. Therefore, SSI dam degradation or erosion to the point that the PMF wave run-up attributed to the SCR can exceed the top of the SSI dam is precluded, the fetch distance for wave run-up within the SSI is unchanged and there will be no impact to the SWIS structure.

As observed in the flooding walkdowns, planned annual inspections of the dam are performed which specifically include observations of slope protection that help ensure continued performance of the SSI dam if such a PMF event where to occur.

Additionally, due to the placement of the SSI in an inlet that is not in the flow path of flood waters flowing down Squaw Creek (Refer to Figure 2-2, "As-built Site Layout and Topography," and Figure 3-12, "Fetch Locations over the Squaw Creek Reservoir and Safe Shutdown Impoundment"), any debris flowing down the Squaw Creek channel during the PMF will collect at the SCR dam and locations downstream of the SSI dam. The collection of debris within the much smaller SSI from the Panther Branch watershed will be non-impactive due to the material of construction, 40' broad top-of-dam crest and relatively flat slope design of the dam's rockfill outer surface. Combined with the seismic design of the dam, dynamic loading due to debris on the dam's overall integrity and capability to satisfy its UHS design function is negligible.

The presence of the equalization channel ensures that the reevaluated PMF of 792.23 ft is equal on both sides of the SSI dam. Considering the maximum Wave Runup value on either side of the dam and the PMF level, the maximum elevation differential between water levels on either side of the SSI dam during the reevaluated PMF event is 3.68 ft (795.91 – 792.23). Combined with the inherent seismically stable design of the structure, the relatively flat slope and broad crest and base of the dam, any hydrodynamic load effects to the dam structure will be balanced out and within the original design margin for overall stability of the dam.

Likewise, in the unlikely event the non-seismic SCR dam fails and rapidly drains, the equalization channel will allow the SSI to drain down freely to the same level as the SCR but no lower than the invert elevation of the channel (i.e, 769.5 ft). This is within the normal design of the SSI dam considering no tail water conditions and still maintains the design basis reserve required to allow continued cooling and safe shutdown of the plant.

In addition, a nonintrusive, floating raft system has been located near the south end of the equalization channel. Should the equalization channel experience flooding conditions, the floating raft system is designed to freely float on the water surface, even if the floats are punctured. If the water level exceeds an elevation above approximately elevation 781 feet, the raft system will float over the bollards used to help retain the raft in position and, due to the direction of flow through the channel, the raft system will be pushed away from the SSI and into the SCR. It was confirmed during the reevaluation that the direction of the flow in the equalization channel is from the SSI to the SCR for both the river flooding hazard and combined events hazard on the

Squaw Creek watershed. Therefore, the floating raft system will not become debris that could impact the SSI dam or SWIS. Furthermore, channel blockage due to the raft is considered unlikely. Consistent with the current licensing basis, this floating raft system does not affect the design function of the equalization channel to freely pass water to the SCR under flooding conditions up to the reevaluated PMF.

SWIS Structure:

As presented in Section 2.6 and in the flooding walkdown report (Reference 5), groundwater is discussed in the current licensing basis. Hydrostatic loads on the SWIS structure are determined with a design basis groundwater level at elevation 793 feet (Reference 2). From Figure 3-4 and Table 3-2, ponding levels attributed to the reevaluated Local Intense Precipitation (LIP) event cannot occur immediately adjacent to the applicable SWIS exterior walls. Thus, the original structural analyses for hydrostatic load impacts are unchanged.

A peak reevaluated water level of 795.03 ft (Combined Event - SSI PMF + Wave Runup) was determined as provided in Table 3-3. The south vertical wall of the SWIS represents a partial separation between the SSI main body of water and the Service Water pump intake area. The wall has large openings below the normal SSI pool water elevation to allow water to flow freely into the SWIS pump intake area below the 796 ft operating deck, As such, the elevation of the water on both sides of the wall will be equal. Any hydrodynamic pressure loads would balance out on all sides of the SWIS walls subjected to the SSI PMF elevation (SSI PMF = 792.23 ft from Table 3-3) and have negligible structural impact to the integrity of the SWIS structure.

With wave runup, the maximum water elevation difference between the SSI side (795.03 ft) and Intake bay side of the 2.5 ft thick south wall will be 2.80 ft (i.e., Intake bay water level below operating deck at 792.23 ft assuming complete wave damping due to the interfering trash racks, traveling water screens and the south wall). The structural analysis for the SWIS south wall takes into account the 2.0 ft slab thickness of the 796 ft operating deck and conservatively used a flood level of 794.5 ft for the design of the wall. Based on review of the SWIS structural analyses, the controlling load combination for the design of the SWIS, (seismic conditions concurrent with normal water level) is significantly larger than the load combination associated with the consideration of the reevaluated PMF of the SSI.

Additionally, debris loading originating from SCR on the SWIS south wall has not been considered due to the location of inflow (equalization channel) of the flood waters into the SSI being located within an inlet and as discussed previously, the direction of the flow in the

equalization channel is from the SSI to the SCR for both the river flooding hazard and combined events hazard on the Squaw Creek watershed. Flood waters flowing down Squaw Creek will collect debris in the areas around the SCR dam and not at the equalization channel.

Obstruction to the SWIS intake area due to debris collected from within the SSI catchment basin and plant grade runoff after a PMF event is non-impactive and within the design of the SWIS. The SWIS is equipped with two levels of debris barriers located ahead of the water entry through the large submerged openings in the south wall as discussed above. These barriers and supporting structures are designed to Seismic Category I requirements to withstand the effects of natural phenomena and maintain their structural integrity without loss of capability to perform their safety function.

The first trash rack barrier is a large steel grating type structure designed to capture larger debris which may be attributed to watershed runoff or high wind/tornado activity deposition and covers the entire vertical length of the two intake areas (i.e., elevation 755 ft to the 796 ft operating deck). The second set of barriers providing the same vertical coverage are located between the trash racks and the service water pump intake area beneath the operating deck and consist of automatic rotating traveling screens/baskets designed to capture the smaller debris greater than 0.375". The screens are designed with a differential head loss assuming 80% blockage without degrading the design intake flow requirements necessary to support the function of the Service Water pumps for both Units. All electrical support systems required to enable the periodic rotation and screen wash functions are located on the 796 ft operating deck above the PMF level. Automatic control is provided to initiate the screen wash sequence of the traveling screen upon receipt of a high differential water level signal or a timed wash signal even if the high differential setpoint is not established. The screens are not required to operate during an accident or loss of power event. Even so, with the assumed 80% screen blockage considered in the design, more than 35 ft of useable vertical screen surface area submerged below the estimated PMF surface elevation of 795.03 ft and floating debris near the surface of the water being the primary location for collection and potential obstruction, there will be sufficient flow capacity through the screens to support Service Water System pump operation.

Finally, an annual Technical Specification surveillance requires an inspection of SSI sedimentation to be performed which has historically shown minimal deposition of silt and sedimentation within the intake channel area since initial operation of the facility. This can be attributed primarily to an upstream Panther Branch watershed basin having limestone based flow channels with a predominately grassland/ herbaceous type land use not subject to significant

erosion characteristics. Additionally, runoff from the plant site into the SSI has limited potential to erode soil particles. The current plant grade was established after excavating to unweathered limestone. Since the plant grade including onsite drainage features and the embankments around the SWIS are covered with an additional surface layer of crushed limestone rock and rip rap and/or concrete, erosion potential including sediment transport and deposition within the SSI is minimal.

Thus, obstruction to the SWIS intake area due to debris or sedimentation during a reevaluated PMF event and the potential impact to Service Water System operation and SSC functionality including SSI UHS cooling capacity are negligible.

Exterior Walls and Floors (Including Penetrations)

No natural ground water was encountered during original excavation and construction of the plant structures. However, during design validation efforts it was determined that ground water or perched water attributed to frequent rainfall events may exist at elevations higher than 775'-0" (Reference 2). Therefore, safety-related plant structures were designed for hydrostatic loads with the design basis ground water level at elevation 810'-0", except for the Service Water Intake Structure as discussed above. From Figure 3-4 and Table 3-2, only the plant structures having a peak ponding elevation attributed to the reevaluated LIP event > 810 ft are considered in an assessment of an increase in the ground water conservatively using a hydrostatic load attributed to a maximum reevaluated LIP water level of 810.34 ft. Ground water hydrostatic pressure on the lowest safety related building elevation (being 773') would increase by the ratio of (810.34-810) / (810-773) = 0.92%. This additional stress level (with all other contributing loads unaffected) in the affected walls & floor base mats are negligible and well within the design margin provided in the structural integrity analyses for all applicable exterior building wall and floor base mat structural elements.

The additional hydrostatic pressure against associated penetration seal locations in applicable walls and floors will not degrade the integrity and credited function of the seals any greater than that previously observed in the Walkdown effort and as addressed for equipment operability in the existing CAP documentation resulting from those walkdowns (Reference 5). The majority of the penetration seals installed in the exterior sub-grade walls of Seismic Category I buildings are not hydrostatically rated (i.e., not required to be leak tight). Although there are some installed seal types and waterstops at sub-grade concrete joint interfaces that are qualified for the hydrostatic load, seals that are not hydrostatically rated may not totally prevent groundwater intrusion under the LIP event. However, the differences between the as-designed and as-found states as identified

during the flooding walkdowns (Reference 5) would not compromise the overall barrier integrity of the associated walls. The Class 1E, Safety Class 3 sump pumps located in the Safeguards Buildings (SB) were determined to be functional and capable of providing the required amount of minute ground water removal which could leak into the lowest SB area as described in the CLB (even under reevaluated LIP conditions), though the design basis for water removal for the pumps is not ground water intrusion (Reference 5). Any minute groundwater intrusion attributed to the reevaluated LIP ponding levels which are marginally higher than plant grade, would (1) be bounded by the existing design basis internal building flooding analyses and (2) be well within the design rating & operating capacity of the applicable building sumps such that there is reasonable assurance that safety related SSCs are fully capable of performing their specified safety functions.

Seismic Category I Roofs

SWIS Structure:

Unlike the Non-SWIS structures, the SWIS roof design is an open slab design which does not contain parapet walls. Any rainfall event regardless of intensity due to the relatively small footprint and slope will provide direct runoff over the side of the SWIS structure with no measurable holdup on the roof such that the roof design is not impacted by the reevaluated flood hazards. Any hydrostatic load contribution attributed to a reevaluated rainfall event in the roof design is bounded by a more limiting load combination including a design live load roof slab design of 50 psf in combination with either a tornado wind load or seismic acceleration.

Non-SWIS Structures:

Hydrostatic loads resulting from contained fluids with fluctuating levels are considered to be live loads for roof design. A design load on all nuclear safety-related buildings of an eight-inch maximum uniform depth of water (weight of about 42 lbs/ft^2) is considered in addition to the regular live loads.

The roof drainage utilizes an exterior system, whereby the openings in continuous parapet walls are actually combined relief and drainage openings. These parapet wall relief openings ensure that the CLB eight-inch level is not exceeded during the PMP. They are specifically located at all roof low points and extend from the roof low point elevation to the top of the parapet (approximate height four feet). The length of an opening is a minimum of six feet. The size of these openings and their location are intended to preclude roof ponding.

Flat roofs with continuous parapets are designed to consider the hydrostatic load resulting from eight inches of standing water caused by the probable maximum precipitation (PMP). This load is considered to be an abnormal extreme environmental load and is not required to be combined with seismic loading. By design, the live load roof slab design for such buildings is a minimum of 40 psf. Nuclear safety-related buildings are designed with a governing load combination of Dead Loads + Live Loads +Seismic Acceleration which are factored up to provide additional margin. For the Live Load parameter, the standard 40 psf load and is factored up by 1.7 times in the loading equations to a minimum of 68 psf which bounds the 42 psf created by 8 inches of standing water. This additional margin in the roof slab live load design parameter would allow a ponding level of approximately 13" without impacting the structural live load constituent of the overall governing load case combination used to qualify the design of the roof slabs.

Utilizing the maximum 6-hour local intensity precipitation rainfall intensities taken from Figure 3-2 and the physical layout of the various nuclear safety related building roof slabs including the available drainage capacity at each parapet wall opening, there is sufficient runoff capacity available to maintain a water level to < 13" with resulting loads maintained within the design margins inherent to the existing design of applicable roof slabs. In addition, there are no roof slab openings or penetrations given a proposed 13" water level which would allow propagation interior to the safety related buildings and impact applicable SSCs.

Class 1E Cable Vaults/Manholes

Given the proposed LIP inundation levels provided in Figure 3-4 and Table 3-2, four of the fourteen Class 1E cable vaults/manholes may become fully submerged for a short period of time after the event (i.e., conservatively assuming no credit is taken for gasket seals provided at the manhole curb –cover plate interface). All four Class 1E cable vault manholes are associated with Unit 2 and located within catchment areas 21692 and 21695 of Figure 3-4. Two of the manholes provide conduit entry into a Unit 2 safety related building area. However, electrical raceway/conduit design drawings show that these conduits all slope up from the manhole within sub-grade ductbanks and enter the safety related building into the base mat floor slab as embedded conduit, then stub-out above the safety related building's 810.5 ft floor slab top elevation approximately four additional inches to elevation 810.83 ft. The maximum LIP inundation level for this manhole location (i.e., catchments 21692/21695) is 810.12 ft < 810.83 ft. Thus, assuming no credit is taken for any internal conduit penetration seals, there is no available propagation pathway into a plant area containing safety related SSCs.

Additionally, if the cable vaults become fully flooded, medium voltage cables located higher within the cable vaults will be submerged. Industry studies by both NEI and EPRI conclude that cables operating at lower voltages do not generate the necessary electric stress to induce water related degradation. These studies also conclude the primary degradation mechanism concerning medium voltage cables in water takes many years, usually decades, of constant submergence to become problematic. The Class 1E cables within the vaults were originally purchased to safety related specifications which requires them to be suitable for use in wet or dry locations, indoors or outdoors in cable trays, conduits, or underground ducts.

Furthermore, the cables within these vaults are included within an existing Cable Aging Management Program which administratively requires a preventive maintenance inspection of the vaults on a quarterly basis (90 days). These existing PMs establish controls to open the vaults and effectively pump out any collection of standing water to prevent long term submergence of the cables. The effects of the reevaluated LIP event would be governed by these existing controls such as to preclude long term submergence and potential impact to the operability of associated safety related equipment consistent with the NEI and EPRI guidance for low and medium voltage cables.

Manhole Covers

The Fuel Building Service Water Pipe Tunnel manhole cover is mounted flush with the plant grade at 810.0 ft and located near the southeast corner of the Fuel Building. Based on the peak LIP ponding elevation of 809.86 ft as shown on Figure 3-4 (i.e., within catchment 21709), this manhole location will not be submerged nor allow flood propagation into the safety related SSCs.

The eight Diesel Generator Fuel Oil Storage Tanks (DGFOST) cover plates are mounted flush with the plant grade and located directly east of each Unit's Diesel Generator building as shown on Figure 3-4 (i.e., within catchments 21721 for Unit 1 and 21692 for Unit 2). Based on the peak LIP ponding elevation from this Figure for the catchments noted, it is anticipated that all eight of the cover plates would become submerged. Given the two sealed boundaries of surface caulking and cover plate gaskets and the relatively small head of water pressure, it is anticipated that any water intrusion for the short period of time exposed to the 6-hour LIP event will be minimal, consistent with the observations and conclusions made in the flooding walkdowns. In addition, the configuration of the DGFOST fill connections prevent direct water intrusion to the fuel oil. For design purposes, each fuel oil tank was assumed to contain a low amount of diesel fuel during possible flood conditions to provide the most conservative design case. The tanks are provided with hold down straps embedded in a concrete foundation. The mass of the concrete foundation

counteracts the buoyancy effects and protects against the possibility of tank floatation (Reference 2).

Conservatively considering complete manhole submergence, the level element transducer cone located within the manhole is enclosed within a standpipe extended up from the tank nozzle via a bolted flange connection mating the top of the flange with the bottom of the transducer via a rubber gasket. Although this connection is not likely to be subjected to water intrusion if the manholes were to become fully submerged, the worse case impact would be the loss of level indication due to failure at the instrument cable connections at the top of the transducer located approximately two feet above the bottom of the manhole. DGFOST level is monitored as a Technical Specification surveillance consistent with the Surveillance Frequency Control Program to ensure each tank contains sufficient fuel oil at all times to operate the diesel continuously for seven days at rated load (Reference 2). In the event normal level indication is not available, the Tech Spec Limiting Condition of Operation is entered to track the impaired condition. The 7-day period provides sufficient time to place the unit in a safe shutdown condition if required and bring in replenishment fuel from an offsite location (Reference 2). Normal fill can be achieved by either of two locations at plant grade with the primary fill being the normal sealed opening in the top of the DGFOST cover plate.

Although it is likely the LIP water level above the DGFOST cover plate will recede below the cover plate elevation and associated normal fill locations long before the need to replenish the fuel oil tanks, a third emergency fill location which feeds fuel oil directly to the Day Tanks inside the DG building is available at the DG building exterior wall located approximately 2.5 ft above the plant grade and the LIP water level. This third emergency fill option is currently documented on plant drawings and actions prescribed by existing operating procedures.

Propagation of water along the buried supply and return fuel oil piping from the West manhole locations and into the DG building is unlikely. Each pipe segment enters the building via a sealed pipe sleeve. Although the walkdowns identified some deterioration of the seals and past evidence of minor water intrusion as documented within the site's CAP, the overall integrity of the seals where determined to be sufficient to preclude extensive water intrusion and potential impact of safety related diesel generator equipment. Given the relatively short duration of the LIP event, the capacity of the west manhole to receive surface water ingress, the long distance of buried pipe between the manhole and entry into the DG building and the fact that the pipe locations are all buried under a thick tornado missile resistant concrete slab the entire distance where surface water

is unlikely to add to the saturation of the underlying soil strata, significant water ingress past the seals is considered unlikely.

Onsite Drainage

The walkdown results observed partial blockage of the site drainage system. Consistent with these observations and NRC NUREG/CR-7046 (Reference 7), the reevaluated determination of runoff due to local intense PMP at the CPNPP Units 1&2 Site, as described in ***Section 3.2.1***, initially assumed that all hydraulic structures were blocked, including the VBS openings. Thus, no allowance is given for water drained by these facilities which conservatively maximizes the estimation of runoff or peak discharges at the Site.

As described in Section 3.2.1.3, the general assumption made to develop the resultant LIP water levels around the plant site was that varying degrees of hydraulic drainage features were considered blocked. All runoff flow from inter-connecting catchments was attributed to natural drainage and limited availability of open pathways to the SCR or SSI outfalls given the current topography of the plant site. No credit was taken for the underground drainage system or the catch basins that support it. From Section 3.2.1.3 and Table 3-2, all exterior entrances which could provide a propagation pathway directly into the safety-related structures are above grade at elevation 810.5 ft or greater. Given that the maximum ponding depth adjacent to any one of these buildings is 0.34 ft, there is no direct propagation of flood water attributed to the local intense precipitation event interior to any safety related buildings.

An equipment ramp entrance on the west face of the Unit 2 non-safety related Turbine Building (TB) is measured at elevation 809.3 ft. Based on Table 3-2 ponding levels, this ramp can communicate the reevaluated effects of local intense precipitation runoff interior to lower TB elevations which are located adjacent to three doorways that provide entrance to the safety related Electrical Control Building at elevation 778 ft. The total volume of runoff that can accumulate in the lower Unit 2 TB sump and condenser pit elevations (i.e., Elevation 759 ft and lower) for the 6-hour duration of the Local Intense Precipitation event was determined to be less than the available combined capacity of these areas, (i.e., The Unit 2 TB sump and condenser pit area will have approximately 10-15% margin of available capacity remaining, considering equipment displacement and TB sump water levels under normal or refueling operation conditions prior to LIP event occurrence).

Although, the similar Unit 1 TB sump and condenser pit capacity is also available to provide additional margin via open communication pathways between Units 1 & 2 TBs, it was not utilized for purposes of this assessment. Thus, runoff propagation to the safety related Electrical Control Building at elevation 778 ft does not occur for this flood causing mechanism.

Severe Weather Abnormal Procedure Simulation

The observations that were most noteworthy as presented in Section 2.6 and in the flooding walkdown report (Reference 5) were related to the robustness of the procedures regarding time dependent actions and staffing requirements which were not specified. In the response procedures, actions are prescribed to isolate Circulating Water System (CWS) discharge valve pathways which could potentially allow backflooding to safety related areas due to the SCR PMF. Turbine Building flooding which can communicate to safety related areas of the 778 ft elevation of the Electrical & Control Building cannot occur until the SCR lake level exceeds a surface elevation of 778 ft AND the CWS discharge valve pathways located within the TB condenser pit are open for maintenance. The response procedures require these isolation actions to begin when the SCR water elevation reaches 777 ft. From the simulation it was estimated to take 3 hours for a crew of 5 to reinstall a discharge valve sufficiently to preclude backflooding, or 20 personnel for all four valves. Special valve transport carts/tools are currently available for each valve to support this minimum set of staffing requirements and the time to implement isolation actions. Based on the CLB PMF, the time it would take for the SCR surface elevation to rise from 777 ft to 778 ft was determined to be approximately 5 hours.

With respect to the rate of rise from elevation 777 ft to 778 ft in the SCR, the unsteady analysis for the combined event hazard is more limiting than the unsteady analysis for river flooding. The combined event will take 3.2 hours for the SCR water level to rise from elevation 777 ft to 778 ft. Although this is a reduction in the overall time for the SCR water levels to change for the level range discussed, the available time to implement restoration of flood protection features (in this case the replacement of the CWS discharge valves to isolate the open pathway during maintenance) is still greater than the actual estimated response time to satisfy the current licensing basis isolation actions (i.e., 3.2 hrs > 3.0 hrs) as currently prescribed in the response procedures and as observed during the procedure simulation effort during the walkdowns (Reference 5).

4.4 CONCLUSIONS

A comparison of all of the current design basis flooding elevations and each of the reevaluated flood elevations for each flood causing mechanism is provided in ***Table 3-3***. It has been determined that the current design basis flood levels do not bound the reevaluated hazard elevations for all flood causing mechanisms. It should be noted that some of the mechanisms considered and the methodologies used in the reevaluation analysis were not entirely consistent with or required to be evaluated as part of the original design basis and so direct comparisons may not be practicable in some cases (***Table 3-3***).

The effects of flood levels that are not bounded by the current licensing basis on the pertinent flood protection and mitigation features described in Section 2.6 and as identified in the CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) is provided in Section 4.3.2. The reevaluated flood levels for each flood causing mechanism do not exceed the elevations of building entrances containing safety related SSCs at the CPNPP Units 1&2 Site. All exterior entrances which could provide a propagation pathway into the safety related buildings listed in Table 3-2 are either above grade at elevation 810.5 ft or greater or do not propagate a sufficient volume of water to lower elevations of non-safety related buildings such as to enter a safety related building entrance. Where credited flood protection and mitigation features including applicable response procedures are subjected to the governing reevaluated flood causing mechanism, it was shown that the integrity of affected safety related structures is bounded by the existing design and that the requirements for affected systems, components or response procedures to satisfy their intend design basis or administrative function would not be defeated.

5.0 INTERIM EVALUATION AND ACTIONS

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

5.1 EVALUATION OF THE IMPACT OF THE REEVALUATED FLOOD LEVELS ON SSCS

Although reevaluated flooding levels exceed the current design basis flood levels due to potential flood causing mechanisms as presented in ***Section 3.0***, they do not exceed the lowest elevation of any available building entrances containing safety related SSCs at the CPNPP Units 1&2 Site. In addition, the effects of increased flood levels on applicable flood protection and mitigation features including applicable response procedures would not defeat the overall capability of safety related SSCs to satisfy their intend design basis functions.

5.2 ACTIONS TAKEN TO ADDRESS HIGHER FLOODING HAZARDS

Requested Information Item 1.d. of NRC Recommendation 2.1 specifies that the flooding reevaluation contain 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, if necessary (Reference 1).

The CPNPP Units 1&2 Post Fukushima Flooding Walkdown Report (Reference 5) concluded that "There are no newly installed flood protection or mitigation measures. Based on the results of the walkdowns, there are no observations which will require the implementation of other planned flood protection measures as currently incorporated into the CPNPP current licensing basis and supporting programs and processes."

Furthermore, from the technical evaluation provided in Section 4.3.2 of the potential effects of the reevaluated flood levels to the pertinent SSCs including flood protection and mitigation features previously addressed in Reference 5 and as summarized in Sections 4.4 and 5.1, no interim actions are required to be taken or planned to address higher flooding hazards relative to the CPNPP Units 1 & 2 current design basis.

The flooding reevaluations were performed in response to the NRC's March 12, 2012 50.54(f) letter (Reference 1) and are beyond design basis analyses. As such, they do not constitute an immediate operability concern and are not reportable outside of the response to the 50.54(f) letter. As a result of this reevaluation, there were no concerns or observations with the current licensing or design basis. However, since the CPNPP Units 1 & 2 current licensing basis flood levels are not bounded by this reevaluation effort, the applicable condition(s) have been entered into the CPNPP corrective action program.

6.0 ADDITIONAL ACTIONS

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

At this time, there are no additional actions beyond Requested Information Item 1.d. of NRC Recommendation 2.1 (Reference 1) (***Section 5.0***) which have been taken or are planned to address flooding hazards at CPNPP Units 1&2.

Per the NRC guidance for performing the integrated assessment for external flooding, addressees are requested to perform an integrated assessment if the current design basis flood hazard elevation does not bound the reevaluated flood hazard elevation for all flood causing mechanisms (Reference 25). Furthermore, "The integrated assessment will evaluate the total plant response to the flood hazard, considering multiple and diverse capabilities such as physical barriers, temporary protective measures, and operations procedures (Reference 25)." Since the current design basis for external flooding does not bound the reevaluated hazards, an integrated assessment will be performed.

7.0 REFERENCES

1. Nuclear Regulatory Commission (NRC), 2012, "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," Washington DC, March 12, 2012.

2. Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August 2011.

3. Luminant Generating Company, LLC (Luminant), 2012, "Final Safety Analysis Report (FSAR) Comanche Peak Nuclear Power Plant (CPNPP) Units 3&4," Revision 3, June 2012.

4. Nuclear Regulatory Commission (NRC), 1976, "Ultimate Heat Sink for Nuclear Power Plants," Regulatory Guide 1.27, Revision 2, NRC, Washington DC, January 1976.

5. Luminant Generating Company, LLC (Luminant), 2012, "Comanche Peak Nuclear Power Plant (CPNPP) Units 1&2 Post Fukushima Flooding Walkdown Report," Enclosure to TXX-12177, November 2012.

6. Texas Water Development Board (TWDB), 2008, "Volumetric and Sedimentation Survey of Squaw Creek Reservoir, December 2007 Survey," prepared for Luminant, Austin, Texas, August 2008.

7. Nuclear Regulatory Commission (NRC), 2011, "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 DC, November 2011.

8. National Weather Service (NWS), 1982, "Hydrometeorological Report No. 52 (HMR No. 52): Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian," August 1982.

9. United States Army Corps of Engineers (USACE), 2010, Hydrologic Engineering Center (HEC), HEC-HMS 3.5 Computer Program Build 1417, Release date: August 2010.

10. Brater et al., 1996, "Handbook of Hydraulics for the Solution of Hydraulic Engineering Problems," E.F. Brater, H. W. King, J. E. Lindell, C. Y. Wei, McGraw-Hill.

11. Natural Resources Conservation Service (NRCS), 1986, "Urban Hydrology for Small Watersheds-Technical Release 55 (TR-55) 2nd Edition," June 1986, 164 pp.

12. Natural Resources Conservation Services (NRCS), 2007, "National Engineering Handbook," Chapter 16, March 2007.

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14. National Weather Service (NWS), 1978, "Hydrometeorological Report No. 51 (HMR No. 51): Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," June 1978.

15. United States Army Corps of Engineers (USACE), 2000, "Hydrologic Engineering Center, Hydrologic Modeling System (HEC-HMS), Technical Reference Manual," March 2000.

16. United States Army Corps of Engineers (USACE), 2010, Hydrologic Engineering Center (HEC), HEC-RAS 4.1.0 Computer Program, release date: January 2010.

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18. United States Army Corps of Engineers (USACE), 2010, "Hydrologic Engineering Center – River Analysis System (HEC-RAS) Version 4.1, Hydraulic Reference Manual," January 2010.

19. American National Standards Institute/American Nuclear Society (ANSI/ANS), 1992, "Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS-2.8-1992, La Grange Park, IL.

20. Nuclear Regulatory Commission (NRC), 2008, "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America," NUREG/CR-6966, PNNL-17397, NRC Job Code J3301, Washington, DC, August 2008.

21. Environmental Protection Agency (EPA), 2012, EPA U.S., STORET, Website <http://www.epa.gov/storet/>, date accessed: October 2012.

22. United States Army Corps of Engineers (USACE), 1994, "Ice Jam Flooding: Causes and Possible Solutions," EP 1110-2-11, November 30, 1994.

23. National Climatic Data Center (NCDC), 2012, NCDC Website: <http://www.ncdc.noaa.gov/cdo-web/search>, date accessed: August 1, 2012.

24. United States Army Corps of Engineers (USACE), 2004, "Ice Engineering, Method to Estimate River Ice Thickness Based on Meteorological Data," TN-04-3, June 2004.

25. Nuclear Regulatory Commission (NRC), 2012, "Guidance for Performing the Integrated Assessment for External Flooding," Interim Staff Guidance, Revision 0, JLD-ISG-2012-05, November 20, 2012.

26. Paul C. Rizzo Associates, Inc., "Comanche Peak Nuclear Power Plant , Units 1 & 2 Flooding Hazard Reevaluation Report," Revision 0, Project No. 12-4891, February 12, 2013.

TABLES

TABLE 2-1
HISTORIC FLOODS NEAR THE SITE

RIVER/STREAM	DATE	GAUGING STATION	MAX WATER LEVEL (ft)	DISCHARGE (cfs)
Brazos River	1876	NA*	NA*	NA*
Brazos River	April 28, 1990	08091000	603.58	79,800
Paluxy River	April 17, 1908	08091500	636.86	59,000
Squaw Creek	April 8, 1975	08091750	610.90	9,030

Note:

* The 1876 flood occurred before any flow monitoring began, but is considered one of the greatest known floods on the Brazos River based on historical observations.

References:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August 2011.

Luminant Generating Company, LLC, 2012, "Final Safety Analysis Report (FSAR) Comanche Peak Nuclear Power Plant (CPNPP) Units 3&4 (FSAR)," Revision 3, June 2012.

TABLE 2-2
LIST OF SAFETY-RELATED STRUCTURES AND THEIR ELEVATIONS

SSC	ELEVATION (ft)*
Auxiliary Building	810.0
Reactor Building (Each Unit)	810.0
Electrical and Control Building	810.0**
Safeguards Building (Each Unit)	810.0
Diesel Building (Each Unit)	810.0
Fuel Building	810.0
Service Water Intake Structure	796.0†
Condensate Storage Tank (Each Unit)	810.0
Reactor Makeup Water Storage Tank (Each Unit)	810.0
Refueling Water Storage Tank (Each Unit)	810.0

Notes:

* All exterior entrances which could provide a propagation pathway into the buildings listed are above grade at elevation 810.5 ft or greater.

** Doors to the Electrical and Control Building are located above the PMF elevation with the exception of doors to the Turbine Building at elevation 778.0 ft. As such, flooding protection measures in accordance with existing programs and procedures are currently in place for the ECB (CPNPP, 2011). All exterior accesses to these buildings are above elevation 810.0 ft, with the exception of Turbine Building entrances to the Electrical and Control Building.

† The operating deck of the Service Water Intake Structure (SWIS) is at elevation 796.0 ft with the pump discharge centerline at elevation 800.0 ft. Access to the SWIS is via a personnel door and truck bay entrance at the grade elevation of 810.5 ft (CPNPP, 2011).

Reference:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August 2011.

TABLE 2-3
DESIGN PARAMETERS IN THE LICENSE DOCUMENT

DESIGN PARAMETER	VALUE
Plant Grade Level (all SSCs except Service Water Intake Structure)	810 ft
Elevation of Service Water Intake Structure	796 ft*
Lowest Elevation of Exterior Entrances to any Safety Related Structure	810.5 ft
Probable Maximum Precipitation	39.1 in (48 hour PMP)
Category 1 Roof Uniform Maximum Design Precipitation Depth	8 in
Onsite Drainage System Design Rainfall Rate	6 in (in one hour) / 7.5 in (in two hours) **
Maximum SCR Water Level at the Site Including Wave Run-up	794.7 ft
Maximum Water Level at the Squaw Creek Dam Including Wave Run-up	793.7 ft
Maximum Water Level at the Safe Shutdown Impoundment Dam Including Wave Run-up (SCR Side of SSI Dam)	791.3 ft
Annual Mean Temperature	66 °F

Notes:

* The operating deck of the Service Water Intake Structure (SWIS) is at elevation 796.0 ft with the pump discharge centerline at elevation 800.0 ft. Access to the SWIS is via a personnel door and truck bay entrance at elevation 810.5 ft (CPNPP, 2011). The SWIS is designed for hydrostatic loads with a design basis groundwater level at elevation 793 ft (CPNPP, 2011).

** The onsite drainage system is designed to remove the water resulting from a rainfall of 6 in. in one hour and 7.5 in. in two hours, in such a manner that runoff does not form ponds on the ground. Further, the drainage system is designed to adequately drain a rainfall of 15 in. in one hour and 22 in. in two hours in such a way that there are no ponds which can back up into the safety-related SSCs (CPNPP, 2011).

Reference:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August 2011.

TABLE 2-4
CLB WATER LEVELS DUE TO ALL FLOODING MECHANISMS

FLOODING MECHANISM	WATER LEVEL (ft)
Local Intense Precipitation	N/A (Maximum is 39.1 in. given as a 48-hour PMP for 64 mi^2 area)
River Flooding	789.7 / 790.5*
River Flooding Including Wave Run-up	794.7 / 793.7 / 791.3**
Dam Failure Flooding on the Brazos River	700.0†
Storm Surge and Seiche Flooding	N/A
Tsunami Flooding	N/A
Ice Flooding	N/A
Channel Diversion Flooding	N/A
Combined Events	N/A

Notes:

* The maximum water level of 789.7 ft is within the SCR, while the maximum water level of 790.5 ft is within the SSI.

** Water level of 794.7 ft is due to the maximum wind wave run-up in the SCR at the Site. The maximum water levels, including wave run-up at the Squaw Creek Dam and SSI Dam, are 793.7 and 791.3 ft, respectively.

† Water level of 700.00 ft is reported for the confluence of the Brazos River with the Paluxy River. The FSAR for CPNPP Units 1&2 states in part that the CPNPP site can in no way be endangered by any dam breaks or series of dam breaks, since it is over 110 ft above postulated water levels.

Reference:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August 2011.

TABLE 3-1
SITE DRAINAGE AREA DETAILS

DRAINAGE SUB BASIN	AREA (mi^2)	RAINFALL INTENSITY I (inch/hr)*	RUNOFF COEFFICIENT C	PEAK RUNOFF Q (cfs) RATIONAL METHOD	PEAK RUNOFF Q (cfs) HEC-HMS MODEL
21652	0.004	69.57	1.00	177	139
21656	0.003	69.57	1.00	154	122
21658	0.004	69.57	1.00	187	130
21660	0.009	69.57	1.00	387	305
21662	0.010	69.57	1.00	445	335
21664	0.0005	69.57	1.00	21	16
21671	0.012	69.57	1.00	556	429
21673	0.001	69.57	1.00	48	38
21677	0.032	69.57	1.00	1,432	1,116
21680	0.003	69.57	1.00	113	89
21682	0.003	69.57	1.00	152	120
21683	0.003	69.57	1.00	113	89
21685	0.001	69.57	1.00	45	35
21686	0.003	69.57	1.00	143	112
21690	0.001	69.57	1.00	41	32
21691	0.001	69.57	1.00	37	29
21692	0.004	69.57	1.00	193	152
21695	0.001	69.57	1.00	34	27
21696	0.002	69.57	1.00	68	53
21699	0.005	69.57	1.00	208	164
21703	0.003	69.57	1.00	120	94
21706	0.001	69.57	1.00	45	35
21709	0.001	69.57	1.00	50	39
21712	0.001	69.57	1.00	61	48
21714	0.001	69.57	1.00	28	22
21717	0.0005	69.57	1.00	22	17
21719	0.0005	69.57	1.00	22	17
21720	0.004	69.57	1.00	161	127
21721	0.002	69.57	1.00	97	77
21724	0.0005	69.57	1.00	22	17
21728	0.006	69.57	1.00	282	222

Note:

* Rainfall intensity was calculated using the reevaluated 1-hour 1-mi^2 PMP and the total travel time.

TABLE 3-2
WATER LEVELS AND PONDING DEPTHS DUE TO LOCAL INTENSE PRECIPITATION

SSC	SAFETY-RELATED?	PEAK PONDING WATER ELEVATION (ft)*	PEAK PONDING DEPTH (ft) **
Condensate Storage Tank 1	Yes	809.53	0.00
Condensate Storage Tank 2	Yes	810.33	0.33
Reactor Water Makeup Storage Tank 1	Yes	809.53	0.00
Reactor Water Makeup Storage Tank 2	Yes	810.33	0.33
Auxiliary Building	Yes	N/A	0.00
Containment Unit 1	Yes	809.86	0.00
Containment Unit 2	Yes	810.12	0.12
Diesel Building 1	Yes	809.53	0.00
Diesel Building 2	Yes	810.33	0.33
Electrical Control Building	Yes	N/A	0.00
Fuel Building	Yes	810.12	0.12
Refueling Water Storage Tank 1	Yes	809.53	0.00
Refueling Water Storage Tank 2	Yes	810.33	0.33
Safeguard Building 1	Yes	809.53	0.00
Safeguard Building 2	Yes	810.33	0.33
SWIS	Yes	795.86(1)	0.00
Switchgear Building 1	No	809.18	0.00
Switchgear Building 2	No	810.33	0.33
Turbine Building 1	No	810.34	0.34
Turbine Building 2	No	810.34	1.04(2)

Note:

* Values are taken from Figure 3-4. Interior located building structures such as the Auxiliary Bldg and the Electrical & Control Bldg. are not subjected to ponding.

** Based on an elevation of 810 ft immediately adjacent to safety related plant structures.

(1) SWIS structure and entrance at 810 ft and slopes down around sides of structure to SSI water level. 795.86 ft represents lowest elevation of catchment area 21656 near the SWIS such that no ponding can occur.

(2) Based on a Turbine Building (TB) entrance elevation of 809.3 ft. An equipment ramp entrance on the west face of the Unit 2 non-safety related TB is at 809.3 ft and has the potential to communicate local intense precipitation runoff interior to a lower TB elevation which is located adjacent to the safety related Electrical Control Building at elevation 778 ft.

TABLE 3-3
COMPARISON OF CURRENT AND REEVALUATED FLOOD LEVELS

FLOODING MECHANISM	REEVALUATED WATER LEVEL (ft)	CURRENT LICENSING BASIS WATER LEVEL (ft)
Local Intense Precipitation	810.34	N/A (39.1 in. given as 48-hour PMP for 64 mi^2 area)**
River Flooding	791.80* (Scenario 4, Existing Conditions)	789.7 (Within SCR) 790.5 (Within SSI)
	796.96 (Scenario 4 + Wave Run-up on CWIS side)	794.7 (Within SCR Including Wave Run-up)
	795.48 (Scenario 4 + Wave Run-up on SSI Dam from SCR side)	791.3 (Within SCR at the SSI Dam Including Wave Run-up)
	794.68 (Scenario 4 + Wave Run-up on SSI Dam from SSI side)	790.5 (Within the SSI; Wave Run-up Negligible)
	794.00 (Scenario 4 + Wave Run-up on SWIS embankment)	
	794.60 (Scenario 4 + Wave Run-up on SWIS vertical face)	
Dam Failure Flooding	Reevaluated Water Levels due to Dam Failure Flooding Including Run-up on SCR and SSI are bounded by the above River Flooding Water Levels	N/A (700.00 is reported downstream of the Squaw Creek Dam, which is not analogous to reevaluated water levels at the Site)
Storm Surge and Seiche Flooding	N/A	N/A
Tsunami Flooding	N/A	N/A
Ice Flooding	N/A	N/A
Channel Diversion Flooding	N/A	N/A

TABLE 3-3
COMPARISON OF CURRENT AND REEVALUATED FLOOD LEVELS (CONTINUED)

FLOODING MECHANISM	REEVALUATED WATER LEVEL (ft)	CURRENT LICENSING BASIS WATER LEVEL (ft)
Combined Events Flooding	792.23*** (Scenario 4, Existing Conditions)	Combined Events not Required to be Evaluated
	797.39 (Scenario 4 + Wave Run-up on CWIS side)	
	795.91 (Scenario 4 + Wave Run-up on SSI Dam from SCR side)	
	795.11 (Scenario 4 + Wave Run-up on SSI Dam from SSI side)	
	794.43 (Scenario 4 + Wave Run-up on SWIS embankment)	
	795.03 (Scenario 4 + Wave Run-up on SWIS vertical face)	

Note:

* The still water level reported for the river flooding hazard is based on a steady state HEC-RAS model. The dam failure flooding hazard also included the evaluation of the PMP on the SCR using an unsteady (transient model). This resulted in a simulated water level of 789.88 ft in the SCR and 791.39 ft in the SSI for the most conservative case of the dam failure hazard and considering no obstruction to the equalization channel.

** CLB PMP event considers a plant grade elevation of 810 ft immediately adjacent to safety related plant structures having exterior entrances @ 810.5' and no ponding > 810.5'.

*** The still water level reported for the combined event is based on a steady state HEC-RAS model. The effect of the Squaw Creek watershed combined event precipitation levels on water levels within the SSI was also evaluated using a unsteady (transient) HEC-RAS model to study the impact on debris blockage of the equalization channel between the two bodies of water. The analysis resulted in a simulated water level for the SSI of 792.17 ft with the equalization channel unblocked. This coincided with a peak water level in the SCR of 792.16 ft. Water levels due to wave run-up coincident with the simulated water level in the SSI of 792.17 ft will be less than those water levels reported for Scenario 4 of the Combined Events hazard.

Reference:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August 2011.

TABLE 3-4
COMPARISON OF CURRENT DESIGN BASIS AND FLOODING REEVALUATION ASSUMPTIONS

ASSUMPTIONS	REEVALUATED HAZARDS	CURRENT DESIGN BASIS
Station used for Wind Analysis	Fort Worth Naval Air Station	Fort Worth Naval Air Station
PMP Calculation	Calculation based on HMR 52 results	Calculation based on HMR 33 results
PMS Rainfall Hyetograph	Time period of 72 hours with 5 minute intervals	48 hour with 3 hour increments
HEC-HMS Model Calibration	Revised Mannings n roughness, basin and peak coefficients based on calibration	HEC-HMS was not used for the FSAR analysis. The model used for the FSAR was not calibrated.
Spillway Rating Curves	Service and Emergency Spillways	Service and Emergency Spillways
Transformation Method	Snyder's Unit Hydrograph Method	Snyder's Unit Hydrograph Method
PMF loss, transformation, and routing	Include loss, transformation, and routing through the channels and the emergency spillway.	Include loss and transformation, with discharge assumed to occur over uncontrolled spillways.
Difference in water elevations due to coincident wind wave activity	Longer fetch lengths used to determine wind setup.	Lower fetch lengths used to determine wind setup.
Water Surface Elevations due to Dam Break	Provides water surface elevation at the Squaw Creek Dam.	Provides water surface elevation downstream of the Squaw Creek Dam.

Reference:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August 2011.

TABLE 3-5
COMPARISON OF CURRENT DESIGN BASIS AND FLOODING REEVALUATION ANALYTICAL INPUTS

ANALYTICAL INPUTS	REEVALUATED HAZARDS		CURRENT DESIGN BASIS	
1-hour 1 mi^2 PMP Values	19.1 in		Values are based on HMR No. 33 methods and are not analogous to reevaluated results. (Drainage system is designed to remove water resulting from a rainfall of 6 in. in 1 hour)	
Predicted Maximum Sustained Wind Speeds	Return Period (years)	Wind Speed for 10 minutes (mph)	Return Period (years)	Wind Speed (mph)**
	2	37.84	2	43.70
	10	47.83	10	51.42
	50	55.57	50	59.02
	100	58.57	100	62.79
Calculated PMF Values*	Discharge from the SCR is 169,464 cfs Inflow to SCR is 257,306 cfs		PMF Discharge is 131,150 cfs Peak Inflow is 149,000 cfs	
Spillway Elevations for the SCR (Service / Emergency)	775 ft / 783 ft		775 ft / 783 ft	
Over Land Wind Speed	37.84 mph		40 mph	

Notes:

* Discharge and inflow values in CPNPP, 2011 are computed by using bounding isohyet PMF values and do not include any routing. Hence, the values reported in the FSAR for Units 1&2 (CPNPP, 2011) are not comparable to the calculated PMF values.

** The values reported in CPNPP, 2011 are converted to obtain the 10-minute (32.8 ft elevation) sustained wind speeds.

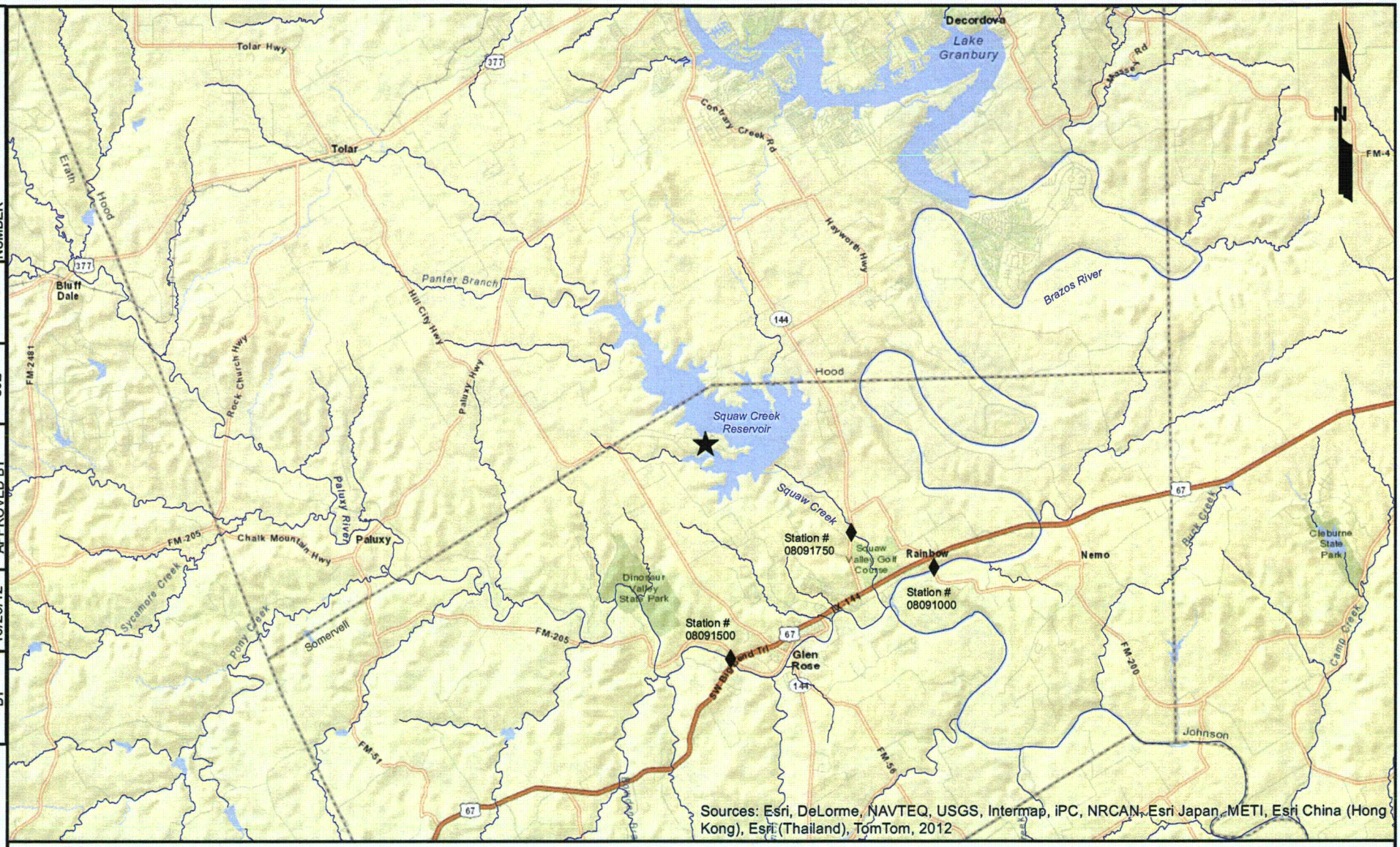
Reference:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August 2011.

FIGURES

DRAWN BY	KMR	CHECKED BY	JML	11 Feb 2013	GIS FILE
	10/29/12	APPROVED BY	CJE	11 Feb 2013	NUMBER

12-4891-GIS-A001



Legend

- ★ Site Centerpoint
- River/Stream
- Roads
- County Boundaries
- ♦ USGS Gauge Stations

0 1 2 4 6 8 10 Miles

Coordinate System: NAD 1983 UTM Zone 14N
Projection: Transverse Mercator
RF: 1:182,000

References:

1) Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand),TomTom, 2012 at: http://goto.arcgisonline.com/maps/World_Street_Map

2) Luminant Generating Company, LLC (Luminant), 2012, "Final Safety Analysis Report (FSAR) Comanche Peak Nuclear Power Plant (CPNPP) Units 3&4 (FSAR)," Revision 3, June, 2012.

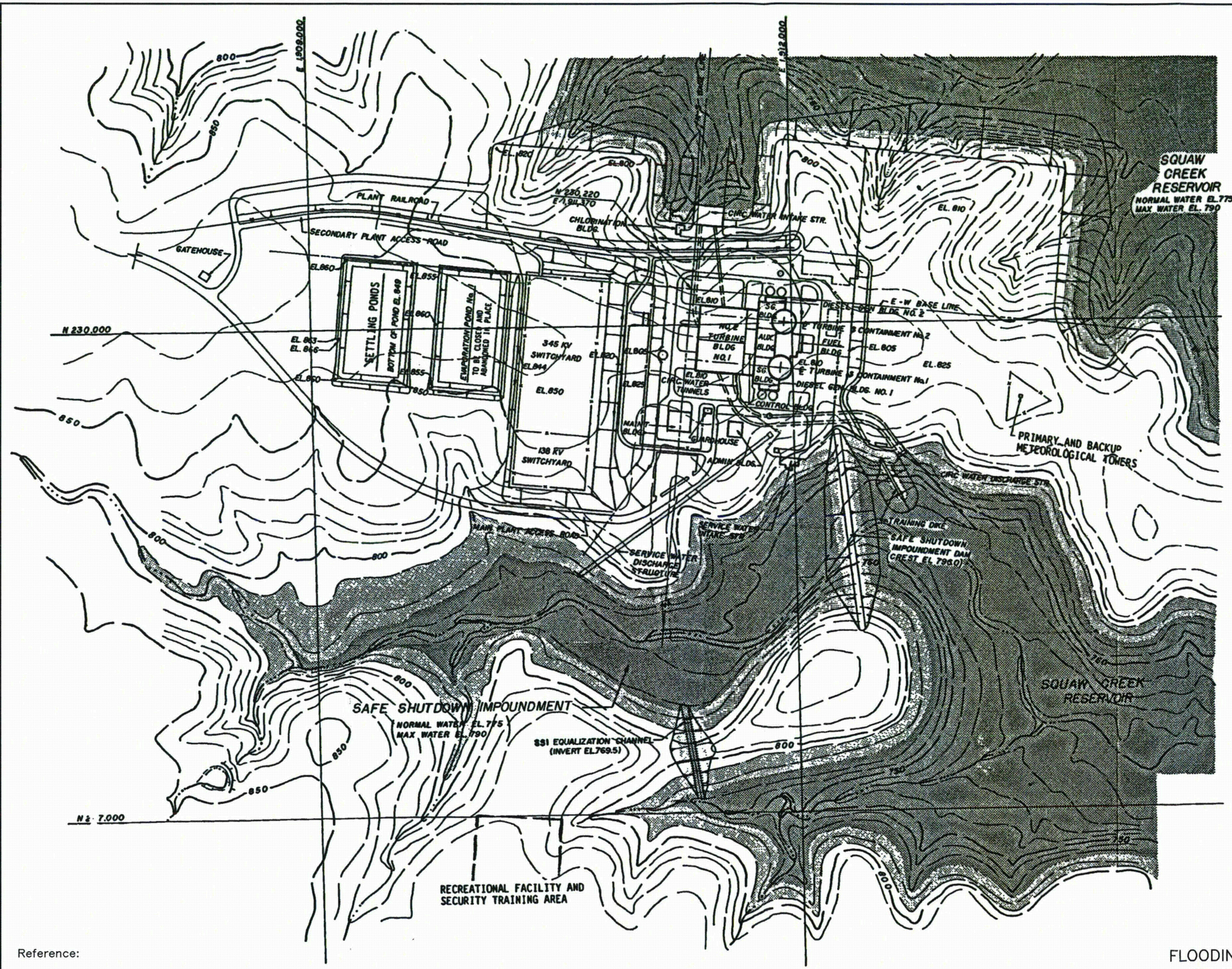
Figure 1-1
Location Map of the Site

PREPARED FOR:

Comanche Peak Flooding Hazard Reevaluation Report

Paul C. Rizzo Associates, Inc.
ENGINEERS/CONSULTANTS/CM

DRAWN BY	CHECKED BY	CAD FILE NUMBER
CVL 10/31/12	APPROVED BY	12-4891-B3
JML 02/11/13	CJE 02/11/13	



(NOT TO SCALE)

FIGURE 2–1

DESIGN SITE LAYOUT & TOPOGRAPHY

PREPARED FOR

COMANCHE PEAK FLOODING HAZARD REEVALUATION REPORT

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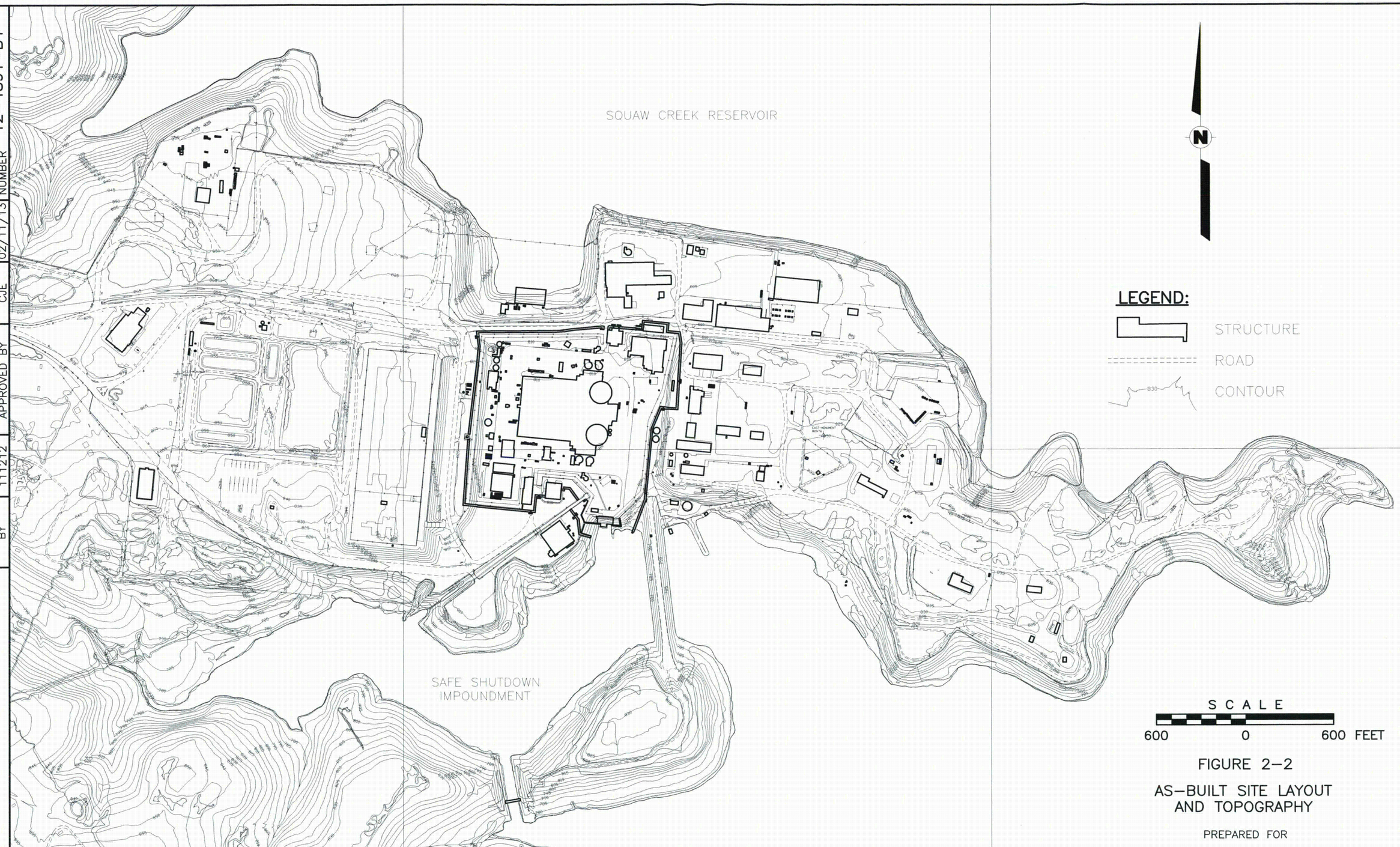
Reference:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August, 2011.



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J.S.S.	JML	02/11/13	12-4891-B4
111212	CJE	02/11/13	

FIGURE 2-2

AS-BUILT SITE LAYOUT AND TOPOGRAPHY

PREPARED FOR

COMANCHE PEAK FLOODING HAZARD REEVALUATION REPORT

Paul C. Rizzo Associates, Inc.
ENGINEERS / CONSULTANTS / CM

Reference:

Comanche Peak Nuclear Power Plant (CPNPP), 2011, "Final Safety Analysis Report (FSAR Units 1&2)," Amendment 104, Glen Rose, Texas, August, 2011.

DRAWN BY	BFR 10/25/12	CHECKED BY APPROVED BY	JML CJE	02/11/13 02/11/13	CAD FILE NUMBER	12-4891-A1

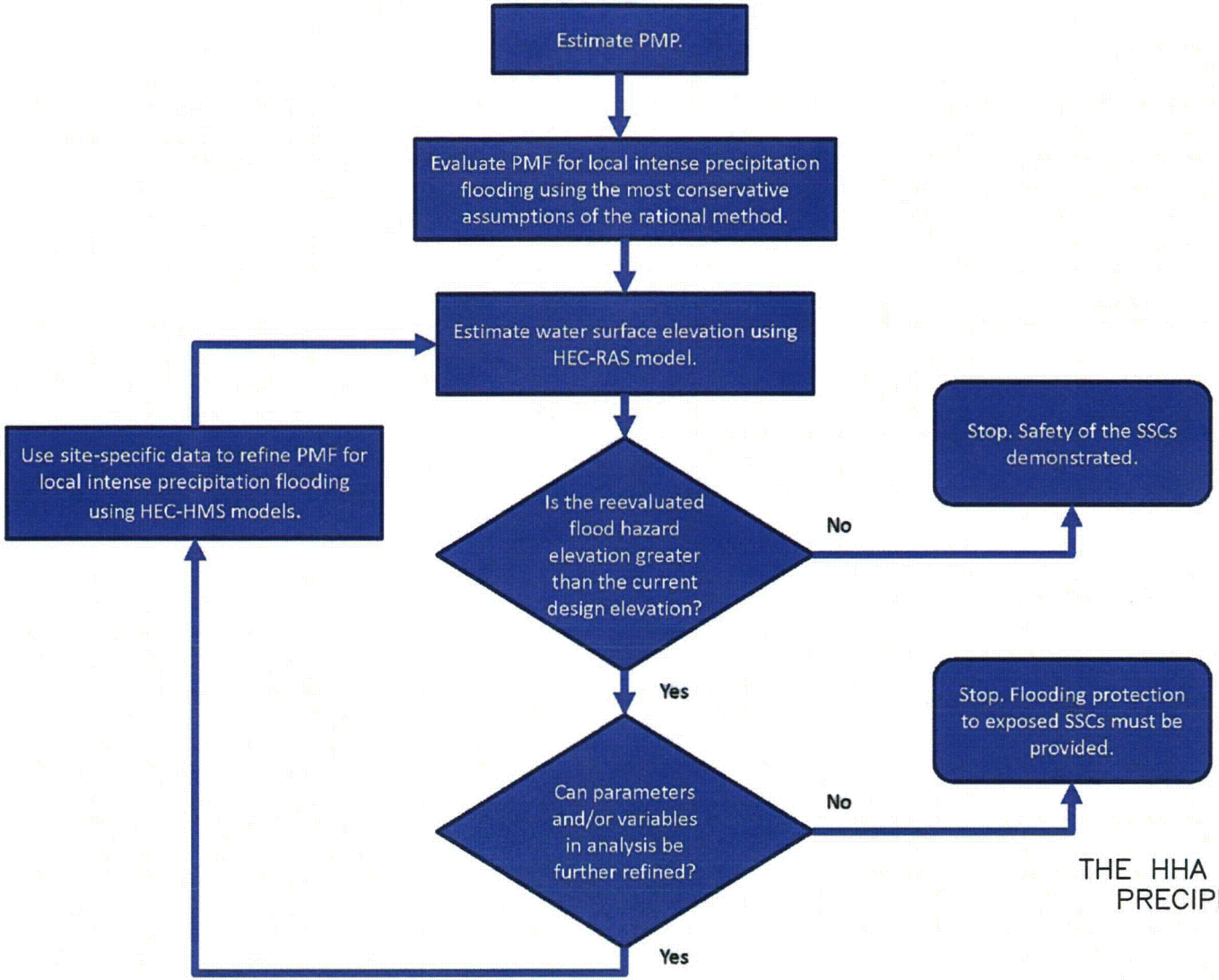

FIGURE 3-1

THE HHA DIAGRAM FOR LOCAL INTENSE PRECIPITATION FLOODING ANALYSIS

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

Paul C. Rizzo Associates, Inc.
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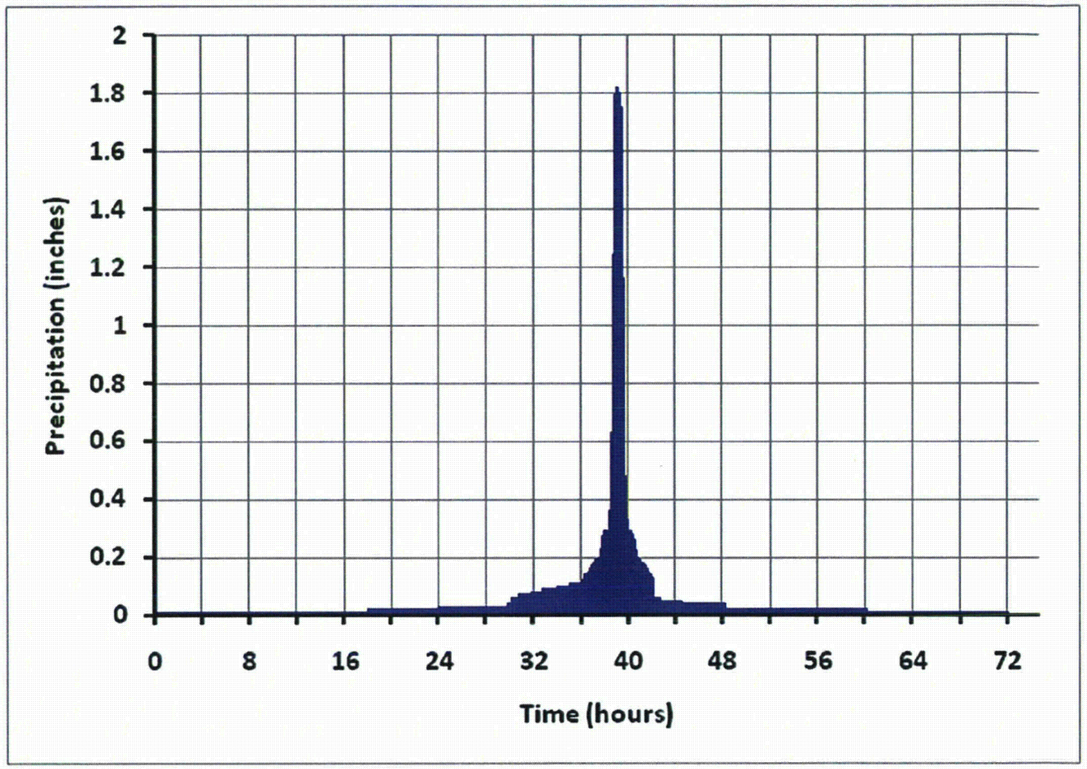
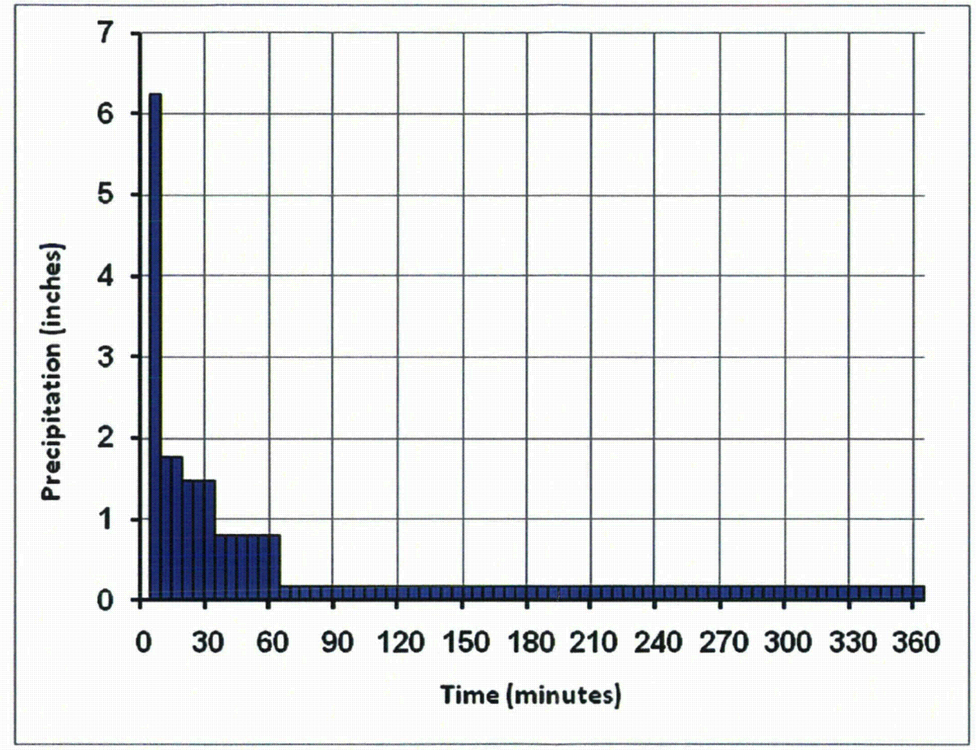
12-4891-B8



PROBABLE MAXIMUM STORM FOR LOCAL INTENSE PRECIPITATION USING HMR 52 PROGRAM



6-HOUR LOCAL INTENSE PRECIPITATION HYETOGRAPH BASED ON HMR 52
FIGURE 24

FIGURE 3-2

LOCAL INTENSE
PRECIPITATION HYETOGRAPHS

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

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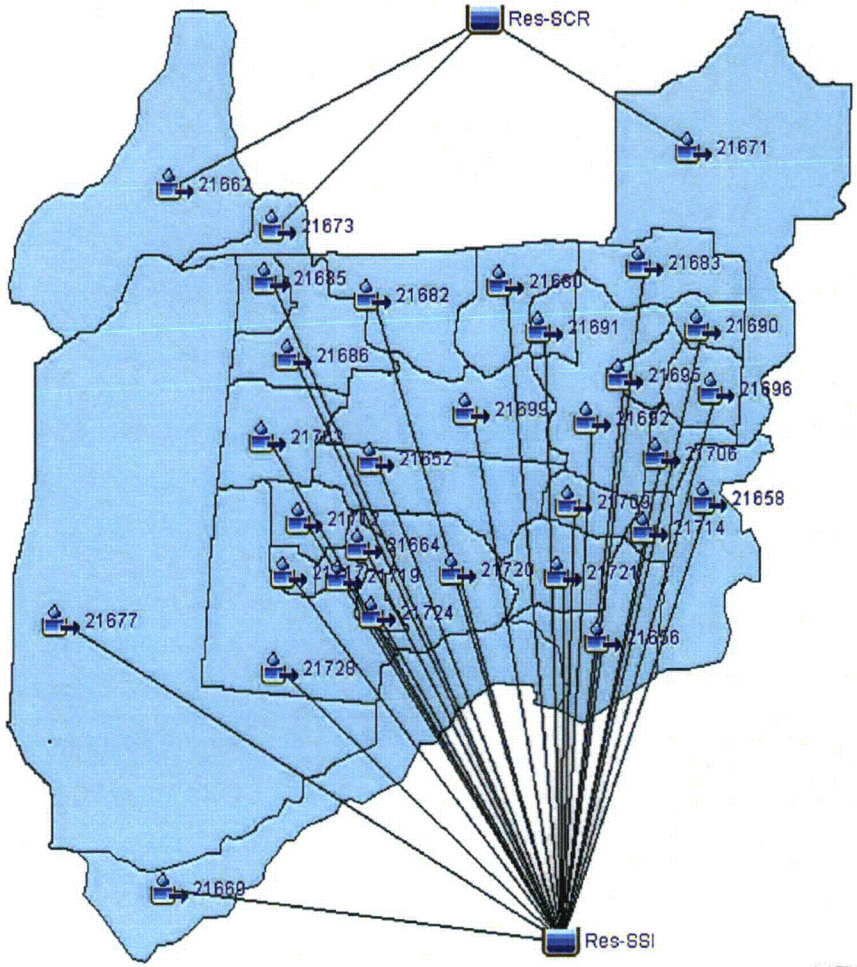

LEGEND:

- SUBBASIN
- 21671 SUBBASIN NUMBER
- RESERVOIR
- HYDRAULIC CONNECTION
- SUBBASIN DELINEATION

(NOT TO SCALE)

FIGURE 3-3

HEC-HMS MODEL FOR THE SITE DRAINAGE ANALYSIS

PREPARED FOR

COMANCHE PEAK FLOODING HAZARD REEVALUATION REPORT

Paul C. Rizzo Associates, Inc.
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Building Number	Building Description
1	Condensate Storage Tank
2	Refueling Water Storage Tank
3	Reactor Water Makeup Storage Tank
4	Service Water Intake Structure
5	Containment Unit 1
6	Containment Unit 2
7	Fuel Bldg
8	Diesel Building 1
9	Diesel Building 2
10	Safeguard Bldg 2
11	Auxillary Bldg
12	Safeguard Bldg 1
13	Switchgear Bldg 1
14	Electrical Control Bldg
15	Switchgear Bldg 2
16	Safe Shutdown Impoundment Dam

Legend

- Model Catchments
- Non-Safety Related Structures
- Safety Related Structures
- Inundated Area

810.34 ft. Peak Flood Elevation (msl) for Catchments

0 250 500 1,000 1,500 2,000 2,500 Feet

Coordinate System: NAD 1983 UTM Zone 14N
Projection: Transverse Mercator
RF: 1:5,000

Reference(s): Source: Esri, i-cubed, USDA, USGS, AEX, GeoEye, Getmapping, Aerogrid, IGN, IGP, and the GIS User Community at http://goto.arcgisonline.com/maps/World_Imagery

Figure 3-4

Inundation Map of the Reevaluated Flood Level Due to Local Intense Precipitation

Prepared For

Comanche Peak Flooding Hazard Reevaluation Report

Paul C. Rizzo Associates, Inc.
ENGINEERS/CONSULTANTS/CM

GJO	JML	28 Feb 2013	GIS FILE NUMBER
05 DEC 2012	CJE	28 Feb 2013	12-4891-GIS-B006
DRAWN BY	CHECKED BY	APPROVED BY	

DRAWN BY	CVL 10/30/12	CHECKED BY APPROVED BY	JML CJE	02/11/13 02/11/13	CAD FILE NUMBER	12-4891-A5

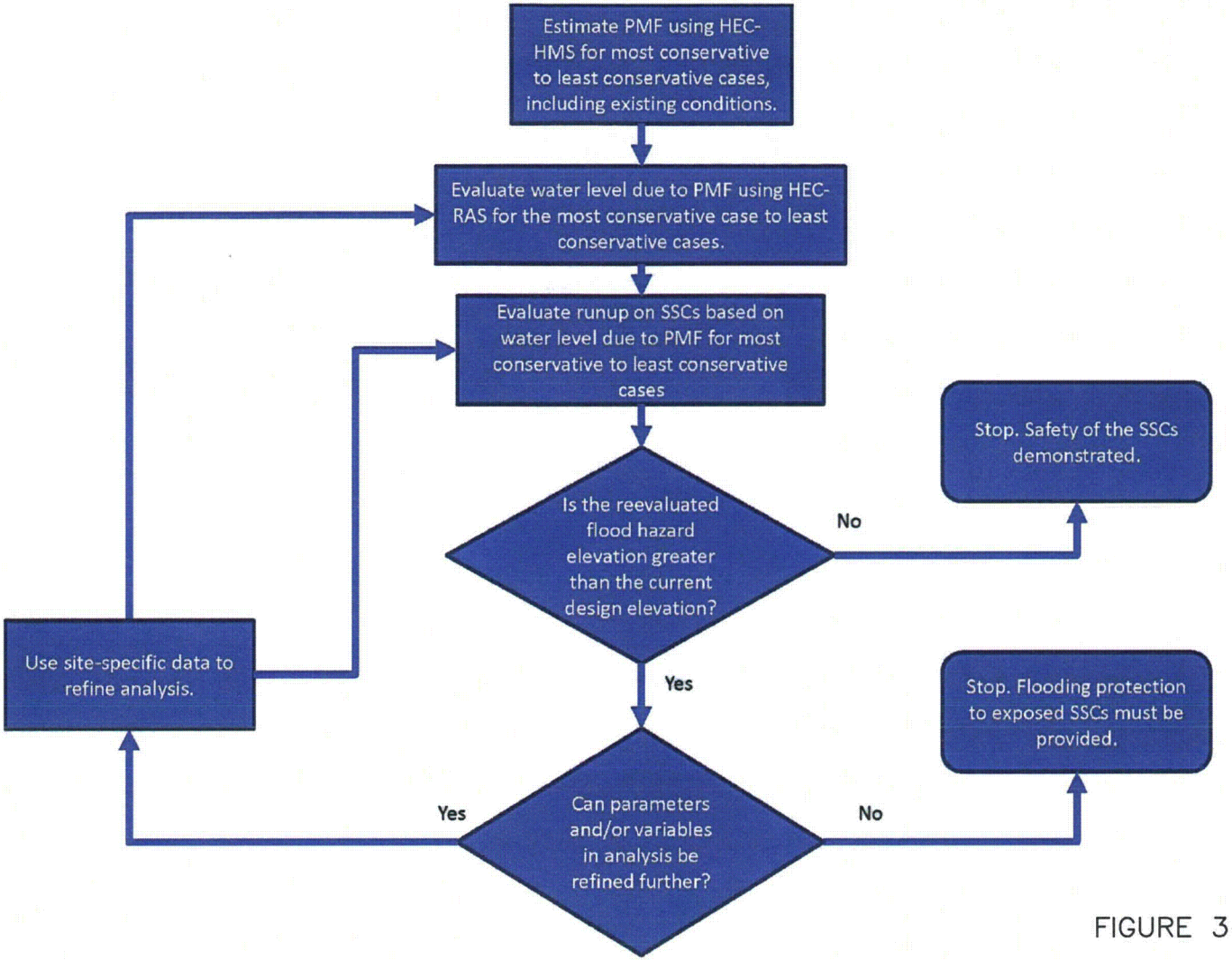

FIGURE 3-5

THE HHA DIAGRAM FOR RIVER FLOODING ANALYSIS

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

Paul C. Rizzo Associates, Inc.
ENGINEERS / CONSULTANTS / CM

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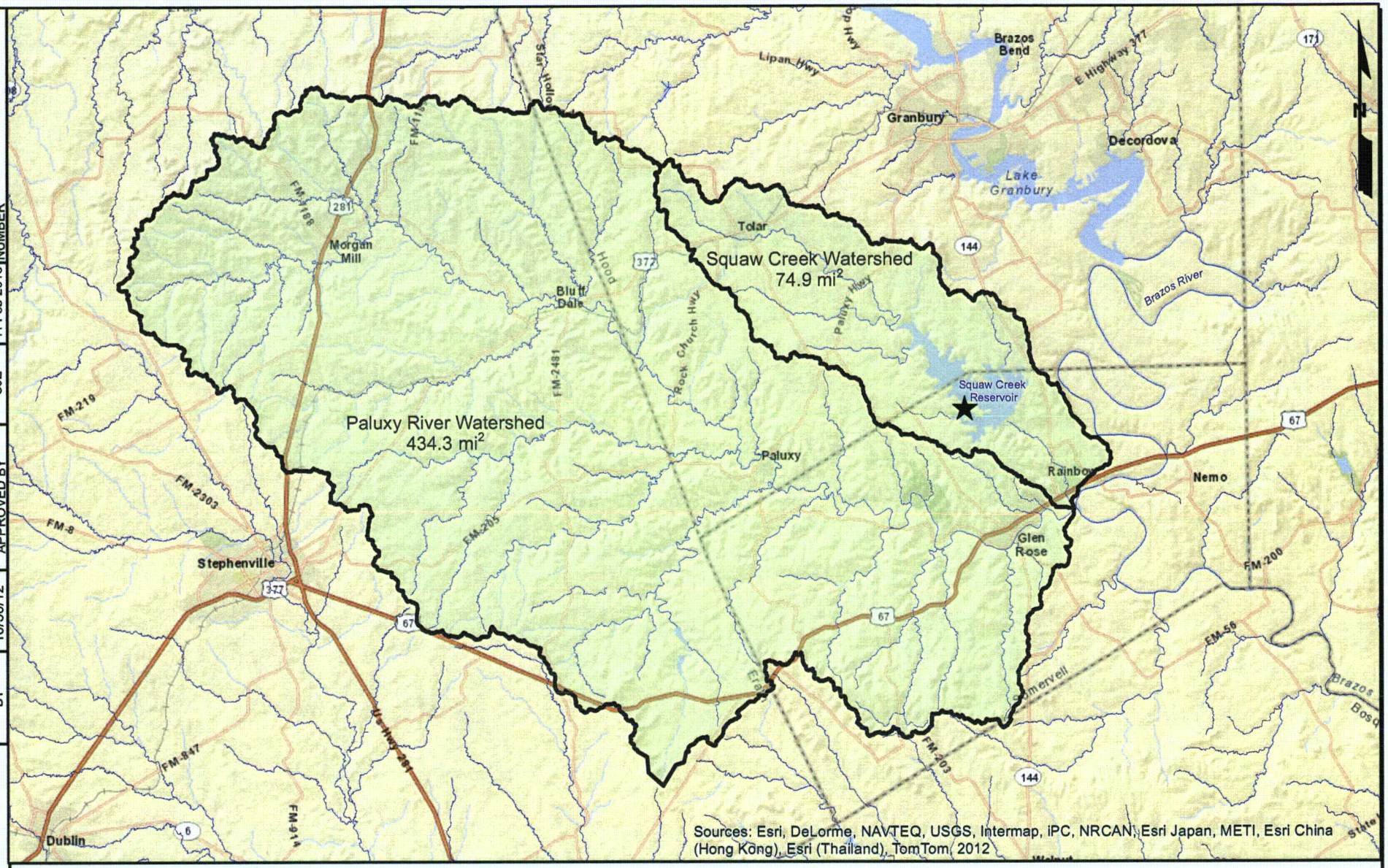
	10/30/12	APPROVED BY	CJE	11 Feb 2013	NUMBER



Legend

- ★ Site Centerpoint
- Watersheds
- Roads
- Streams
- County Boundary

0 2.5 5 10 15 20 Miles

Coordinate System: NAD 1983 UTM Zone 14N
Projection: Transverse Mercator
RF: 1:300,000

References: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand),TomTom, 2012 at: http://goto.arcgisonline.com/maps/World_Street_Map

Figure 3-6

Watersheds Map

PREPARED FOR:

Comanche Peak Flooding Hazard Reevaluation Report

Paul C. Rizzo Associates, Inc.
ENGINEERS/CONSULTANTS/CM

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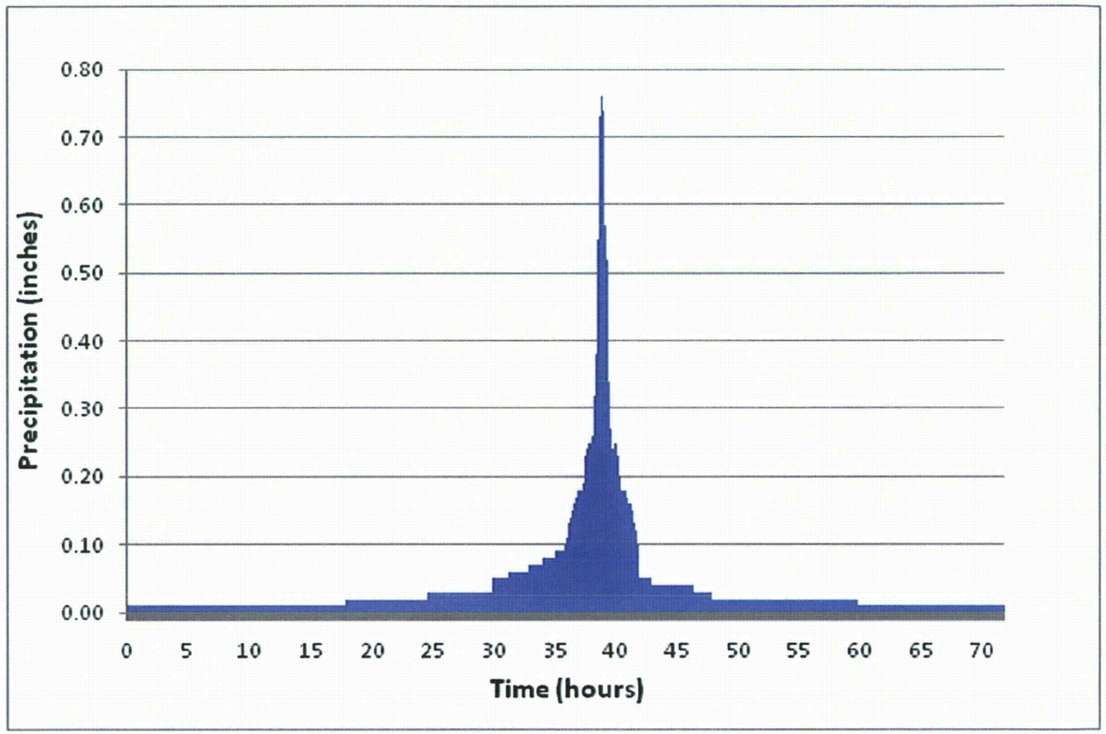
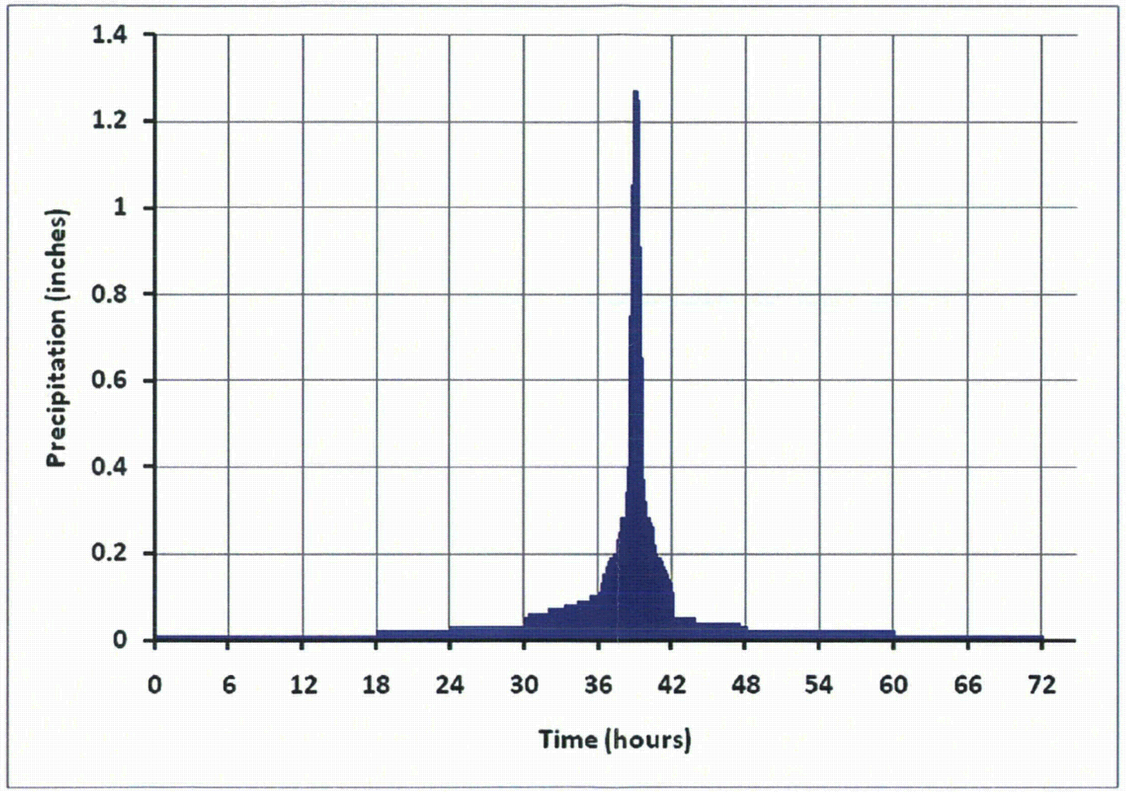


PROBABLE MAXIMUM STORM FOR PALUXY RIVER WATERSHED



PROBABLE MAXIMUM STORM FOR SQUAW CREEK WATERSHED

FIGURE 3-7

WATERSHED
HYETOGRAPHS

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

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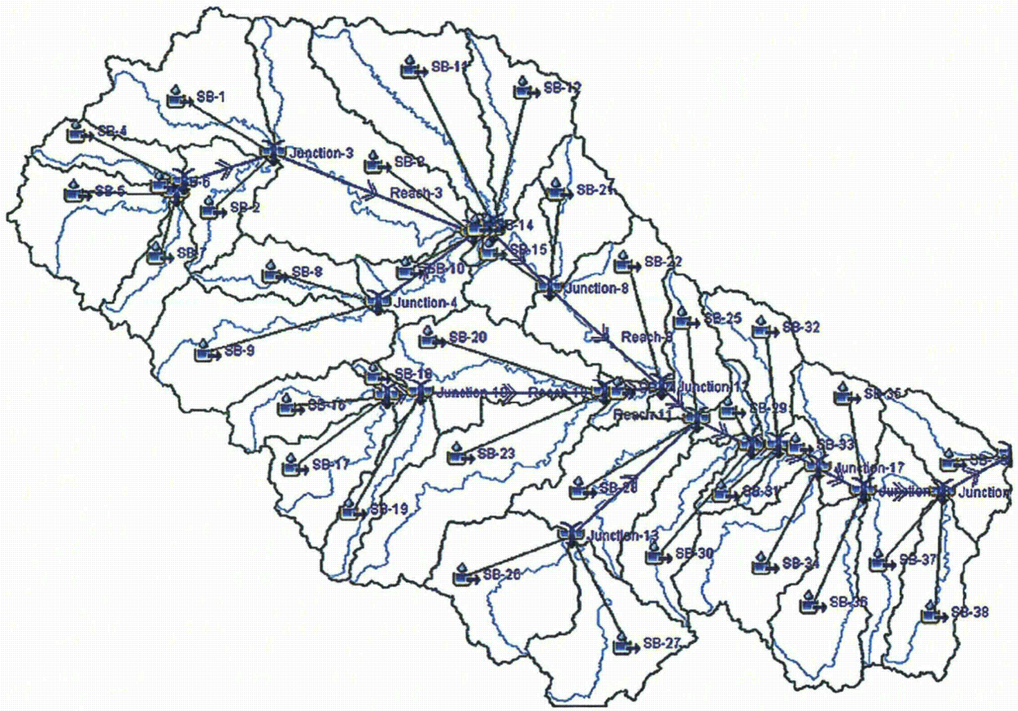
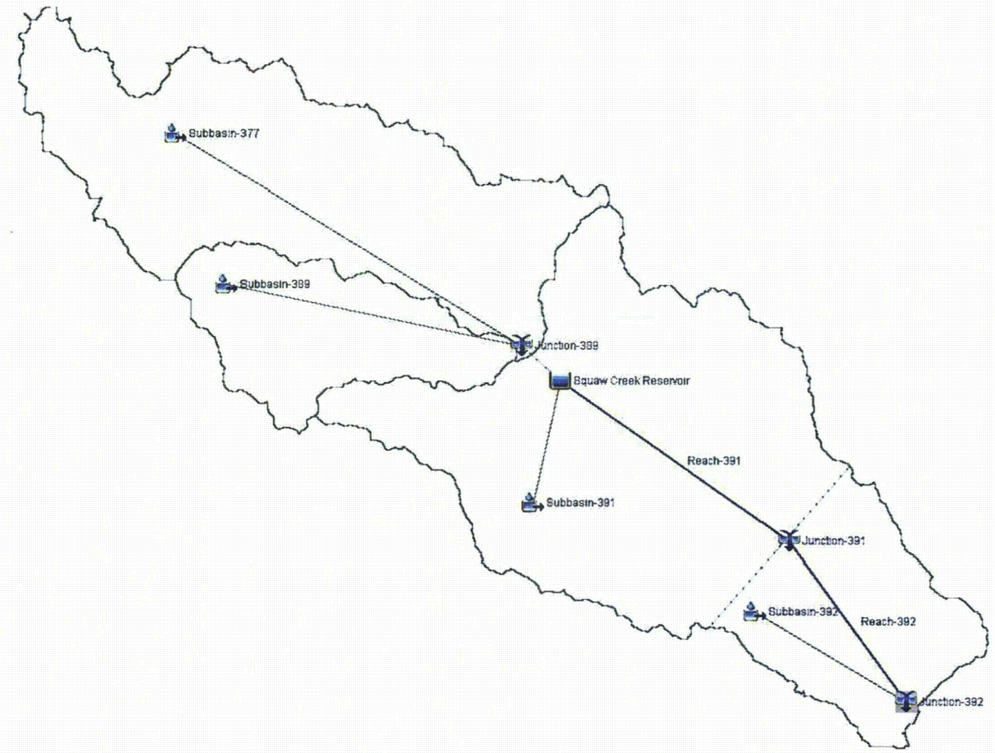


PALUXY RIVER WATERSHED
(NOT TO SCALE)



SQUAW CREEK WATERSHED
(NOT TO SCALE)

LEGEND:

SUBBASIN

SB-1, 391 SUBBASIN NUMBER

RIVER REACH

JUNCTION

HYDRAULIC CONNECTION

SUBBASIN DELINEATION

NOTE:

Squaw Creek Watershed HEC-HMS model set up for case with reservoir with additional basin.

FIGURE 3-8

HEC - HMS MODELS FOR THE WATERSHED DRAINAGE ANALYSIS

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

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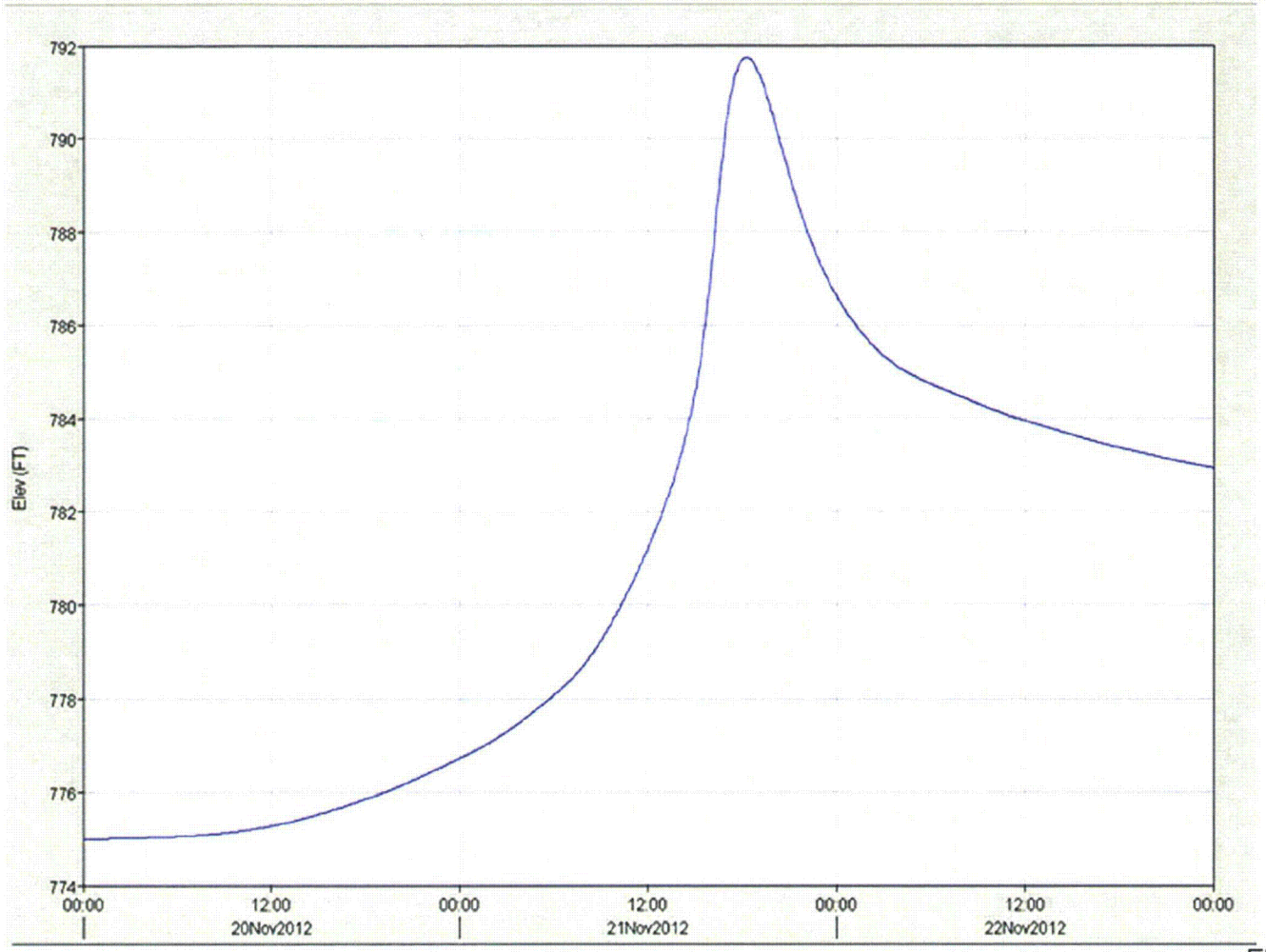

POOL ELEVATION IN SQUAW CREEK RESERVOIR FOR SCENARIO 13

FIGURE 3-9

FLOOD LEVEL TIME HISTORY OF THE SQUAW CREEK RESERVOIR FOR RIVER FLOODING ANALYSIS

PREPARED FOR

COMANCHE PEAK FLOODING HAZARD REEVALUATION REPORT

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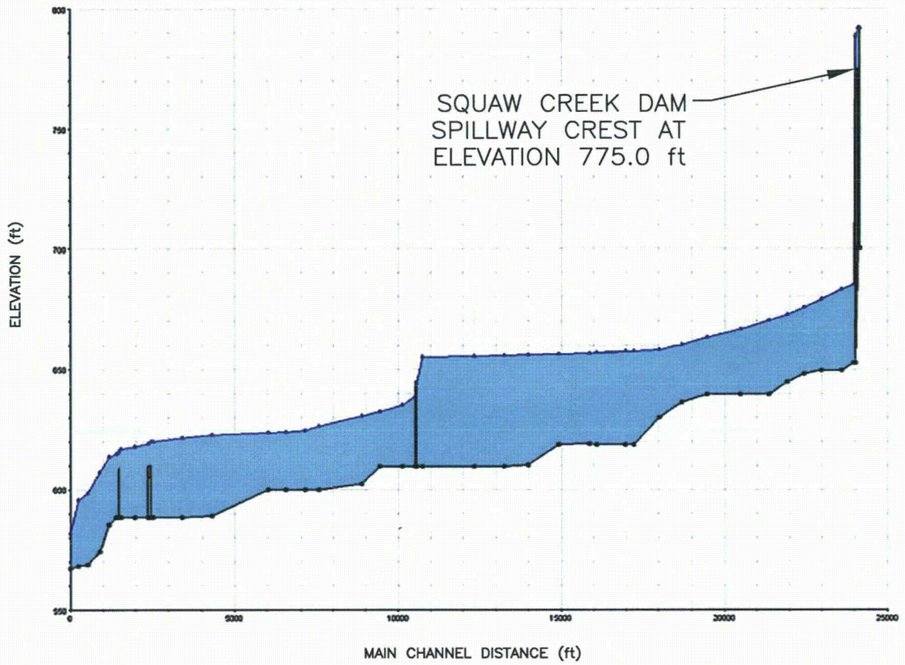
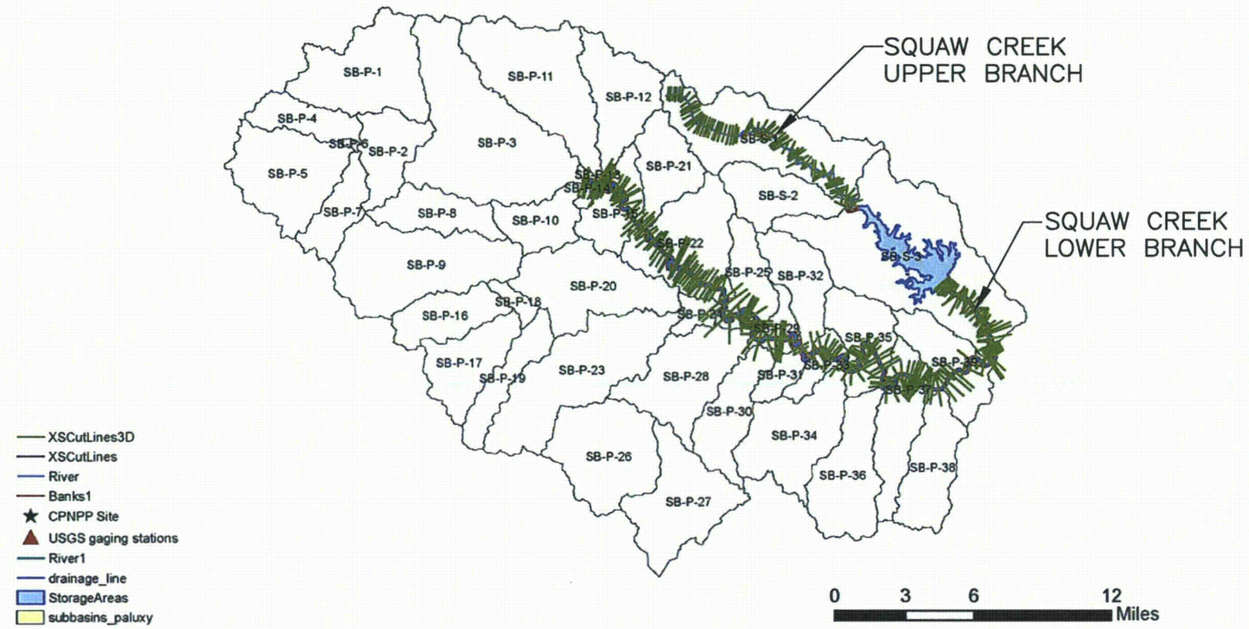


SQUAW CREEK LOWER BRANCH WATER SURFACE PROFILE
(NOT TO SCALE)



HEC-RAS MODEL
(NOT TO SCALE)

LEGEND

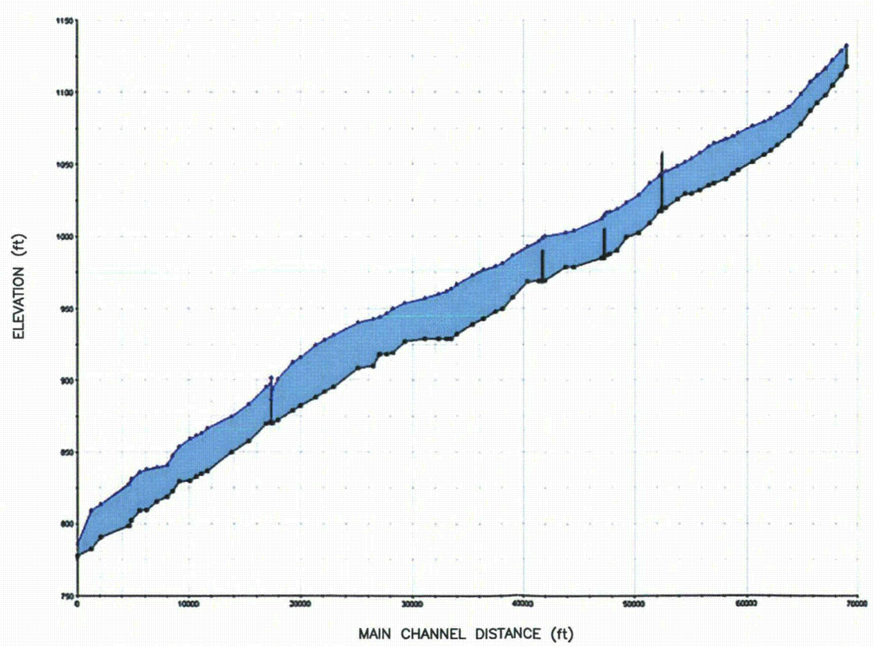
HEC–RAS SCENARIO 4 EXISTING CONDITIONS

HEC–RAS SCENARIO 5 PROJECTED CONDITIONS

GROUND

NOTE:

WATER SURFACE PROFILES FOR SCENARIOS 4 AND 5 APPEAR TO OVERLAP AT THIS SCALE.



SQUAW CREEK UPPER BRANCH WATER SURFACE PROFILE
(NOT TO SCALE)

REFERENCE:

LUMINANT GENERATING COMPANY, LLC (LUMINANT), 2012, "FINAL SAFETY ANALYSIS REPORT (FSAR) COMANCHE PEAK NUCLEAR POWER PLANT (CPNPP) UNITS 3&4 (FSAR)," REVISION 3, JUNE, 2012.

FIGURE 3–10

HEC–RAS MODEL FOR
RIVER FLOODING

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

Paul C. Rizzo Associates, Inc.
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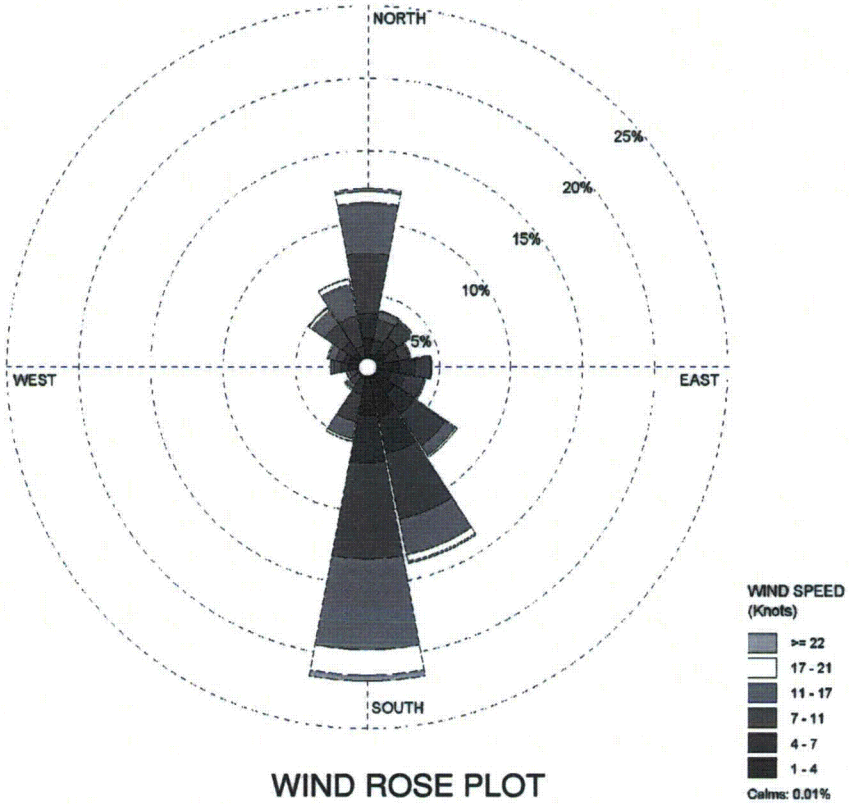
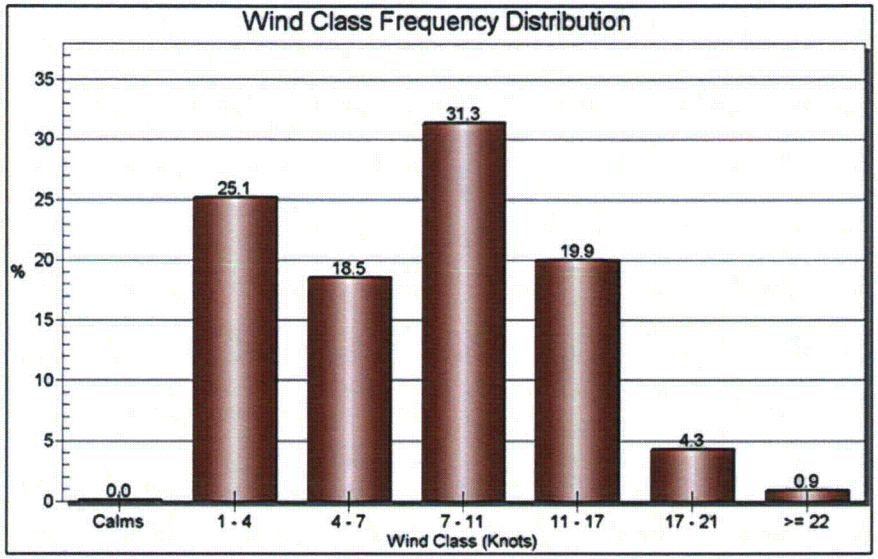


WIND ROSE PLOT



WIND SPEED
FREQUENCY DISTRIBUTION

FIGURE 3-11

DIRECTION AND SPEED OF
WIND AT THE SITE

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

Paul C. Rizzo Associates, Inc.
ENGINEERS / CONSULTANTS / CM

REFERENCE:

(NCDC, 2012), "WIND SPEED AND DIRECTION DATA" WEBSITE:
hppt://wwwl.ncdc.noaa.gov/pub/orders/8408665848446dat.txt

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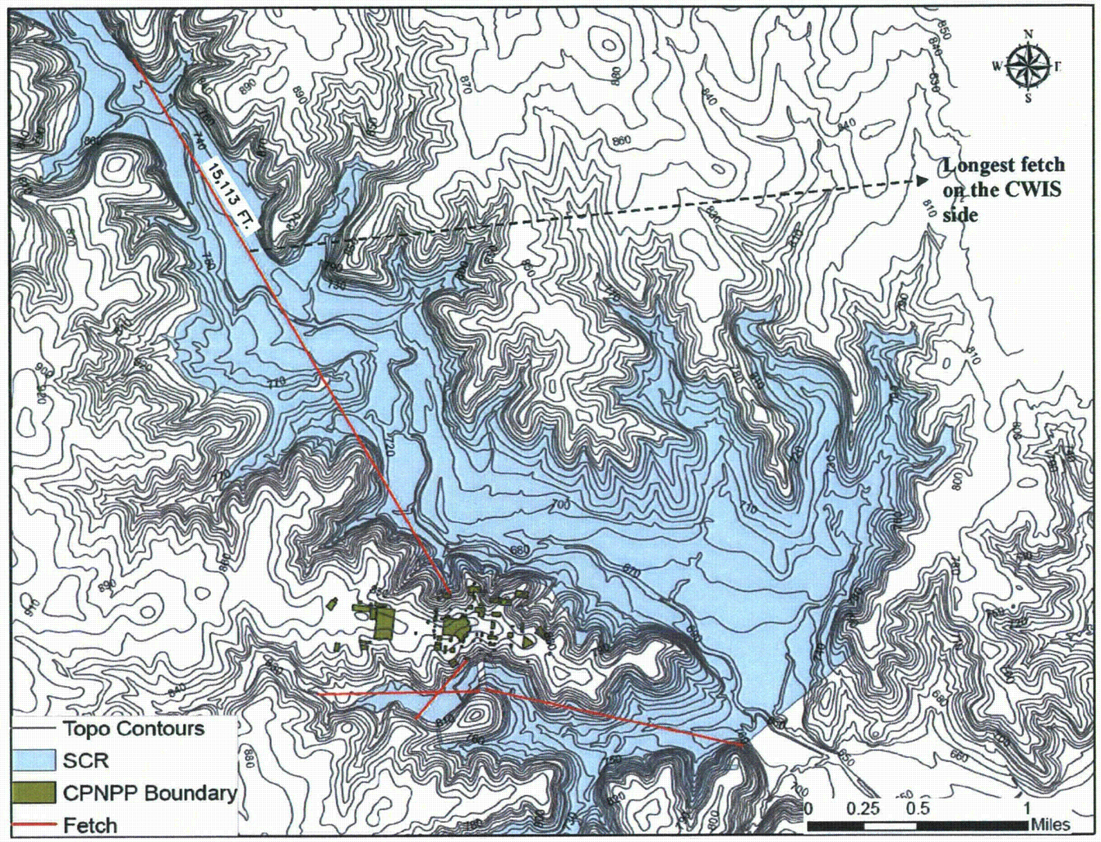
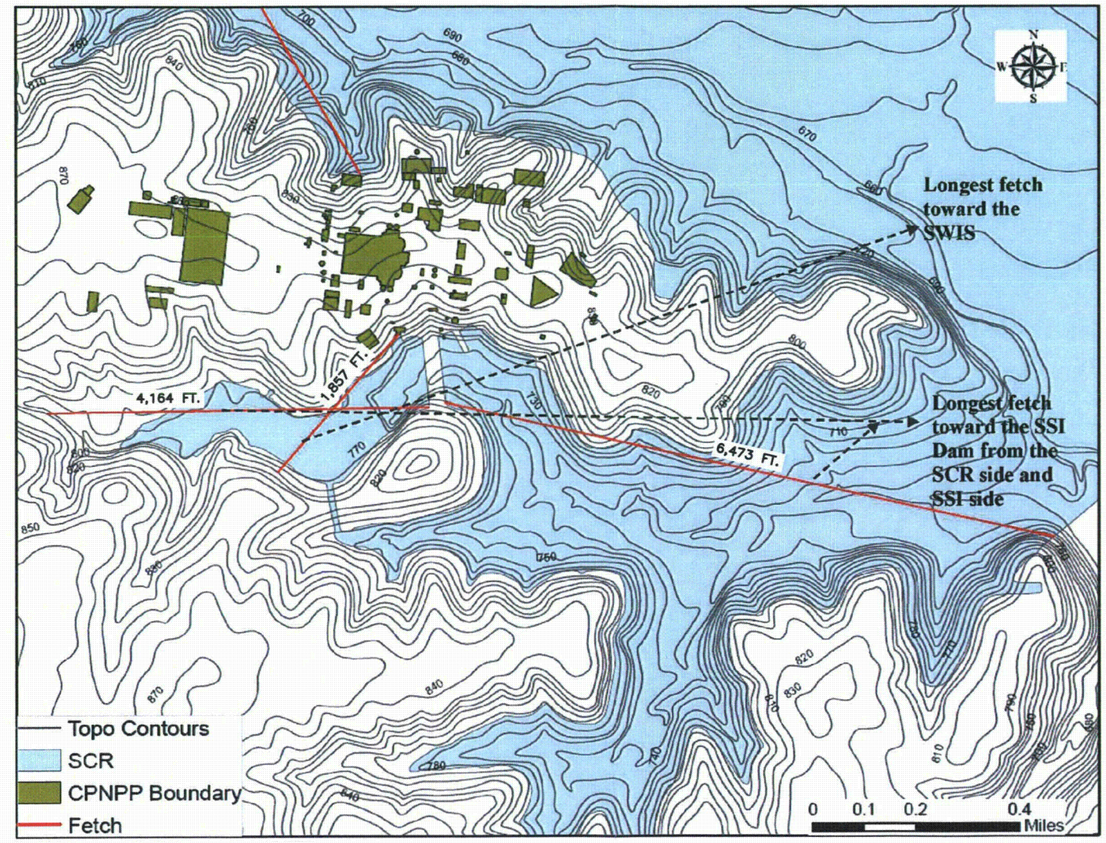
CVL 02-08-13	JML 02/11/13	12-4891-B9
	APPROVED BY CJE 02/11/13	NUMBER



FETCH LENGTH ON THE CIRCULATING WATER INTAKE STRUCTURE SIDE CPNPP 1 & 2



FETCH LENGTH TOWARD THE SURFACE WATER INTAKE STRUCTURE AND ON THE SAFE SHUTDOWN IMPOUNDMENT DAM FROM THE SQUAW CREEK RESERVOIR SIDE AND THE SAFE SHUTDOWN IMPOUNDMENT SIDE OF CNNPP 1 & 2

FIGURE 3-12

FETCH LOCATIONS OVER THE SQUAW CREEK RESERVOIR AND SAFE SHUTDOWN IMPOUNDMENT

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

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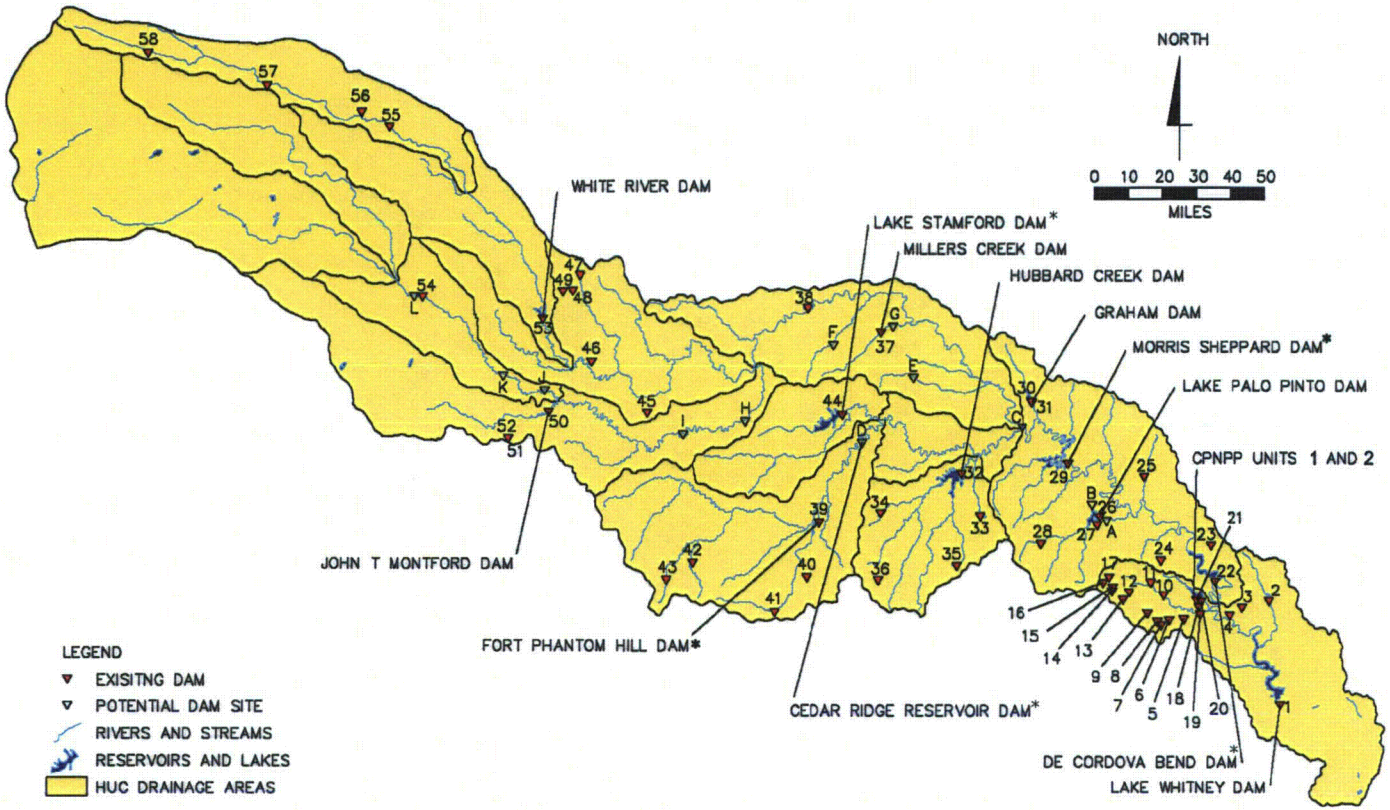
		APPROVED BY	CJE	02/11/13		



NOTE:

* DAMS USED IN DAM BREAK ANALYSIS

REFERENCE:

LUMINANT GENERATING COMPANY, LLC (LUMINANT), 2012, "FINAL SAFETY ANALYSIS REPORT (FSAR) COMANCHE PEAK NUCLEAR POWER PLANT (CPNPP) UNITS 3&4 (FSAR)," REVISION 3, JUNE, 2012.

FIGURE 3-13

LOCATION OF DAMS NEAR THE SITE

PREPARED FOR

COMANCHE PEAK FLOODING HAZARD REEVALUATION REPORT

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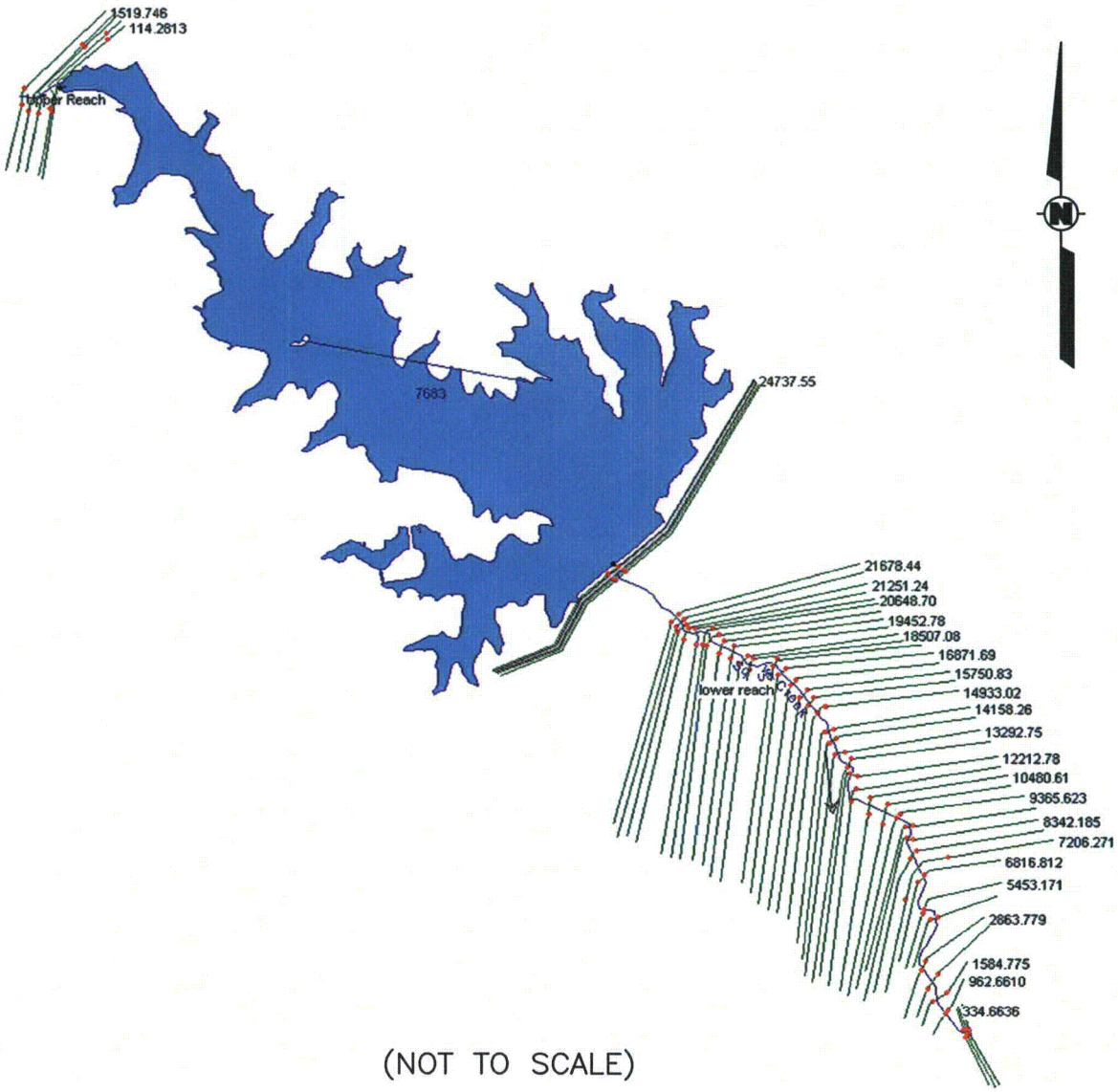
	11-30-12	APPROVED BY	CJE	02/11/13		

HEC-RAS MODEL SCHEMATIC FOR SQUAW CREEK (UNSTEADY STATE)



(NOT TO SCALE)

LEGEND:

RIVER/STREAM

CROSS-SECTIONS

FIGURE 3-14

HEC-RAS MODEL
FOR DAM BREAK ANALYSIS

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

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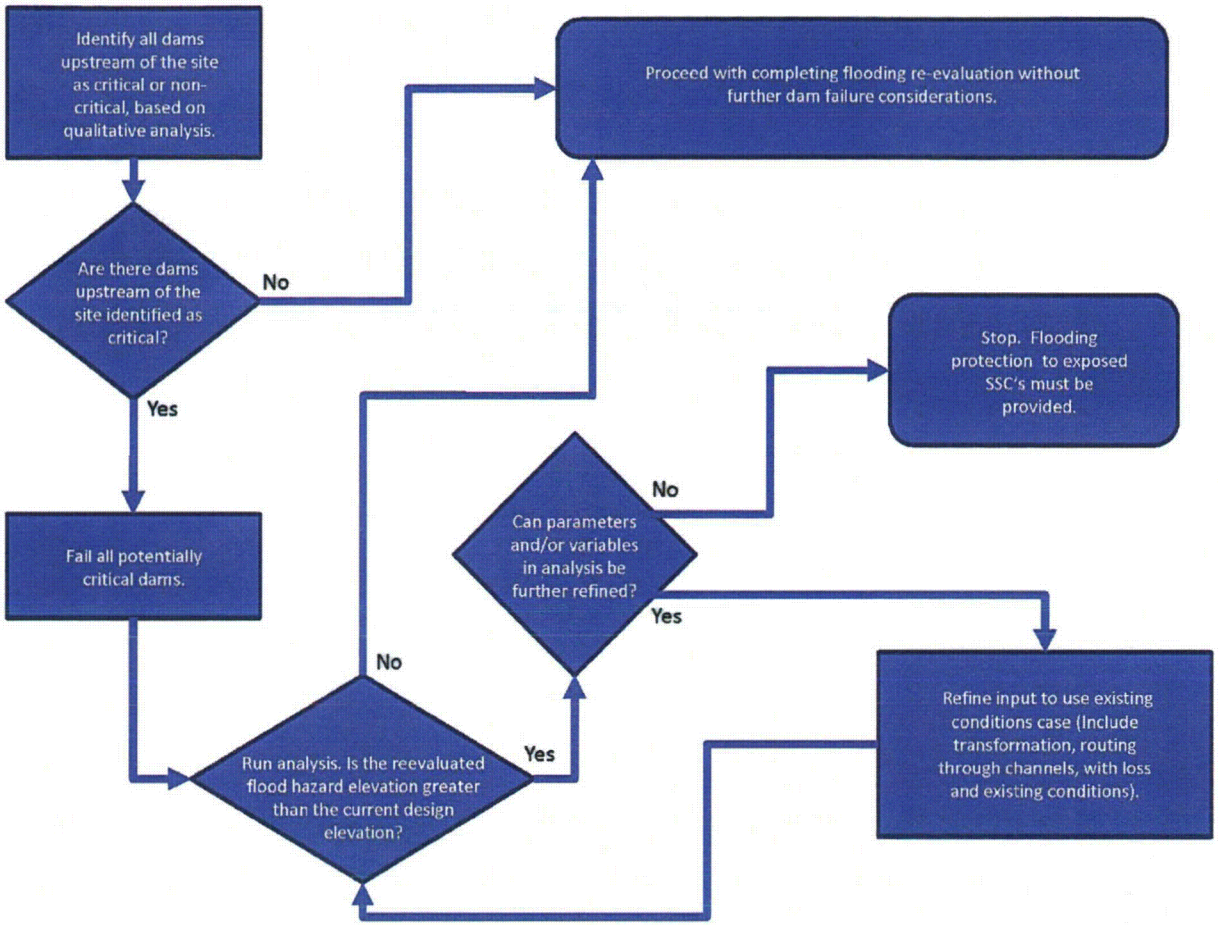

FIGURE 3-15

THE HHA DIAGRAM FOR DAM BREAK ANALYSIS

PREPARED FOR

COMANCHE PEAK FLOODING HAZARD REEVALUATION REPORT

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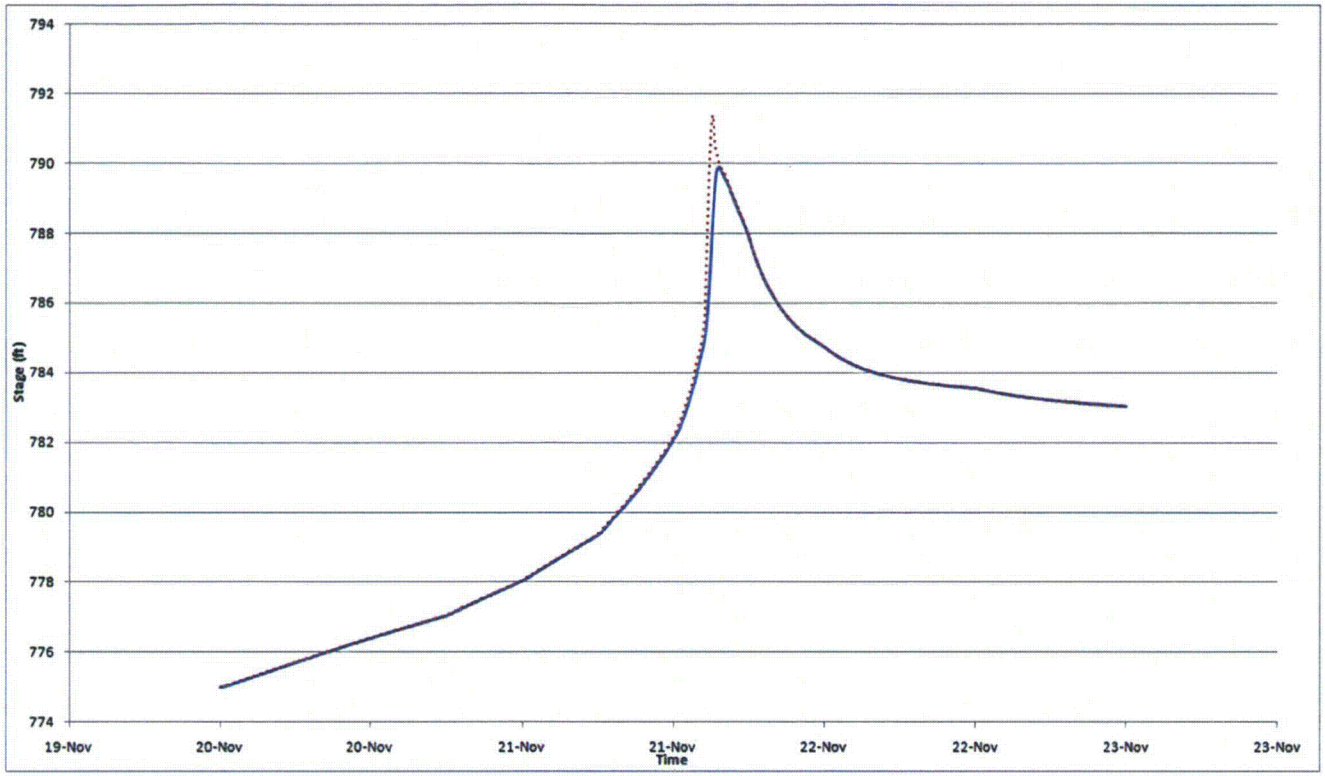
	12-28-12	APPROVED BY	CJE	02/11/13		



POOL ELEVATION IN SQUAW CREEK RESERVOIR FOR SCENARIO 1

LEGEND

STORAGE AREA 7683 AT SCR

STORAGE AREA SSI

FIGURE 3-16

FLOOD LEVEL TIME HISTORY OF THE SQUAW CREEK RESERVOIR FOR DAM FAILURE ANALYSIS

PREPARED FOR

COMANCHE PEAK FLOODING HAZARD REEVALUATION REPORT

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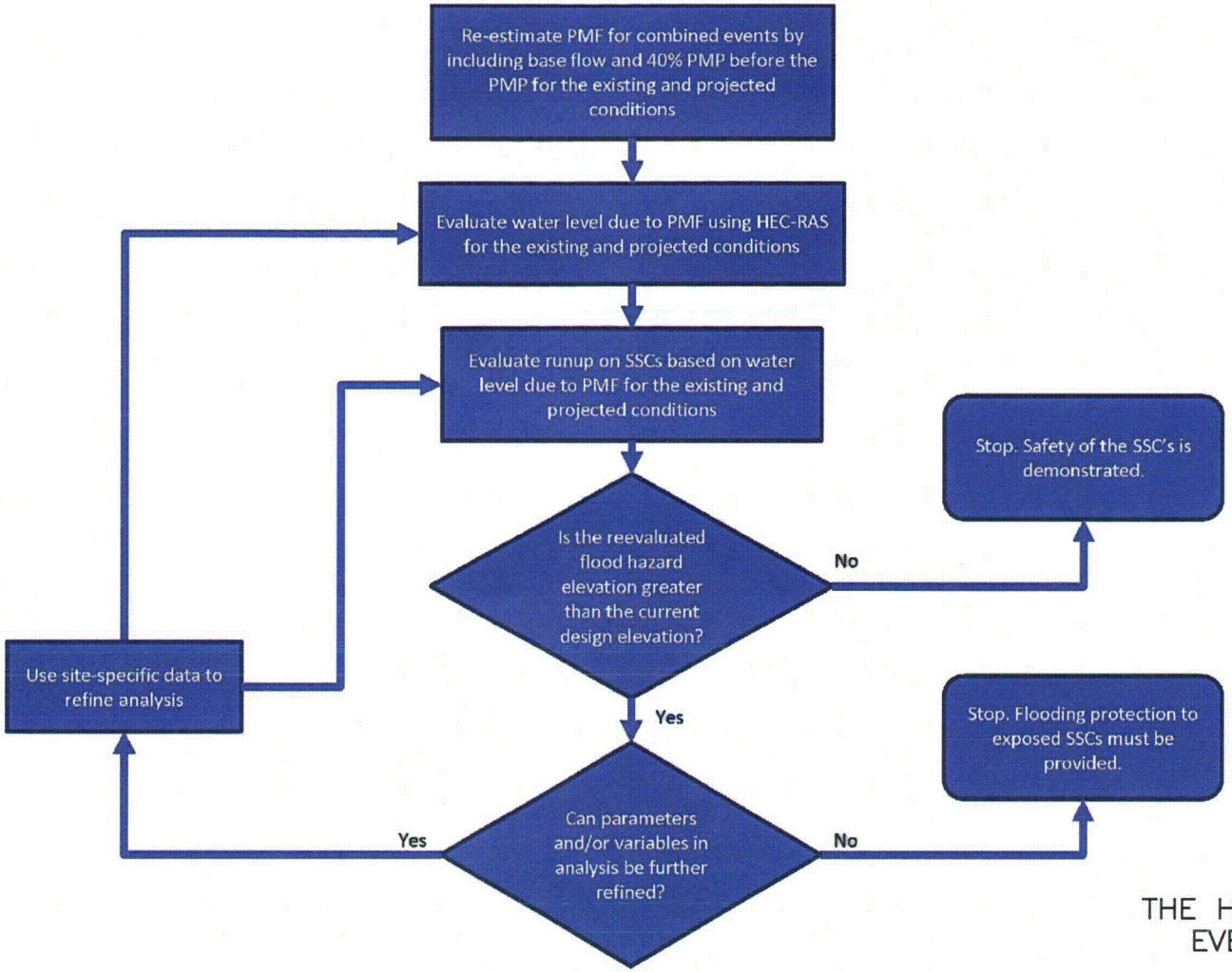

FIGURE 3-17

THE HHA DIAGRAM FOR COMBINED EVENTS FLOODING ANALYSIS

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

Paul C. Rizzo Associates, Inc.
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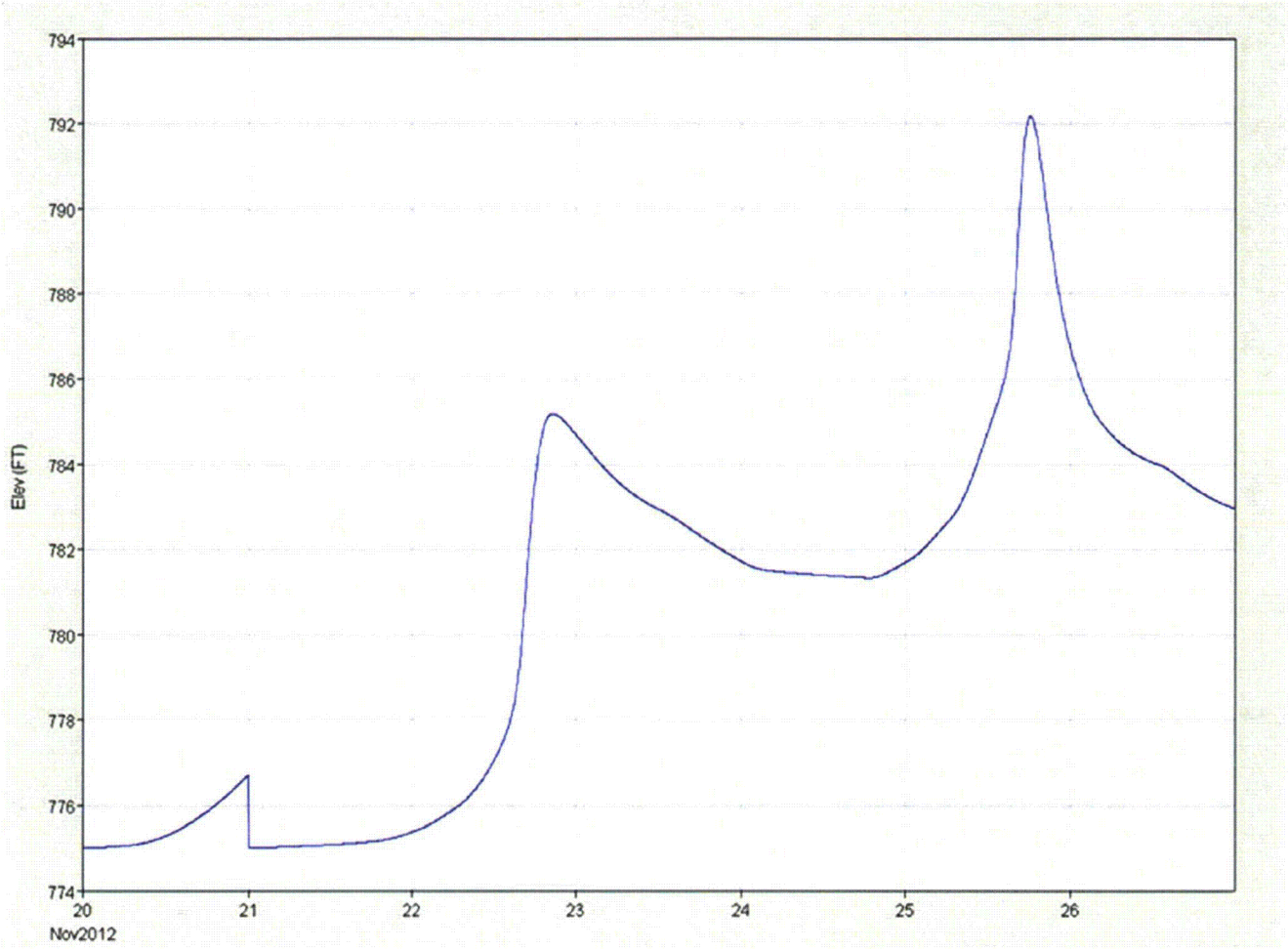
	12-28-12	APPROVED BY	CJE	02/11/13		



POOL ELEVATION IN SQUAW CREEK RESERVOIR FOR SCENARIO 4

FIGURE 3-18

FLOOD LEVEL TIME HISTORY OF THE SQUAW CREEK RESERVOIR FOR COMBINED EVENTS ANALYSIS

PREPARED FOR

COMANCHE PEAK
FLOODING HAZARD REEVALUATION REPORT

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